Sediment TMDL calculations for Amite River

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SEDIMENT TMDL CALCULATIONS FOR AMITE RIVER

A Thesis

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
Master of Science in Civil Engineering

in

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By
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Abstract

The Amite River is recognized as one of the 15 water bodies impaired by sediments in Louisiana. A sediment TMDL calculation for the Amite River is required by the EPA. Based on EPA’s Protocol sediment TMDL calculations for the Upper Amite River are conducted in this thesis. The sediment TMDL calculations are composed of four parts: (1) Development of a new sediment transport and dispersion model for the Amite River, (2) Estimation of sediment loads (sources) produced by watershed erosion, (3) Flow computation, and (4) Determination of sediment TMDL for the Amite River. Using the mass conservation principle and Reynolds transport theorem a new model has been developed for computation of sediment transport in the Amite River. Sediment erosion in the Amite River Basin is calculated by combining the USLE (Universal Soil Loss Equation) model with GIS and the digital elevation model of the Amite River Basin. Digital elevation data was imported into the GIS. The calculated soil erosion rate for the Upper Amite River Basin is 5.42 ton/acre/year, producing sediment load of 0.103 kg/m³ to the Amite River. The flow computation is performed under steady and unsteady flow conditions using the HEC-RAS software developed by the U.S. Army Corps of Engineers. Under the steady flow condition the computed sediment concentration varies in the range of 3-114mg/L. The numeric target criterion was not to exceed 50 NTU or 64 mg/L of suspended sediment. Based on this criterion and the new model developed in the thesis, the sediment TMDL calculations were conducted for steady and unsteady flow. It is found that there is significant difference between TMDLs for steady and unsteady flow due to high sediment loads produced by unsteady flow. It is recommended that (1) sediment TMDL calculation need to take account of the influence of unsteady flow; (2) Sediment
criteria for the Amite River can be met by adopting practices such as terraces on the steep slopes, creation of buffer zones along the river. Results indicate that the new model can be an effective tool for sediment TMDL calculations.
Chapter 1: Introduction and Literature Review

1.1 Introduction

United States has a major and lengthy history of federal water legislation which dates back to nineteenth century. The first major act was passed in 1886 and was called the River & Harbor Act. However, it is in recent years that most of the major water pollution legislation has been passed. Prior to 1972, water pollution control efforts were based on achievement of ambient water quality standards. This approach did not result with much success, as water quality modeling was rudimentary at that time. It was in 1972 that the Federal Water Pollution Control Act was amended and the focus shifted from water pollution control by ambient water quality standards to effluent limitations. This led to publication of guidelines for effluent limitations, waste load allocations, and advocating of minimum level of control based on available treatment technology. The result was substantial reduction in pollution. However, such an approach even though effective, had shortcomings, especially, dealing with complex water pollution problems. In 1977, the Clean Water Act was passed and water quality standards for hazardous and toxic substances were prepared. It supported the approach of secondary treatment required for municipal wastewater which resulted in consequential improvement in the water quality of many natural water systems. There were, however, some water bodies which were water quality limited and for that Section 303 of the Clean Water act outline additional work leading to the formulation of Total Maximum Daily Load (TMDL). Section 303 provisions augmented the process of state established water quality standards to a very large extent. One of its provisions, Section 303(d), made it mandatory for the state to identify, prioritize and establish TMDLs for the waters which would remain polluted even after the application of effluent limits. Fig. 1 shows the developments made in water quality modeling leading to the development of TMDL.
This list is to be submitted from time to time. In the event the state fails to develop the list or to develop TMDLs, the EPA is obligated to do so.

1.1.1 Total Maximum Daily Loads (TMDLs)

The EPA defines a TMDL as “a calculation of the maximum amount of a single pollutant – from all contributing point and non point sources- that a water body can receive and still meet water quality standards [without adverse impact to fish, wildlife, recreation, or other public uses]...” The calculation includes a margin of safety and accounts for seasonal variations in water quality.
A load is the quantity of input matter entering the water environment. If the load is human induced, it is termed as pollutant and if it is natural then it is the background load. The load capacity of a water body is the greatest amount of loading, which the body can possibly take without violating water quality standards. Its value is not perpetual and is therefore determined on variable flow and temperature variations. It is possible to divide the load capacity to various quantities. The load allocation (LA) is the component of load capacity, whose allocation is made to existing or future non point sources of pollution or to natural background sources. Accurate estimates of LA depend on availability of data and in cases where data availability is not adequate gross allotments are made. A waste load allocation (WLA) is the component of load capacity, whose allocation is made to existing or future point sources of pollution. Margin of safety (MOS) is a reserved component of load capacity, whose allocation is to account for modeling uncertainty, data inadequacies, and future growth and safety. The Total Maximum Daily Load (TMDL) for a substance is the sum of the individual WLAs for point sources, the LAs for nonpoint sources and for natural background, and the MOS. The TMDL is less or equal to the load capacity. The expression for TMDL in terms of various loadings can be given by

\[ \text{TMDL} = \text{LA (Nonpoint & Natural)} + \text{MOS (Reserved)} + \text{WLA (Point sources)} \]

1.1.2 Description of TMDL

A TMDL has thus been described as essentially a “pollution budget designed” to restore the health of the polluted body of water. When a stream or a specific water body does not meet the water quality standards (WQS), it is necessary to conduct a study which would aid in ascertaining the amount of pollutants that can be put in a water body from point sources and non-point sources, satisfying all the water quality specifications and also incorporating a factor of safety. A TMDL is thus a short acronym to describe the process.
Pollutants enter the water environment by two ways: point sources and non point sources. Water discharged from a pipe or a drain constitutes a point source. Point sources are easily identifiable and most of the industrial waste water and sewage waste water is discharged from the point sources. Nonpoint sources, on the other hand, are not easily identifiable as they generally prevalent over a wide area and are associated with various topographic features like slope, land use etc. It is more difficult to control non point source pollution than point source pollution. The first major step in the determination of TMDL of a particular pollutant is the identification of all point and non point sources. Because of the fundamentally different nature of the two sources, determination of TMDL is a difficult process.

1.1.3 Components of TMDL Development

The following components of TMDL development may be completed concurrently or iteratively depending on the site-specific situation

(a) Problem identification
(b) Identification of water quality indicators and target values.
(c) Source assessment
(d) Linkage between water quality targets and sources
(e) Allocations
(f) Follow-up monitoring and evaluation plan
(g) Assembling the TMDL

1.1.4 Water Quality Modeling for TMDL

Once a pollutant enters the water body, then the entire system may undergo various changes and transformation depending upon the quantity and nature of the pollutant. This whole process of transformation and changes is very complex as it may involve various sub processes, reactions,
mass transfer kinetics, degradation and resuspension etc. All these factors render water quality as non static both in temporal and spatial terms. Thus, to describe all these factors and link them together various water quality models are necessary for the development of TMDLs.

1.2 Literature Review

1.2.1 Challenges of Meeting TMDL Requirements

Federal requirements for TMDLs have presented many challenges. Over the past thirty years, regulators have focused primarily on technological improvements to help reduce point sources of water pollution. Despite reductions in point source loadings, water quality problems have persisted and in 1996, several non-governmental organizations (NGOs) filed suit against the United States Environmental Protection Agency (EPA), for not forcing the state to carry out provisions of TMDLs. The Lawsuits have succeeded in requiring EPA, to establish TMDLs. Section 303(d) of the Clean Water Act provides that states, territories, and authorized tribes are to list waters for which technology based limits alone do not ensure attainment of water quality standards. Regulations of the U.S. Environmental Protection Agency (USEPA) for implementing section 303(d) are codified in the Water Quality Planning and Management Regulations at 40 CFR Part 130, specifically in sections 130.2, 130.7, and 130.10 (USEPA, 1999). Once streams are listed, states, territories, and authorized tribes must establish TMDLs that will meet water quality standards for each listed water body, considering seasonal variations and a margin of safety. According to the USEPA, agriculture is the largest source of water quality impairment in the United States and therefore TMDLs also must include a description of the control actions and/or management measures that will be implemented to achieve the required pollutant loads. TMDLs must be appropriate to the characteristics of the water body and pollutant. The maximum allowable pollutant load may be expressed as daily, monthly, seasonal, or annual
averages (USEPA, 1999). Louisiana is among the 23 states in which EPA was under court order or agreed in consent decree to establish TMDLs if state department does not establish TMDLs.

1.2.2 Effect of Sediment on Water Quality

It is of common knowledge, that sediment is a carrier of pollutants. However, the question is how safely it can be concluded that sediment to be termed as a pollutant. The answer to this lies in, the reason that excess sediment in the river has often resulted in degradation of water quality and subsequent loss of aquatic life. Hence, from water quality point of view, sediment can be termed as “pollutant”. Current agricultural practices, primarily row crop agriculture, have led to increased sediment that influences the structure and function of streams and often results in changes in fish diversity, altered size, age structure, species composition and greater temporal variability in fish abundance (Berkman and Rabeni, 1987; Schlosser, 1991; Wang et al., 1997; Harding et al., 1998; Schleiger, 2000). Suspended sediment may cause avoidance behavior, impaired respiration, reduced feeding rates and growth, reduced tolerance to disease or toxicants, increased physiological stress, and mortality (Newcombe and Jensen, 1996). Suspended sediment can also skew fish assemblages toward sediment tolerant species (Poff and Allan, 1995). Excess bed sediment reduces the depth of pools and decreases the complexity of stream substrate by covering gravel, cobbles, and boulders (Saunders and Smith, 1965; Berkman and Rabeni, 1987; Paragamian, 1989; Schlosser, 1991; Richards and Host, 1993; Wood and Armitage, 1997). Although hydrological processes generate some sediment loading in minimally altered systems, excessive sediment is a pollutant and can have negative effects on aquatic biota (Waters, 1995).

Sediments can cause taste and odor problems, block water supply intakes, foul treatment systems, and fill reservoirs. Although most treatment systems can remove most turbidity, very
high sediment levels sometimes require that water supply intakes be shutdown until turbidity clears or system maintenance (e.g., backflushing) is performed. High levels of sediment can impair swimming and boating by altering channel form, creating hazards due to reductions in water clarity, and adversely affecting aesthetics. Because of the range of environmental impacts, there are no nationwide water quality criteria for acceptable amounts of sediment, although EPA has began the process to develop sediment criteria. As such, it is a technically challenging step of defining an acceptable level of sediment in the water as this target can vary substantially from water body to water body because of the absence of numeric sediment criteria.

1.2.3 Development of Sediment TMDL

An overview of current TMDLs by the EPA shows that over 40% of the United States assessed waters still do not meet the water quality standards which the states, territories, and authorized tribes have set for them. This amounts to over 20,000 individual river segments, lakes, and estuaries. These impaired waters include approximately 300,000 miles of rivers and shorelines and approximately 5 million acres of lakes -- polluted mostly by sediments, excess nutrients, and harmful microorganisms from nonpoint sources. In fact, the largest water pollutants in the United States, by volume, are instream suspended sediment and bed load (Fowler & Heady, 1981).

Developing TMDLs is a rather complex process that includes problem identification, target values, source assessment, linkage between targets and sources, allocations among sediment inputs, monitoring, and evaluation plans. Most sediment related 303(d) listings result from suspended or deposited sediments at levels detrimental to designated uses, including aquatic life, water supply, and recreation (USEPA, 1999). Mathematical models are one of the best tools available for determining the quantitative relationship between pollutant sources and water quality criteria. They can not only be used for calculating watershed loads for existing conditions
but also relate loads to water quality response, and evaluate the effectiveness of proposed control alternatives in reducing loads and improving water quality to meet standards. However, TMDL modeling is not being properly implemented, and that improvements are needed (De Pinto et al., 2004). Several evaluations have identified the need for improvement in the modeling applications in the TMDL process (NRC/NAS, 2001; Water Environment Federation 2001; Limno-Tech, Inc. et al., 2002; U.S. EPA, 2002). In January 2000 the first sediment TMDL documents developed in Georgia were released by the United States Environmental Protection Agency. Protocols were proposed for establishing sediment TMDLs when time and data are very limited (Phase I TMDLs) and when time and data are less limited (Phase II TMDLs). Numerous models exist to support TMDL development. One of the most popular pollution models is AGNPS (Agricultural Non-Point-Source). AGNPS is an event-based model. It calculates runoff from agricultural watershed and transport processes of sediment, nitrogen, phosphorous, and COD. Sediment runoff is estimated from the modified version of USLE (Universal Soil Loss Equation) and its routing is performed for five particle size classes. The application of AGNPS is limited to about 200 km² watersheds (Young et al., 1989, DeVries and Hromadka, 1993, Engel et al., 1993). QUAL2E model uses a finite-difference solution of the advective-dispersive mass transport, reaction, and sink/source equation. The stream network is divided into headwaters, reaches, and junctions. The changes in flow conditions are represented as a series of steady-flow water profiles. Such parameters as velocity, cross-sectional area, and water depth that are required for the mass transport calculations are computed from the flow rate. For each river reach, QUAL2E requires specification of as many as 26 physical, chemical, and biological parameters. (DeVries and Hromadka, 1993, Camara and Randal, 1984, Schoellhamer, 1988). Compiling such data at a regional scale would take a very great investment of time and resources. In 1993 Arnold, Engel and Srinivasan (from Mamillapalli et al., 1996) developed a
new version of the SWRRB--Soil Water Assessment Tool (SWAT). In SWAT, the watershed can be divided into practically unlimited number of cells and/or subwatersheds. New features have been added such as routing of the flow through the basin streams and reservoirs, simulating lateral flow, groundwater flow, stream routing transmission losses, modeling sediment and chemical transport through ponds, reservoirs, and streams. The major components of the SWAT include weather, hydrology, erosion, soil temperature, crop growth, nutrients, pesticides, subsurface flow, and agricultural management. However none of the above models takes into account all the natural and physical processes and therefore not applicable for all situations.

1.2.4 Problem Identification and Objectives

The Amite River is identified as one of the 59 water bodies in Louisiana (EPA, 1992). The sediment TMDL calculation for the Amite River is required by EPA. The overall goal of this thesis is thus to conduct the sediment TMDL calculation for the Amite River. The Amite River Watershed is located in the Lake Pontchartrain Basin in southeast Louisiana. It consists of 2200 square miles and comprises St. Helena, East Feliciana, East Baton Rouge, Livingston, Iberville and Ascension parishes (USACE, 1997).

The currently recommended or used models for TMDL calculation by the Louisiana Department of Environmental Quality (Brent , 2001) do not include the method for modeling the transport of suspended sediment although the sediment oxygen demand is included. Most of existing models for TMDLs generally use Fischer’s equation (Fischer et al, 1979) for estimation of the longitudinal dispersion coefficient. A significant progress has been made in the estimation of the longitudinal dispersion coefficient in rivers and streams. It is important and possible to find a more accurate method for estimation of the longitudinal dispersion coefficient for Amite River. Since there are no widely accepted modeling techniques available for the sediment pollution it is
therefore necessary to improve current TMDL models so that the sediment transport related pollution can be taken into full account.

Therefore, the objective of this thesis is to

(1) Determine sediment load into the Amite River.
(2) Determine the flow parameters for Amite River using HEC RAS.
(3) Develop a new sediment transport model for determination of suspended sediment concentration in the Amite River.
(4) Determine the sediment TMDL for Amite River.
Chapter 2: Development of 1-D Cohesive Sediment Dispersion and Transport Model

2.1 Derivation of 1-D Cohesive Sediment Dispersion and Transport Model

In order to calculate the TMDLs in Louisiana waterways it is essential to determine the transport and fate of the suspended sediment in the waterways. To that end, a sediment dispersion and transport model is necessary. Although a number of sediment dispersion and transport models are available, these models are generally developed for noncohesive sediments. However, the impairment of the Amite River is mainly caused by cohesive sediments. It is therefore necessary to analyze the mechanisms involved in the soil erosion/deposition and sediment transport and to derive an equation for description of the rainfall-induced overland soil erosion and deposition processes and sediment transport. Taking a short channel reach as a control volume as shown in Fig. 2 using the principle of mass conservation and the Reynolds transport theorem, the following model is derived to simulate sediment dispersion and transport in a channel network like the Amite River. The control volume is based on the following assumptions:

1. The flow and soil erosion/deposition and sediment transport is one dimensional;
2. Suspended sediments are fully mixed vertically at any location and thus the vertical sediment concentration gradient is negligible;
3. Sediment concentration gradient caused by the dispersion term is negligible as compared to other terms (Boardman et al. 1990).
4. The flow is incompressible.
5. Suspended sediments are fully mixed laterally.
Based on the assumptions and the Reynolds transport theorem, one dimensional mass conservation equation or continuity equation of suspended sediment in the overland flow on a unit width surface can be written as

$$\frac{\partial}{\partial t} \int_{CS} \rho \, dV + \int_{CS} \rho \, \mathbf{V} \cdot dA = 0$$

(2.1.1)

Where $\rho_s$ = sediment density $[ML^{-3}]$. The first integral represents the accumulated mass of sediment in the control volume. The second integral stands for the net mass efflux of sediment through the entire control surface. The control volume has six control surfaces and a uniform concentration $S$. Then, the equation (2.1.1) can be rewritten as

$$\frac{\partial}{\partial t} \int_{CS} \rho_s \, dV + \sum_{i=1}^{6} \int_{CS_i} \rho_s \mathbf{V} \cdot dA = 0$$

(2.1.2)

Where $\mathbf{V}$ is the velocity vector of flow, $B$ is the width. The equation states that the rate of accumulation of sediment mass in the control volume plus the net mass efflux through all controls surfaces is zero. The net efflux is the mass flow rate of sediment out of the control
volume minus the mass flow rate in. In other words, values of the integrals are negative if sediment entering the control volume and positive if leaving the control volume.

The net sediment entering or leaving the control surface CS1-CS2 is

\[ \int_{CS1} \rho_s V dA + \int_{CS2} \rho_s V dA = \left( \frac{\partial(uS)}{\partial x} - \frac{\partial}{\partial x} \left[ (K_c \frac{\partial S}{\partial x}) \right] \right) \Delta x B h \]  

(2.1.3)

The mass flow rate at control surface CS3 can be divided into two categories, namely, rate at which there is deposition of mass due to settling and the erosion in the bed.

Therefore, the net sediment entering or leaving the control surface CS3 is

\[ \int_{CS3} \rho_s V dA = \int_{cs3}^{DEPOSITION} \rho_s V dA + \int_{cs3}^{EROSION} \rho_s V dA \]  

(2.1.4)

Where

\[ \int_{cs3}^{DEPOSITION} \rho_s V dA = \beta \omega_s S \Delta x B \]  

(2.1.5)

\[ \int_{cs3}^{EROSION} \rho_s V dA = -\alpha (u_s - u_w)(S - S) \Delta x B \]  

(2.1.6)

The rate of mass transfer across the control surface CS4 is zero. The rate of mass transfer entering or leaving the control surface CS5-CS6, due to the lateral inflows and outflows can be given by

\[ \int_{CS5} \rho_s V dA + \int_{CS6} \rho_s V dA = q_L (S - S_L) \]  

(2.1.7)

After the substitution of the above equations in Equation 2.1.2. and dividing area A through out, the sediment transport model derived can be written as

\[ \frac{\partial S}{\partial t} + \frac{Q}{A} \frac{\partial S}{\partial x} = \frac{1}{A} \frac{\partial}{\partial x} \left( AK_x \frac{\partial S}{\partial x} \right) + \frac{\alpha (u_s - u_w)(S - S)}{h} - \frac{\beta \omega_s S}{h} + \frac{q_L}{A} \left( S_L - S \right) \]  

(2.1.8)

in which \( A = \) channel cross-sectional area \((L^2)\), \( S = \) sediment concentration \((M/L^3)\), \( S_L = \) sediment concentration of lateral inflow \((M/L^3)\), \( S_* = \) suspended sediment concentration under equilibrium
conditions or suspended-load carrying capacity (M/L³), \( Q \) = volumetric flow rate (L³/T), \( q_L \) = lateral inflow rate (L³/T-L), \( t \) = time [T], \( x \) = distance [L], \( K_x \) = longitudinal dispersion coefficient (L²/T), \( u^* \) = flow shear velocity (L/T), \( u_{*c} \) = critical flow shear velocity of boundary sediment (L/T), \( \omega_s \) = settling velocity of sediment particles (L/T), \( \alpha \) = constant , \( \beta \) = constant .

The main advantage of Equation (2.1.8) over existing 1-D sediment transport modules is that sediment erosion (described by the second term on the right hand side of the equation) and sediment settling (represented by the third term on the RHS) are treated as two different processes and thus modeled by two separate terms. The last term on the RHS of Equation (2.1.8) stands for the influence of tributaries on sediment transport in the Amite River.

2.2 Parameter Estimation

2.2.1 Estimation of Dispersion Coefficient

Longitudinally dispersion co-efficient is a necessary component in hydraulic modeling of river pollution and its determination gives an idea as to what extent the intensity of the mixing of pollutants is. The dispersion coefficient is therefore essential for approximating particle transport. By qualitative analysis Fischer et al (1979) proposed a method for estimation of longitudinal dispersion coefficient in rivers. Since then, a wide spectrum of methods for estimation of the longitudinal dispersion coefficient in streams has been proposed. Deng et al (2001) established an equation for prediction of the longitudinal dispersion coefficient in straight rivers. The equation is

\[
K_x = \frac{0.15}{8 \varepsilon_{t0}} \left( \frac{U}{u_*} \right)^2 \left( \frac{B}{H} \right)^{5/3} H u_*
\]

(2.2.1.1)

where \( \varepsilon_{t0} \) can be computed as give in Equation 2.2.1.2.
Here, $\varepsilon_{0.1} = 0.145 + \left( \frac{1}{3520} \right) \left( \frac{U}{u_*} \right) \left( \frac{B}{H} \right)^{1.38}$ \hspace{1cm} (2.2.1.2)

where

\begin{align*}
B &= \text{Surface width of river channel (m)} \\
H &= \text{Cross-sectional average flow depth (m)} \\
U &= \text{Cross-sectional average velocity (m/s)} \\
 u_* &= \text{Shear velocity (m/s)}
\end{align*}

The above equation has been recognized as the most accurate method available and has been increasingly used in prediction of longitudinal dispersion coefficient in rivers and streams (Chanson 2004, Scofolofsky and Jirka 2005). Therefore, the above equation is employed in this thesis for determination of the parameter $K_x$. Fig 66 in the Appendix shows the dispersion variation in the Amite River.

**2.2.2 Shear Velocity**

Shear velocity is a major factor, regarding the state of suspension of sediments. When shear velocity is higher, and the fluid turbulence picks up the displaced particles and keeps them in suspension.

Here $u_* = \text{shear velocity} = \sqrt{\frac{\tau_o}{\rho}}$ \hspace{1cm} (2.2.2.1)

Where

\begin{align*}
\tau_o &= \text{Shear stress (Kg/m}^2) \\
\rho &= \text{Density}
\end{align*}

The shear velocity is tangent to the wall surface. The concept in this is that roughness elements in the boundary introduce perturbation in the fluid movement and this perturbation is characterization by shear velocity.
2.2.3 Critical Shear Stress ($\tau_c$)

To reach the condition of critical motion or incipient motion, the bed shear stress corresponding to incipient motion is known as critical shear stress or critical tractive force and is designated by $\tau_c$. From many laboratory and field experiments, numerous empirical formulas have been obtained which related, critical shear stress with sediment properties like diameter and specific gravity. To use the Shields curve to estimate critical shear stress for a given particle with size less than 6mm, one has to adopt a trial and error procedure. Thus, it is difficult to use Shields curve directly in numerical modeling. Many other investigators and researchers have used explicit and implicit relationships, which can be useful for modeling purposes.

Chien and Wan (1983) developed a relationship between two non dimensional parameters

1. Shields parameter, which depends upon the critical shear stress
2. Non-dimensional representative diameter which depends upon sediment representative diameter $d_{50}$ to represent the Shields curve.

\[
\begin{align*}
\theta &= 0.126D_*^{-0.44}, D_* < 1.5 \\
&= 0.131D_*^{-0.55}, 1.5 \geq D_* < 10 \\
&= 0.0685D_*^{-0.27}, 10 \geq D_* < 20 \\
&= 0.0173D_*^{0.19}, 20 \geq D_* < 40 \\
&= 0.0115D_*^{0.30}, 40 \geq D_* < 150 \\
&= 0.052, D_* \geq 150
\end{align*}
\]

where

\[
D_* = d_*(\frac{g(s-1)}{v^2})^{1/3} = \text{non dimensional representative diameter}
\]

\[
\theta = \text{Shields parameter} = \frac{\tau_c}{(\gamma_s - \gamma)d_s}
\]

$\tau_c$ = critical shear stress in kg-m/s$^2$

$\gamma_s$; $\gamma$ = specific weight of sediment and water in KN/m$^3$

$s$ = specific gravity
\[ g = \text{gravitational acceleration in } m/s^2 \]

\[ \nu = \text{kinematic viscosity in } m/s^2 \]

In this relation both parameters are dimensionless so consistent unit can be used in the relationship.

### 2.2.4 Settling Velocity \((\omega_s)\)

The terminal velocity of sediment particles settling in liquids, often called the fall velocity, is an important physical quantity, which is used in characterizing sediment transport. The settling velocity of a single sphere falling with a constant velocity in quiescent water of large extent depends on force of gravity that acts on a sphere as it falls in water and the resistance to the motion. If the flow velocity is greater than the settling velocity, sediment will be transported downstream as suspended load. Thus sediment settling velocity would be the minimum flow velocity needed to transport the given sediment particle. Settled sediments termed as bed loads can be still be transported by mechanisms such as siltation, rolling and sliding.

Cheng (1997) proposed following formula to calculate the sediment settling velocity:

\[
\omega_s = \frac{\nu}{d} \left( \sqrt{\left(25 + 1.2 d_s^2\right)} - 5\right)^{1.5} 
\quad \text{(2.2.4.1)}
\]

\[
d_s = \left( \frac{s}{\nu^2} \frac{1}{d} \right)^{\frac{1}{3}} d
\quad \text{(2.2.4.2)}
\]

where

- \(\omega_s\) = sediment settling velocity in \(m/s\)
- \(d\) = sediment particle diameter in \(m\)
- \(s\) = specific gravity of sediment mixture
- \(\nu\) = Kinematic viscosity in \(m^2/s\)
- \(g\) = acceleration due to gravity \(m/s^2\)
2.2.5 Suspended Sediment Transport Capacity

Sediment transport capacity should be estimated. It can be defined as the maximum amount of suspended sediment, which water can carry at a given flow condition. Researchers at WIHEE (Wuhan Institute of Hydraulic and Electric Engineering) made an extensive analysis of field data collected from rivers and canals including Yangtze River, Yellow River, Yongding River, People’s Victory Canal, and Qingtong Xia Irrigation System; they concluded the following expression for suspended sediment capacity (Mechanics of Sediment transport).

\[
S_{VM} = k \left( \frac{U^3}{gh\omega} \right)^m
\]  
(2.2.5.1)

in which the coefficients \( k \) and \( m \) are functions of \( \frac{U^3}{gh\omega} \).

The above equation is used in the thesis to calculate the sediment carrying capacity involved in Equation (2.1.8). In Equation (2.2.5.1) the velocity \( U \) and depth are calculated using the HEC-RAS software.

Aquatic life in rivers and streams are sensitive to the variation of sediment concentration. Sediments in rivers are transported as bed load and suspended load. The bed load in rivers moves in sliding or rolling or jumping mode along the bed and will not affect turbidity of river waters. The suspended loads which moves in suspension and occupies the entire flow depth control the concentration of sediments in rivers. Therefore, suspended sediment concentration is employed as the targeted water quality index for Sediment TMDL development. There are a number of numerical models available for computation of suspended sediment concentration in rivers. Most of current models are developed using a differential control volume approach without taking account of the difference between sediment settling and erosion. The current approach is a step towards addressing such a problem.
2.3 Numerical Solution of 1-D Sediment Dispersion and Transport Model

The sketch for sediment transport calculation is shown below in Fig 3.

Fig 3: Sketch of steps to be taken for Sediment Transport Calculation

To solve the new sediment transport equation, a standard split approach by Sobey (1983), is used. Such an approach requires solving the advection and diffusion part, separately and at each time step. In general terms of the split-operator, the advection-dispersion equation is decomposed into the hyperbolic (pure advection) and the parabolic (pure dispersion with sink and source terms) partial differential equations. The two sub-equations are then solved separately in two or three consecutive fractional steps by the corresponding numerical approaches that best fit the features of each PDE for one time step. Based on the split-operator algorithm it is commonly
assumed that the pure advection process and the pure dispersion process alternate with time: the
advection process occurs in the first sub-time step, the dispersion takes place in the second sub-
time step, and the reaction is considered in the final sub-time step (Holly and Preissmann, 1977).
In the current case, there is no reaction term. The real disadvantage is using such a method is that
it introduces a splitting error. This is because, the approach proceeds with solving advection and
dispersion one after another, while in real scenario, they occur simultaneously. The fraction step
method procedure can be explained below. An ADE transport equation can be written as
\[
\frac{\partial c}{\partial t} + L_{ADV}(c) - L_{DISP}(c) = 0
\]  
(2.3.1)

Where \( L_{ADV}(c) \) is the advection part and \( L_{DISP}(c) \) is the dispersion part including all source sink
terms.

\[
\frac{C^{N+1} - C^N}{\Delta t} + L_{ADV}(C^N) - L_{DISP}(C^N) = \frac{\partial^2 C^N}{\partial t^2} \frac{\Delta t}{2} + \ldots \ldots = O(\Delta t)
\]  
(2.3.2)

Introducing the fraction step approach and an intermediate variable ‘\( c \)’, advection and diffusion
parts can be written separately as:

\[
\frac{C - C^N}{\Delta t} + L_{ADV}(C^N) = \frac{\partial^2 C^N}{\partial t^2} \frac{\Delta t}{2} + \ldots \ldots = O(\Delta t)
\]

\[
\frac{C^{N+1} - C^N}{\Delta t} - L_{DISP}(C^N) = 0
\]  
(2.3.3)

The fraction step procedure is independent of the schemes used for solving advection and
diffusion parts. Discretization of velocity and sediment concentration is as shown below in Fig 4.

![Fig 4: Discretization of Velocity and Concentration](image)
2.3.1 Advection Part

Operator splitting algorithms are commonly used in advection dominated transport problems. Leveque (1996) developed a high resolution conservative algorithm for advection in incompressible flow. In this algorithm, a basic upwind method and many correction terms were used in order to achieve the required accuracy and stability.

Concentration is scalar and the conservative form of advection of such a scalar function is the density function $C(x, t)$ and can be written in a general form as shown below

$$C_t + \nabla \cdot (u \ C) = 0 \quad (2.3.1.1)$$

Assuming the flow is incompressible

$$\nabla \cdot \vec{u}(x,t) = 0 \quad (2.3.1.2)$$

From such a generalized equation, one dimensional advection equation can be written as

$$C_t + (Cu)_x = 0 \quad (2.3.1.3)$$

and assuming flow is incompressible

$$u_x (x,t) = 0 \ , \ \text{for all} \ x, \ t \quad (2.3.1.4)$$

For incompressibility in discrete form for every cell in the discretized domain the following condition should satisfy:

$$(u_i^{N+1} - u_i^N) = 0 \quad (2.3.1.5)$$

Such a conservative equation could be solved using Leveque (1996) algorithm which uses a basic upwind method in flux differencing and later added correction terms to achieve better accuracy and stability. The upwind method is actually based on the flux calculation of the concentration at the cell interfaces and can be written as

$$C_i^{N+1} = C_i^{N+1} - \frac{k}{h}[F_{i+1} - F_i] \quad (2.3.1.6)$$
where \( F_i \) represents the flux at the interface of the cell \( C_i \) and \( F_{i+1} \) represents the flux at the right interface of the cell. Fig 5 shows the discretization of concentration, flux and velocity for advection.

\[
\begin{array}{c|c|c|c|c}
C_{i-1} & u_i & C_i & u_{i+1} & C_{i+1} \\
F_i & & F_{i+1} & & \\
\end{array}
\]

Figure 5: Discretization of concentration, Flux, velocity for Advection

These fluxes at the cell interfaces can be calculated as:

\[
F_i = u_i^{N+1}C_{i-1}^N
\] (2.3.1.7)

In this whole section \( u \) is taken in the X-direction positively and all derivations are done by assuming \( u \) is positive. In reality, the directions of these fluxes depend on the directions of the velocity vector. Thus the equations can be rewritten as

\[
C_i^{N+1} = C_i^N + \frac{k}{h} [u_{i+1}^{N+1}C_i^N - u_i^{N+1}C_{i-1}^N]
\] (2.3.1.8)

In this upwind method it is assumed that waves carrying differences \( C_i - C_{i-1} \) and propagate perpendicular to the interface in the X directions at speed and direction given by velocity \( u \). This function can be achieved by using the wave propagation method and assuming the above specified condition. In case of wave speed \( u \) in the grid oblique to the interfaces a proper correction factor is implemented. This correction can be incorporated by a two step procedure. In the first step the same upwind method is used in which wave is propagated perpendicular to the interface and in the next step the remaining triangular part of the wave is used to update the flux between the cells due to its transverse motion. The area of the triangular part of the wave is 

\[
\frac{1}{2} k^2 u
\]

and due to this the cell average is modified by the value of 

\[
\frac{1}{2}\frac{k^2}{h} u \Delta c
\]

In this quantity \( \Delta c \) is
the difference across the wave. This modification can be incorporated in the flux calculation of $F_i$. For wave propagating,

$$F_i = F_i + u_i^{N+1}C_{i-1}^N$$  \hspace{1cm} (2.3.1.9)

To achieve second order accuracy in the algorithm, a second order Lax-Wendroff method is combined with the upgraded upwind method. The Lax-Wendroff method to calculate flux can be expressed as:

$$F_{i-1}^{LW} = \frac{1}{2}u_i(C_{i-1} + C_i) - \frac{k}{2h}u^2(C_i - C_{i-1})$$  \hspace{1cm} (2.3.1.10)

The Lax-Wendroff scheme can be rearranged as a combination of upwind method and correction term as;

$$F_{i-1}^{LW} = u_iC_{i-1} + \frac{1}{2}|u|(1 - \frac{k}{h}|u|)(C_i - C_{i-1})$$  \hspace{1cm} (2.3.1.11)

$$F_{i-1}^{LW} = F_{i-1}^{UP} + \frac{1}{2}|u|(1 - \frac{k}{h}|u|)(C_i - C_{i-1})$$  \hspace{1cm} (2.3.1.12)

This approach is used to supply another correction term in the updated upwind method by adding the following term in the flux at the interface between $C_i$ and $C_{i-1}$. To avoid oscillation a flux limiting factor is also used.

$$F_{i-1} = F_{i-1}^{UP} + \frac{1}{2}|u|(1 - \frac{k}{h}|u|)(C_i - C_{i-1}) \cdot \phi_i$$  \hspace{1cm} (2.3.1.13)

The flux limiting term $\phi_i$ is defined as

$$\phi = f(\theta), \quad \theta_i = \frac{q_i - q_{i-1}}{q_i - q_{i+1}}$$  \hspace{1cm} (2.3.1.14)

Where $I = \{ \begin{array}{ll} i-1, \text{ if } u > 0 \\ i+1, \text{ if } u \leq 0 \end{array} \}$  \hspace{1cm} (2.3.1.15)
Some standard limiters used in the algorithm are as follows:

minmod: \( f(\theta) = \max(0, \min(1, \theta)) \),

superbee: \( f(\theta) = \max(0, \min(1, 2\theta), \min(2, \theta)) \),

Van Leer: \( f(\theta) = \frac{\theta + |\theta|}{1 + |\theta|} \)

monotonized centered: \( f(\theta) = \max(0, \min(1 + \theta)/2, 2, 20) \)

Leveque (1996) included the transverse propagation concept for the correction waves to increase the accuracy of the second order accurate method developed till now. This transverse motion of the correction wave at the interface between cells \( C_i \) and \( C_{i-1} \) modifies the flux \( F_i \) as described in the previous part.

These modifications in the flux calculations reduce the error. Leveque (1996) developed an algorithm by following all these steps one by one and that algorithm is shown in the appendix. The algorithm takes care of the directions of the velocity vectors.

### 2.3.2 Dispersion Part

To solve for the dispersion part of the advection dispersion sediment transport equation, a Semi-implicit finite difference scheme is used. The semi-implicit finite difference scheme is implemented in such a way that it can easily be converted to a completely explicit or completely implicit scheme. A finite difference representation of the diffusion part can be written as follows:

\[
A \frac{\partial S}{\partial t} = \frac{\partial}{\partial x} (AK_x \frac{\partial S}{\partial x}) + SS \quad \text{(2.3.2.1)}
\]

Let \( AK_x \frac{\partial S}{\partial x} = T_x \), \( C = AS \) and \( SS \) be source sink

Therefore,

\[
\frac{C^{N+1}_i - C^N_i}{\Delta t} = \frac{T_x(i+1) - T_x(i)}{\Delta x}
\]
Here, \( T_x(i) \) can be calculated as

\[
T_x(i) = K_x(i)A(i)[\theta(S_i^{N+1} - S_{i-1}^{N+1}) + (1 - \theta)(S_i^N - S_{i-1}^N)]
\]

\[
T_x(i + 1) = K_x(i + 1)A(i + 1)[\theta(S_{i+1}^{N+1} - S_{i+1}^{N+1}) + (1 - \theta)(S_{i+1}^N - S_{i+1}^N)]
\]

\[
\frac{AS_i^{N+1} - AS_i^N}{\Delta t} = \frac{T_x(i + 1) - T_x(i)}{\Delta x}
\]

\[
A(S_i^{N+1} - S_i^N) =
\]

\[
K_x(i + 1)A(i + 1)[\theta(S_{i+1}^{N+1} - S_{i+1}^{N+1}) + (1 - \theta)(S_{i+1}^N - S_{i+1}^N)] - K_x(i)A(i)[\theta(S_i^{N+1} - S_{i-1}^{N+1}) + (1 - \theta)(S_i^N - S_{i-1}^N)]
\]

\[
+ SS
\]

\[
(2.3.2.2)
\]

\[
S_i^{N+1} - S_i^N = \frac{\Delta t}{A\Delta x} [K_x(i + 1)A(i + 1)\theta S_i^{N+1}_{i+1}] - \frac{K_x(i + 1)A(i + 1)\theta S_i^{N+1}_{i-1}}{\Delta x} - \frac{K_x(i)A(i)\theta S_i^{N+1}}{\Delta x} +
\]

\[
K_x(i)A(i)\theta S_i^{N+1}_{i-1}] + \frac{\Delta t}{A\Delta x} \left[ K_x(i + 1)A(i + 1)(1 - \theta)S_i^N \right] - \frac{K_x(i + 1)A(i + 1)(1 - \theta)S_i^N}{\Delta x} -
\]

\[
\frac{K_x(i)A(i)(1 - \theta)S_i^N}{\Delta x} + \frac{K_x(i)A(i)(1 - \theta)S_i^N}{\Delta x} + (\Delta t)SS
\]

\[
(2.3.2.3)
\]

\[
S_i^{N+1} = \frac{\Delta t}{A(\Delta x)^2} [K_x(i + 1)A(i + 1)\theta S_i^{N+1}_{i+1}] + \frac{\Delta t}{A(\Delta x)^2} [K_x(i + 1)A(i + 1)\theta S_i^{N+1}_{i-1}] +
\]

\[
\frac{\Delta t}{A(\Delta x)^2} [K_x(i)A(i)\theta S_i^{N+1}_{i-1}] - \frac{\Delta t}{A(\Delta x)^2} [K_x(i)A(i)\theta S_i^{N+1}_{i-1}] =
\]

\[
S_i^N - \frac{\Delta t}{A(\Delta x)^2} [K_x(i + 1)A(i + 1)(1 - \theta)S_i^N] + \frac{\Delta t}{A(\Delta x)^2} [K_x(i + 1)A(i + 1)(1 - \theta)S_i^N] +
\]

\[
- \frac{\Delta t}{A(\Delta x)^2} [K_x(i)A(i)(1 - \theta)S_i^N] + \frac{\Delta t}{A(\Delta x)^2} [K_x(i)A(i)(1 - \theta)S_i^N] + SS(\Delta t)
\]

\[
(2.3.2.4)
\]

The Equation (2.3.2.4) can be further written as given in Equation (2.3.2.5)
\[ S_{i}^{N+1} \{ 1 + \frac{\Delta t}{A(\Delta x)^2} K_x(i+1)A(i+1)\theta + \frac{\Delta t}{A(\Delta x)^2} K_x(i)A(i)\theta \} \]

\[ + \quad S_{i+1}^{N+1} \left[ \frac{\Delta t}{A(\Delta x)^2} K_x(i+1)A(i+1)\theta \right] - \quad S_{i-1}^{N+1} \left[ \frac{\Delta t}{A(\Delta x)^2} K_x(i)A(i)\theta \right] \]

\[ = \quad S_{i}^{N} \{ 1 - \frac{\Delta t}{A(\Delta x)^2} K_x(i+1)A(i+1)(1-\theta) - \frac{\Delta t}{A(\Delta x)^2} K_x(i)A(i)(1-\theta) \} \]

\[ + \quad S_{i+1}^{N} \left[ \frac{\Delta t}{A(\Delta x)^2} K_x(i)A(i)(1-\theta) \right] + \quad S_{i-1}^{N} \left[ \frac{\Delta t}{A(\Delta x)^2} K_x(i+1)A(i+1)(1-\theta) \right] + SS(\Delta t) \]

\[ \Rightarrow S_{i}^{N+1} - S_{i+1}^{N+1} \left( \frac{\Delta t}{A(\Delta x)^2} [K_x(i+1)A(i+1)\theta] \right) \]

\[ = \quad S_{i}^{N} \left[ 1 - \frac{\Delta t}{A(\Delta x)^2} K_x(i+1)A(i+1)(1-\theta) - \frac{\Delta t}{A(\Delta x)^2} K_x(i)A(i)(1-\theta) \right] \]

\[ + \quad S_{i+1}^{N} \left[ \frac{\Delta t}{A(\Delta x)^2} K_x(i+1)A(i+1)(1-\theta) \right] \]

\[ + \quad S_{i-1}^{N} \left[ \frac{\Delta t}{A(\Delta x)^2} K_x(i+1)(1-\theta) \right] \]

\[ + \quad SS(\Delta t) \left( \frac{1}{1 + \frac{\Delta t}{A(\Delta x)^2} K_x(i+1)A(i+1)\theta + \frac{\Delta t}{A(\Delta x)^2} K_x(i)A(i)\theta} \right) \]
Thus the above equation can be written as

\[ S_{i}^{N+1} - Cu1(i) S_{i+1}^{N+1} - Cu2(i) S_{i-1}^{N+1} = b(i, N) \]  

(2.3.2.6)

where \( Cu1(i) = 1 \)

\[ Cu2(i) = \left( \frac{\Delta t}{A(\Delta x)^2} K_{x} (i+1) A(i+1) \theta \right) \]

\[ Cu3(i) = \left( \frac{\Delta t}{A(\Delta x)^2} K_{x} (i) A(i) \theta \right) \]

and

\[ b(i, N) = \text{right hand side known components} \]

\[ b(i, N) = \frac{\Delta t}{A(\Delta x)^2} K_{x} (i+1) A(i+1) (1-\theta) - \frac{\Delta t}{A(\Delta x)^2} K_{x} (i) A(i) (1-\theta) \]

\[ + \frac{\Delta t}{A(\Delta x)^2} K_{x} (i) A(i) (1-\theta) \]

\[ + \frac{\Delta t}{A(\Delta x)^2} K_{x} (i+1) A(i+1) \theta + \frac{\Delta t}{A(\Delta x)^2} K_{x} (i) A(i) \theta \]

\[ + \frac{\Delta t}{A(\Delta x)^2} K_{x} (i+1) A(i+1) (1-\theta) \]

\[ + \frac{\Delta t}{A(\Delta x)^2} K_{x} (i) A(i) (1-\theta) \]

(2.3.2.7)

The step size for distance was taken to be 375 m and the time step size was taken to be 10 seconds.

The grid size was chosen carefully, so as to meet the stability criterion and also avoid being computationally expensive. The above equation can be written as a linear system of equations and can be represented as \( Ax = B \). To solve this dispersion part, the above linear system needs to be solved. Precisely, for this reason, various numerical solver schemes can be employed.
2.3.3 Numerical Scheme for Solving Linear System of Equations for Dispersion part

The need for various numerical schemes for solving linear system of equations in scientific computing problems is a very common situation. There are many algorithms available in this regards. Iterative methods are often used for solving discretized partial differential equations. One of the many such algorithms is the Jacobi method.

The Jacobi method is a method of solving a tridiagonal matrix equation with largest absolute values in each row and column dominated by the diagonal element. Each diagonal element is solved for, and an approximate value plugged in. The process is then iterated until it converges. This algorithm is a stripped-down version of the Jacobi transformation method of matrix diagonalization. The Jacobi method is easily derived by examining each of the equations in the linear system of equations in isolation.

\[ \sum_{j=1}^{n} a_{ij} S_j = b_i \]  
\hspace{1cm} (2.3.3.1)

Solve for the value of while assuming the other entries of remain fixed. This gives

\[ S_i^{n+1} = \frac{b_i - \sum_{j=1}^{n} a_{ij} S_j^n}{a_{ii}} \]  
\hspace{1cm} (2.3.3.2)

which is the Jacobi method.

\( S_i^{n+1} \) is the \( n^{th} \) iteration for \( S_i \). The iteration error for the entire can be calculated as below

\[ \text{Error} = \sqrt{\sum \text{abs}(S_i^{n+1} - S_i^n)^2} \]

The iteration terminates at when the desired tolerance equals the error.

The order in which the equations are deal with is irrelevant, since the Jacobi method examines them independently. For this reason, the Jacobi method is also known as the method of simultaneous displacements, since the updates could in principle is done simultaneously.
Chapter 3: Soil Erosion Calculation for Amite River Basin

3.1 Soil Erosion Issues in Amite River Basin

Louisiana, where over 25% of the nation's wetlands and 40% of the nation's salt marshes (exclusive of Alaska) are located, is losing its coastal land at a catastrophic rate. Erosion, if unchecked, could speed up in some other parts too. The Amite River basin is one such portion where soil erosion has lead to major water bodies impairment.

The Amite River has very high turbidity than a comparable stream in the vicinity (LDEQ, 1998). The turbidity levels in the Amite River could be attributed to basin floods, which cause lot of soil erosion. The general erosion rates for unvegetated areas are 5 to 10 tons per acre per year, whereas erosion rates for fully vegetated systems has been found to be virtually nil (USGS, 1998). Urban development in parts of the basin, results in increased surface water runoff, resulting in increase of highly turbid water. These highly turbid waters carrying sand destroys vegetation, thereby making the soil prone to erosion. Flats along the river and activities such as Sand and gravel mining, timber cutting, farming, and residential construction result in high erosion rate, likely over several tons of material per annum (USACE, 1997). Fig 6 shows the Amite River’s contributing watershed.

From Table 1, one can infer that land use in the Amite River Basin is largely forestry and agriculture. Forest and agricultural land uses have declined since 1954 from about 83 percent of the total land use to 72 percent in 1985. About 85 percent of the forest and agricultural lands are located in the Mississippi portion of the basin, East Feliciana, Livingston, and East Baton Rouge Parishes (USACE, 1997).
A spatial estimation of potential soil loss for the watershed will, therefore, help to identify possible non point sources to the Amite River. The Universal Soil Loss Equation (USLE) is an
empirical model used to estimate erosion, the first step of sediment transport. The objective of is to construct a GIS based potential soil loss spatial index model based on the USLE equations. Once soil detachment occurs sediment is accumulated and deposited during transport within the watershed. The USLE can only estimate potential soil loss, not transport, so the index is a relative measure that will indicate where soil loss “hot spots” may exist. The index will provide useful information to aid decision makers in considering natural resource management strategies for the watershed. Sediment transported to the Amite River will be considered in a separate section.

3.2 Predicting Soil Loss Using the Universal Soil Loss Equation (USLE)

Soil erosion is one of the major contributors of non point source pollution. Several commonly used methods have been used provide estimation of erosion from multiple sources, hill storage and sediment delivery to streams. Methods that have been used but not limited to are USLE/RUSLE, AGNPS, BASINS-NPSM, WEPP, HSPF, and SWAT (Sediment TMDL TAG, 2002). Most of these methods apply models that estimate erosion as a function of several key factors potentially including soil characteristics, topography, vegetation characteristics and precipitation.

The USLE, developed by ARS scientists W. Wischmeier and D. Smith (1958), has been the most widely accepted and utilized soil loss equation for estimation of sediment loss and for predicting non point sediment losses in pollution control programs. The USLE for estimating average annual soil erosion is:

\[ A = RKLSCP \]  \hspace{1cm} (3.2.1)
• A = average annual soil loss in t/a (tons per acre)
• R = rainfall erosivity index
• K = soil erodibility factor
• LS = topographic factor - L is for slope length & S is for slope
• C = cropping factor
• P = conservation practice factor

Values for each of the six factors are determined from research data. It is important to note, that values generally differ from one location to another. Predicted soil loss is nothing but the calculated annual soil loss expressed in tons per acre. In most of the nonpoint source models, USLE or one of its other versions is used for predicting soil erosion. Although originally developed for agricultural purpose, its use has been extended to watershed with other land uses. It is therefore suitable, to apply this model for predicting soil erosion in Amite River Basin. ArcGIS, the Spatial Analyst Extension was used to process input data, construct the model, and present results. The model within an ArcGIS project, with a single view for each equation factor’s input and pre-processed data is adopted. The Raster Calculator in the Spatial Analyst Extension is the tool that allows users to build a model and processes then run the model. Several such model makers exist in other GIS software such as Arcview and also in Remote sensing software such as spatial model maker in ERDAS-IMAGINE. For this work, the ArcGIS spatial analyst was used to derive the erosion map from all the input maps. In order to achieve this USLE equation was modeled spatially by designating a raster grid for each equation factor as input parameters of the model. A mathematical overlay process multiplies the corresponding cells of each grid. Five major factors are used to calculate the soil loss for a given site. Each equation factor is used as a model input parameter. The P, K and R factors were obtained from
the literature, so the only processes required were conversions to grids. C, L and S factors were derived within the model construction. Input data, their sources, and the equation factor to which they apply, are listed in Table 2.

Table 2: GIS data used for determination of Soil Erosion in Amite River Basin

<table>
<thead>
<tr>
<th>Input Data</th>
<th>Equation Factor</th>
<th>Data Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>1985 Land Use</td>
<td>C</td>
<td>Table of C factors in Agriculture Handbook by USDA</td>
</tr>
<tr>
<td>LIDAR DEM</td>
<td>LS</td>
<td><a href="http://www.atlas.lsu.edu">www.atlas.lsu.edu</a></td>
</tr>
<tr>
<td>Rainfall Erositivity Index</td>
<td>R</td>
<td>Rainfall Index Map</td>
</tr>
<tr>
<td>Soil Erositivity Index</td>
<td>K</td>
<td>STATSGO</td>
</tr>
<tr>
<td>Assumed Value</td>
<td>P</td>
<td>NA</td>
</tr>
</tbody>
</table>

The P factor mainly takes into account land management practices that help to reduce erosion, such as contouring on farms. The P factor is nothing but the ratio of soil loss by an adopted practice to that of straight-row farming up and down the slope. The most commonly used supporting farming practices for crops are cross slope cultivation, contour farming and strip-cropping. Different support practice adopted would lead to different P factor values. A up and down slope practice would have the highest P value whereas strip cropping practice would have the lowest P value. The selection of a support practice that has the lowest possible factor associated with it will result in lower soil losses. For exp, the adoption of contour farming would
cause deposition of soil near the source. Because detailed management practices were unknown in the Amite River basin, the P factor is assumed to be 1.

Fig 7: Crop Management factor distribution in Amite River Basin (P=1)

Fig 7 shows the P factor distribution in the basin. R factor was determined from the rainfall index map and set at 350. The significance of R factor is that a very high value would suggest that the intensity and rainfall is also very high as a result of which higher would be the soil erosion potential. Fig 8 shows the rainfall index map distribution for the United States and thus one can
infer that Louisiana is among the 6 states with the highest R in the nation. R factor grids were converted from the watershed boundary polygon shape file, containing constant cell values of 350.

Fig 8: Rainfall Index Map for United States (Source: Elementary Soil & Engineering)

K is the soil – erodibility factor. It is defined as the rate of erosion per unit of erosion index from unit plots on a given soil. The K factor is a measure of soil erodibility in terms of susceptibility to detachment as well as transport, and ranges from 0.05 for low erodibility to 0.4 for high erodibility. While clays have low K values because they are not as easily detached, sandy soils also have low K values because they are difficult to transport via runoff. K factors for this grid were obtained from the STATSGO database and polygons were converted to the K factor raster.
grid. The Fig 9 shows the K factor distribution throughout the watershed. Higher values of K factor are in the middle portion of the basin. Portion of the basins adjacent to Mississippi and in the upper regions, have relatively less K factor values than the middle portion and vary between 0.2-0.4. The K factor values in the lower portion of the basin vary from 0-0.32.

Fig 9: K factor distribution in Amite River Basin

C is the crop management factor in the USLE. It is the ratio of soil loss from a field with specified cropping and management or plant cover, to that from a fallow condition on a unit plot. Estimated C values were assigned to Amite River Basin land use data based on values from the Table of C factors in Agriculture Handbook by USDA. The land use polygon shape file was then
converted to a raster grid, with a C factor assigned to each cell. Table 3 shows example C factors extracted from the literature.

Table 3: List of C factors according to Land Use (Source: Agriculture handbook, USDA)

<table>
<thead>
<tr>
<th>Land Use</th>
<th>C factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Row Crops</td>
<td>0.24</td>
</tr>
<tr>
<td>Pasture/hay</td>
<td>0.05</td>
</tr>
<tr>
<td>Water/wet areas</td>
<td>0.00</td>
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<tr>
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<td>0.03</td>
</tr>
<tr>
<td>Urban, high density</td>
<td>0.00</td>
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<tr>
<td>Deciduous Forest</td>
<td>0.009</td>
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<tr>
<td>Evergreen/Coniferous Forest</td>
<td>0.004</td>
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<tr>
<td>Mixed Forest</td>
<td>0.007</td>
</tr>
<tr>
<td>Forest/Woody Wetland</td>
<td>0.003</td>
</tr>
</tbody>
</table>
Fig. 10 shows the C factor distribution throughout the watershed. The distribution shows that the C factor values generally vary between 0.004-0.05 for most of the upper Amite river basin portion. However the C factor values are equal to 1 in some sand gravel and mining portions of the upper Amite river basin.

Fig 10: C factor distribution in Amite River Basin

The LS factor was calculated from the values obtained by analyzing the composite DEM for the entire basin. Fig 11 shows the DEM for the Amite River watershed.
LS is the slope length-gradient factor and also known as the topographic factor. The LS factor represents a ratio of soil loss under given set of conditions to that at a site with the "standard" adopted slope steepness of 9% and slope length of 72.6 feet. Therefore the first step in LS factor determination is to find slope. The steeper and longer the slope, the higher is the risk for erosion. Reduction of slope length factor can be possible with the construction of terraces. Fig 12 shows the slope derived for the Amite River Basin DEM. The slope value varies from 0 – 49 degrees for the basin. The upper portion of the basin, adjacent to Mississippi are characterized by high slope values.
It is quite possible that in a given DEM there would be locations which would not give the existence of a natural drainage path. Such locations are termed as “Sinks”. In general sinks are the cells with negative slope and therefore there is only inflow and no outflow. However some sinks can be lakes and natural depressions and therefore some of them are valid. However, formation of a sink at a location where a stream is supposed to flow is problematic. Sinks frequently occur on DEMs due to data errors and noise and are mostly spurious representation of topography. These errors are often caused by sampling effects and the rounding of elevations to integer numbers. It is important to preprocess the DEM before calculation flow direction and accumulation correctly. Using the Raster Calculator the sinks are found out. The Fig 13 shows the sinks in the Amite River Basin DEM.
Fig 13: Presence of Sinks in Amite River Basin DEM

These sinks have to be filled before proceeding ahead. Thus, using the Grid function option in ArcGIS the sinks are filled. The next step is to create a grid of flow direction from each cell to its steepest down slope neighbor. Flow direction is direction in which water would flow from each cell on the DEM. Generally, downstream tracing is achieved by deriving the flow direction grid from the perfected DEM. By knowing flow direction, delineation watersheds can be easily done in a more precise way. Figure 14 shows the flow direction grid, for the Amite River Basin. Flow direction grid is an aspect grid. If cell is assigned a "no data" value, then it means that it does not contribute flow to any of the defined outlets in the basin. The flow direction of a cell is expressed in degrees.
Once flow direction is determined, the next step is to find the flow accumulation grid for the basin. The flow accumulation gives an area that is derived from all cells that flow into each downslope cell. It is an indirect way of determining the drainage areas in terms of grid cells. In a raster-based analysis, the total count of cells, including non-neighboring cells that drain into a particularly identified cell. Flow accumulation data is derived from the flow direction data in GIS after correction to the DEM and can be used for delineation of stream networks within a basin. This was done by applying the flow accumulation function on the flow direction grid in the raster calculator. Figure 15 shows the flow accumulation grid for the Amite River Basin.
L and S were treated as a combined factor to find the LS index. The LS factor can be estimated from the DEM. There are several ways to compute the LS factor using a DEM. One of the most popular approaches uses the upslope drainage contributing area, which is calculated using the formula developed by (Mitasova et. al. 1996). It is possible to determine the empirically based LS factor using the Equation 3.2.2 appropriately in the raster calculator tool in the spatial extension of ArcGIS. Slope length and stiffness factors are typically calculated together for input into the equation below. The inputs for for the equation have been previously derived from the DEM.

\[
LS = \left(\frac{\text{flowaccumulation} \times \text{cellsize}}{22.13}\right)^{0.6} \left(Sin(slope \times 0.01745)\right)^{1.3} \times 1.6
\]  

(3.2.2)

Figure 16 shows the LS factor distribution in the Amite River Basin.
The soil erosion map was derived by a mathematical overlay process which multiplied the corresponding cells of each input grid. The average soil erosion was found to be **5.42 tons/acre/year** even though; the soil erosion value was very high at some points in the basin. This value is quite in consonance with the soil erosion values between 5-10 tons/acre/year in the Amite River basin, which is cited in literature (USACE, 1997). Fig 17, shows the computed soil erosion distribution and available land use distribution map for the Amite River Basin.
Fig 17: Soil Erosion and Land Use distribution for the Amite River Basin
3.2.1 Single Event Sediment Yields

The USLE model is applied for determination of soil erosion for average data and is not valid for individual storms. In order to estimate soil erosion in Amite River Basin for individual storm, it is important to take into account the volume of runoff and not rainfall erositivity. A method called the Modified Universal Soil Loss Equation (MUSLE) (Williams, 1975) illustrates how sediment yields for individual storm events might be obtained:

\[ Y = 95(Qq)^{0.56} KLSCP \]  \hspace{1cm} (3.2.1.1)

Where \( Y \) is the single storm sediment yield in tons, \( Q \) is the storm runoff volume in acre-ft, \( q \) is the peak discharge in cfs, and the other terms are the standard USLE factors. The Modified Universal Soil Loss equation follows the USLE model, with the replacement of rainfall erositivity factor with runoff factor. The approach has seen widespread application, but is used with caution as it was developed empirically based on limited data for Texas and the southwestern United States. Since Louisiana is located in the south with decent proximity with the region where the formula was developed, this procedure can be used with considerable judgment. It is however important to select an appropriate slope length when determining the \( LS \) topographic factor.

3.2.2 Sediment Delivery Ratio:

Sediment yield is a critical factor in identifying non-point source pollution as well as in the design of the construction such as dams and reservoirs. However, sediment yield is usually not available as a direct measurement but estimated by using a sediment delivery ratio (SDR).

An accurate prediction of SDR is important in controlling sediments for sustainable natural resources development and environmental protection. Soil erosion is the first step in the
sedimentation processes which consist of erosion, transportation and deposition of sediment. A fraction of eroded soil passes through channel system and contributes to sediment yield while some of them deposit in water channels. Sediment yields can be quantified using the SDR, expressed as the percent of gross soil erosion by water that is delivered to a particular point in the drainage system. It is computed as the ratio of sediment yield at the watershed outlet (point of interest) to gross erosion in the entire watershed. Gross erosion includes sheet, rill, gully and channel erosions (Da Ouyang, 1997).

There is no precise procedure to estimate SDR, although the USDA has published a handbook in which the SDR is related to drainage area (USDA SCS, 1972). The relationship established for sediment delivery ratio and drainage area is known as the SDR curve. A small watershed with a higher channel density has a higher sediment delivery ratio compared to a large watershed with a low channel density. A watershed with steep slopes has a higher sediment delivery ratio than a watershed with flat and wide valleys. In order to estimate sediment delivery ratios, the size of the area of interest should also be defined (Da Ouyang, 1997). In general, the larger the area size, the lower the sediment delivery ratio as shown in the Fig 18.

Fig 18: Plot of Delivery ratio vs. Drainage area (Source: Mechanics of Sediment transport)
Sediment delivery ratio can be expressed as:

$$\text{SDR} = \frac{\text{SY}}{\text{E}}$$  \hspace{2cm} (3.2.2.1)$$

Where SDR = the sediment delivery ratio and SY = the sediment yield

E = the gross erosion per unit area

For determination of SDR in the Amite River Basin the model adopted was the USDA SCS (1979) developed a SDR model based on the data from the Blackland Prairie, Texas. A power function is derived from the graphed data points:

$$\text{SDR} = 0.51 \ A^{-0.11}$$  \hspace{2cm} (3.2.2.2)$$

Where A = drainage area in square miles.
Chapter 4: HEC-RAS Based Flow Calculation

4.1 Introduction to HEC-RAS

In order to determine as how the sediment is transported in the river domain certain flow parameters such as velocity, depth, slope etc. would be required as inputs to the 1-D sediment transport model, given by equation 3.17. The U.S. Army Corps of Engineers’ River Analysis System (HEC-RAS) is software that allows performing one dimensional steady and unsteady flow river hydraulics calculations. HEC-RAS is an integrated system of software, designed for interactive use in a multi-tasking, multi-user network environment. The system is comprised of a graphical user interface (GUI), separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities.

The HEC-RAS system contains three one-dimensional hydraulic analysis components for: (1) steady flow water surface profile computations; (2) unsteady flow simulation; and (3) movable boundary sediment transport computations. A key element is that all three components use a common geometric data representation and common geometric and hydraulic computation routines. In addition to the three hydraulic analysis components, the system contains several hydraulic design features that can be invoked once the basic water surface profiles are computed.

4.2 Input Data

HEC-2 input files for the Amite River above Denham Springs labeled “UAR.DAT” was obtained from US Army Corps of New Orleans district. These upper Amite river existing conditions, cross-sections are obtained from LADOTD. The updated model was done in May 1990 and was for one year storm. Making HEC-2 runs of these input files gave most of the hydraulic and geometric parameters. Comment cards in the input file was used to determine the locations of the
cross sections that are located at bridges, confluences etc., and the reach lengths can be used to determine the locations of other cross sections relative to the cross sections that have comment cards associated with them.

4.3 Importing HEC-2 Data into HEC – RAS

An important feature of HEC-RAS is the ability to import HEC-2 data. This feature makes it easy for a user to import existing HEC-2 data sets and start using HEC-RAS immediately. Once the HEC-2 file “UAR.DAT” for the Amite River Basin was imported, then identification of the cross-section becomes an important issue. In HEC-RAS, each cross section is identified with a River name, Reach name, and a River Station. The river stationing in the data file was in order from highest river stationing upstream i.e. River mile 97 to lowest river stationing downstream i.e. River mile 50.1. There are two river identification methods namely:

1. Using HEC-2 id
2. Using sequential counter

Use of HEC-2 Section ID's," enabled the program to use the first field of the X1 record for the river stationing of the cross section. While selecting this method, the cross sections in the HEC-2 file were numbered with highest river stationing upstream, and that no two cross sections had the same river station identifier. The program would not have been able to import the data correctly, had these two requirements not been met.

After the HEC-2 data is imported into HEC-RAS, there were some modifications made to the data. There is a good compatibility of HEC-2 data with the HEC-RAS software as a result of which, all the data input like geometric and flow data could be read from the HEC-2 file into HEC-RAS. The following is a list of features in which the data requirements for HEC-2 and HEC-RAS have changed, and it was necessary to modify the data after it is imported:
- Special Bridge (SB)
- Special Culvert (SC)
- Normal Bridge (X2, BT)
- Encroachments and Floodway Determination (X3, ET)
- Ineffective Flow Areas (X3)

The Schematic representation of the Amite River is as shown below. The river is divided into numerous cross sections. Each cross-section represents a location and the corresponding river mile from the downstream end is labeled. The river mile 97 is the starting point and river mile station 50.1 is the most downstream location for the river reach. Fig 31 – Fig 65 in the appendix shows the cross-section along the Amite River.

The direction of the flow is from the higher river mile station towards the lower river mile stations. Other required flow parameters like Manning’s coefficient, cross-section profile are appropriately defined at each cross section.

Fig 19: Schematic Representation of Amite River with various River Mile Stations.
4.4 Steady Flow Analysis

This component of the modeling system is intended for calculating water surface profiles for steady gradually varied flow. The basic computational procedure is based on the solution of the one dimensional energy equation. The effects of various obstructions such as bridges, culverts, weirs, spillways and other structures in the flood plain has been considered in the computations. Once all of the geometric data were entered, it was possible then to enter any steady flow data that were required. The steady flow data editor was brought up, and “Steady Flow Data” was selected from the “Edit” menu on the HEC-RAS main window. The last step in developing the steady flow data was to save the information to a file. The outputs from HEC- RAS gave the necessary flow information in the Amite River. Fig 21 and Fig 22 shows the discharge and velocity profile. Fig 67 and Fig 68 shows the depth and flow variations respectively in the Amite River.

4.5 Unsteady Flow Analysis

Sediment concentration is very sensitive to flow variations. Therefore, to study the impact of unsteady flow regime on sediment transport in the Amite River basin arising due to individual storm events, it is important to get flow parameters under unsteady flow conditions. The objective of unsteady flow simulation is to get the flow hydrograph at each of the river mile stations of the Amite River. The HEC-RAS modeling system was capable of simulating one-dimensional unsteady flow. The hydraulic calculations for cross-sections, bridges, culverts, and other hydraulic structures that were used for the steady flow component were also incorporated into the unsteady flow module. For the current problem, a lateral inflow hydrograph was taken as the boundary condition. In addition to
the boundary conditions, the initial conditions of the system at the beginning of the unsteady
flow simulation were also established. Initial conditions consisted of flow and stage information
at each of the cross sections. Once all of the geometry and unsteady flow data was enter in the
HEC-RAS main window and “Unsteady Flow Analysis” was selected from the “Run” menu to
perform unsteady flow simulation.
Chapter 5: Total Maximum Daily Load (TMDL) Calculation

5.1 Application of 1-D Sediment Dispersion and Transport Model to Amite River

The longitudinal variations of Suspended Sediment Concentration for steady flow is shown in the Fig 20:

Fig 20: Longitudinal Variations of SSC along the Amite River

The steady flow discharge in the Amite River is as shown in the plot i.e Fig 21.
In the Fig 21, the most downstream point in the river reach is the river mile 50.1 Amite River at U.S.190 near the Denham Springs gage. The most upstream point in the upper Amite River portion, which is under consideration, is river mile 97.0 State Line. There are a couple of points where the concentration peaks more than 100 mg/L and also, there exists points where the SSC concentration fall below 10 mg/L. From a first hand look at the plot, it can be safely concluded that there exists an uneven distribution of SSC concentration along the reach of the river.

The reason for such high concentration in some segment of the Amite River can be attributed to the presence of tributaries, which have high sediment concentrations due to soil erosion in the
watershed. Also the upstream portion is marked by high channel velocity, which results in bank and channel boundary erosion. Due to sediment deposition and dispersion the SSC concentration graphs have low SSC values in the middle segment of the river. The upstream regions is characterized by high velocities as a result of which, the transport of suspended sediments would be much faster downstream. The flow velocity plot is as shown below in Fig 22.

![Velocity variations along the Amite River](image)

**Fig 22: Velocity variations along the Amite River**

Added to it, the presence of tributaries in the downstream portion would lead to major accumulation of SSC values. The plot for various tributaries, inflow and outflow is as shown in Fig 23.
The high SSC can also be attributed to bank and channel erosion. A first hand look at the shear velocity plot reveals that the downstream portion of the Upper Amite River is characterized by high shear velocity. The shear velocity plot is as shown in Fig 24.

From the above points, one can derive the conclusion that over the time, in the downstream region, there would be major SSC concentration which might result in water body impairment. Furthermore, the values of the major flow parameters are not constant but vary from cross-section to cross-section. These longitudinal variations in the flow parameters, lead to different SSC values along the Amite River.

Fig 23: Inflow and Outflows in the Amite River
Fig 24: Shear Velocity variations along the Amite River

This pattern of longitudinal variation of SSC can be directly related and justified by the variation of other flow parameters. Table 5 lists the major flow parameter values.
### Table 4: Major Flow Parameter Values

<table>
<thead>
<tr>
<th>River Station(m)</th>
<th>lateral flow(cubic m/s)</th>
<th>Dispersion(sqm/s)</th>
<th>Discharge Q(total)</th>
<th>Velocity (m/s)</th>
<th>Shear Velocityl(m/s)</th>
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</thead>
<tbody>
<tr>
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5.2 Sensitivity Analysis

To analyze sensitivity of different terms in Equation (2.1.8), it is important to vary each parameter to see the resultant effect on the suspended sediment concentration. In this test the various source sink parameters and dispersion coefficient was performed. The dispersion coefficient values were increased at a steady rate and the effect on the suspended sediment concentration was noted by running the model for a given time period. In Fig 25 plots have been drawn to show the, the effect of various dispersion coefficient values on SSC.

Fig 25: Sensitivity analysis for dispersion

A sensitivity analysis of source/ sink term was done. Initially, the source/sink term was taken to be zero and thus ended up in results which did not reflect the heterogeneity of such a natural scenario. Appropriately, the source/sink term value was increased to find its increased effect on
the suspended sediment concentration. Initial results showed that at some locations there are drastic changes in the suspended sediment concentration. Thus, it could be concluded that everything remaining the same, the presence of source/sink term can cause the model results to vacillate between gross underestimation to close approximation. To study, the actual action of the source/sink terms on the model results, major parameters value have to be varied one by one to see, the impending effect of its change on the SSC. Sensitivity analysis was performed on the constant parameters, settling velocity and suspended sediment transport capacity terms. The prime reason of taking these parameter values is based on the assumption that any one or each of them has the potential to be the most dominant factor in the overall source / sink component of the sediment transport model.

The parameter $\alpha$ (alpha) in the source/sink value initially taken as 1. It was then increased by 50%, and the model was run to see the impact of such an increase on the SSC. The result is shown in the plot below in Fig 26.

![Sensitivity analysis for alpha](image-url)

**Fig 26: Sensitivity analysis for parameter alpha**
An increased value of alpha has an important final effect on the concentration. A small change of the value leads to an appreciable change in the final concentration results. Model sensitivity analysis was then performed to identify and compare quantitatively the effect of settling velocity on the SSC. The settling velocity was increased by 50% of its computed value and then over it, again another increase of 50% was done. Such an increase in the settling velocity can be justified because, rate of settling depends on the diameter of the sediment. In a real time scenario, the river domain would have sediments with different sizes with different settling velocity. The result of this increase on the SSC is shown below in Fig 27:

![Sensitivity Analysis for Settling Velocity](image)

**Fig 27: Sensitivity analysis for settling velocity**

The settling velocity has some major and minor impacts on SSC at various points along the river. An increased value of settling velocity, leads to decrease in peak concentration at majority of points. Thus, SSC is sensitive to major increases in settling velocity. Now, the sensitivity
analysis for suspended sediment transport capacity was performed. The value for the parameter was increased from the current computed value by 50%. The result is as shown below in Fig 28

![Graph showing varying SSC values](image)

Fig 28: Sensitivity analysis for Sediment Capacity

The suspended sediment transport capacity, as it seems, may be the most swing factors in the regions where there are high flows and major dispersion values. An increase in its value leads to perceptible increase in SSC values. A sensitivity table is made for various parameters as shown in Table 5.

Table 5: Sensitivity Analysis table

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<tr>
<td>ωS</td>
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<td>Max change = 18.298</td>
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<td>Sx</td>
<td>±50%</td>
<td>Max change = 91.2</td>
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<td>Kx</td>
<td>±50%</td>
<td>Max change = 10</td>
</tr>
</tbody>
</table>
The results show that the most sensitive parameters in the model are the suspended sediment capacity and the parameter alphas are the most sensitive. Other model parameters such as settling velocity and dispersion are also found to be having effect on resuspension and on SSC.

5.3 Linkage Analysis

In order to determine the TMDL, it is important to assess the magnitude of in stream sediment problems and the associated levels of sediment source reductions needed to address in stream problems. This section assesses the degree to which sediment reductions are needed from sources in the Amite River Basin to alleviate the instream sediment problems discussed in the Problem Statement section. The analysis can based on three methods of comparing existing and desired conditions for the watershed:

1. Comparison of average sediment loading rates per square mile in highly impacted and relatively unimpaired basins in Louisiana, and applying these comparisons in the Amite River Basin setting,
2. Qualitative analysis of the linkages between sources and in stream conditions, and
3. Comparison of existing and historical conditions with target levels for the in stream indicators selected in the next section.

5.4 Desired Endpoint for Amite River and Sediment Load Reduction

To develop a TMDL it is necessary to establish quantitative measures, or indicators, that can be used to establish the relationship between pollutant sources and their impact on water quality. Once an indicator has been selected, a target value for that indicator which distinguishes between the impaired and unimpaired state of the water body must be established (USEPA,
One such indicator is turbidity which is measured in terms of Nephelometric turbidity units (NTU).

Turbidity is the measure of the optical property of water that causes light to be either scattered or absorbed. Turbidity may be influenced by a number of factors but is primarily affected by suspended matter such as clay, silt, plankton, or microscopic organisms (APHA, 1992).

Numeric criterion for turbidity may be found in the EPA Sediment related water quality standards and Louisiana Water Quality Standards at C1113.B.9. This reads:

“Turbidity

a) Turbidity other than that of natural origin shall not cause substantial visual contrast with the natural appearance of the waters of the state or impair any designated water use. Turbidity shall not significantly exceed background; background is defined as the natural condition of the water. Determination of background will be on a case-by-case-basis.

b) As a guideline, maximum turbidity levels, expressed as nephelometric turbidity units (NTU), are established and shall apply for the following named water bodies and major aquatic habitat types of the state:

i.) Red, Mermentau, Atachafalaya, Mississippi, and Vermilion Rivers and Bayou Teche—150 NTU;

ii.) Estuarine lakes, bays, bayous, and canals—50 NTU;

iii.) Amite, Pearl, Ouachita, Sabine, Calcasieu, Tangipahoa, Tickfaw, and Techefuncte Rivers—50 NTU;

iv.) Freshwater lakes, reservoirs, and oxbows—25 NTU;

v.) Designated scenic streams and outstanding natural resource waters not specifically listed in Subsection B.9.b.i-iv of this Section—25 NTU.

vi.) For other state waters not included in Subsection B.9.bi-v of this Section, and in waterbody
segments where natural background turbidity exceeds the values specified in these clauses, turbidity in NTU caused by any discharges shall be restricted to the appropriate background value plus 10 percent. This shall not apply to designated intermittent streams.”

Thus, the criteria that was taken as benchmark for establishing TMDL was 50 NTU or 64 mg/L. This numeric target is prescribed by the EPA for the Amite River Basin. Based on this numeric target the load reduction is made.

5.5 Calculation of TMDL

The computed and the prescribed loading capacity (LC) can be given by the expression below:

\[
\text{LC (computed)} = \text{Discharge} \times \text{SSC (computed)}
\]

\[
\text{LC (Prescribed)} = \text{Discharge} \times \text{SSC (prescribed)}
\]

The estimate of the actual average load capacity from the Amite River watershed for 1990 was calculated as \(1.35 \times 10^6\) tons/year. The difference between the standard load capacity permissible for sustenance of aquatic life and actual load capacity derived from suspended sediment concentration (SSC) would be used for determination of load reduction. Due to variations in SSC concentration along the length of the river, there would exists different load capacity and difference would not be the same. For determining the load reduction, the largest difference between the two load capacities was taken into account.

Based on Sediment TMDL protocol

\[
\text{Load reduction} = \text{LC (computed)} - \text{LC (Prescribed)}
\]

The largest value of the difference was found to be **219,456 tons/day**. This value (i.e., the load reduction) is apportioned among the various sources of the sediments as to focus attention on the sources that are influenced by human activities. In establishing TMDLs, EPA generally apportions the loading capacity among: (1) the background loading; (2) the waste load
allocations for point sources; and (3) the load allocations for non-point sources. For this TMDL, there are no point sources, so the Waste load allocations equal zero. The background loading is not considered in this case and the TMDL is expressed as the sum of three major components namely load allocations, margin of safety and future growth.

Therefore, the TMDL allocations for the Amite River is shown below:

Allocations:

\[
\text{TMDL} = \text{Background Loading} + \text{Waste Load Allocation} + \text{Load Allocations} + \text{Margin of Safety} + \text{Future Growth}
\]

EPA considered several factors in setting load allocations for various source categories, including the effectiveness of available methods of controlling sediment from the particular source category, equity in imposing needed controls, and the feasibility of monitoring to determine compliance with the allocations. Even though there might be some point sources and NPDES dischargers in the watershed, it is assumed that the effect on water quality due to point sources is negligible. The existing loads contributed by these facilities are unknown and will need to be determined in the future through monitoring of effluent and ambient receiving stream flow. Therefore, the allocation for point sources is to be considered as future work. At this stage of the TMDL, the assumed condition would be effect of non point source pollution.

The composition of the watershed indicates a mixture of rural non-point sources and wetland which may contribute to the downstream impairment. These sources tend to become dominant under higher flow conditions. Since load reductions are primarily attributed to nonpoint sources, then the MOS should be relatively large. Assuming a MOS of 25% for the Phase I TMDL and taking the population growth in the Amite River basin to be around 1.3 % (USACE, 2005), the
future growth values is taken to be the same for TMDL calculations. Thus taking into all these values the final TMDL can be easily calculated.

The final calculations show that

\[
TMDL = 3693.28 \text{ tons/day} + 0.25 \times 3693.28 \text{tons/day} + 0.013 \times 3693.28 \text{ tons/day} = 4665.235 \text{ tons/day}.
\]

### 5.6 Impact of Unsteady Flow on Sediment TMDL

From the steady flow simulation, it came clear that at some portion of the river segment, there are very high concentration peaks. In steady flow, the discharge at every cross section remains same throughout the total time period. However, unsteady flow is the norm in natural rivers and streams. Thus, a consideration of unsteady flow would help in getting the impact of variability of flow on SSC concentration. In order to seek the impact of unsteady flow on sediment concentration, the erosion was calculated using the MUSLE model. Geometric data were entered, and then unsteady flow data that are required were entered. The boundary conditions at all of the external boundaries of the system, as well as in the internal locations were entered and the initial flow and storage area conditions were set at the beginning of the simulation. In HEC-RAS many different boundary conditions are available for performing unsteady flow simulation. Some of the most commonly used boundary conditions were Flow Hydrograph, Stage Hydrograph, Stage and Flow Hydrograph, Rating Curve, Normal Depth, Lateral Inflow Hydrograph, Uniform Lateral Inflow Hydrograph etc. The lateral inflow hydrograph for an inflow was put as a boundary condition in HEC RAS. The unsteady lateral inflow induces an unsteady pattern in the discharge at the particular cross section upstream. The unsteady flow analysis was then done in HEC RAS to get the appropriate flow parameters. The River Mile 95 is the largest tributary to the Amite River and therefore the sediment transport model was used to
find the SSC variation. The discharge at the river mile station 95 is as shown in Fig 29. In addition to the boundary conditions, initial conditions of the system at the beginning of the unsteady flow simulation were established. The flow value was entered at the upper end of the reach.

![Discharge at River Mile 95](image)

**Fig 29: Discharge at River Mile 95**

Due to this variation in discharge at the cross section, the suspended sediment concentration was found to varying and reaching a peak value close to 140 mg/L. The subsidence of peak concentration of SSC over the time can be attributed either to their transport towards the downstream or deposition as bed load. Fig 30 shows the temporal variation of suspended sediment concentration in the River mile station 95.
Fig 30: Temporal Variations of SSC at River Mile Station 95

The median value of the TMDL was taken in this case and was found to be 4290.069 tons/day and calculation was done in the same way as it was in the steady flow case. The median daily reduction was found to be 2653.29 tons.

\[
\text{TMDL} = 4290.069 \text{ tons/day} + 0.25 \times 4290.069 \text{ tons/day} + 0.013 \times 4290.069 \text{ tons/day} = 5418.357 \text{ tons/day}.
\]

The TMDL was calculated to be **5418.357 tons/day**. Thus, this value is higher compared to the TMDL due to steady flow. Thus, once can safely conclude that incorporation of unsteady flow factor into future TMDL works, would give a better picture of the Best Management Practices that is to be followed for controlling non point source pollution. Table 6 shows the suspended sediment concentration corresponding to the discharge values.
Table 6: Table of Output values for unsteady flow

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Chapter 6: Conclusions and Recommendations

The following points could be drawn as the summary of the thesis.

(1) Using mass conservation & Reynolds transport theorem, a new 1-D sediment dispersion and transport model has been developed for the Amite River sediment TMDL calculations. The new model has numerous source, sink and flow parameters and for its solution, parameter estimation was done appropriately.

(2) For the major sediment source, soil erosion estimation was done combining USLE and GIS. GIS is very useful compared to traditional methods by breaking up the land surface into many small cells which enables an analysis to be performed on both large regions as well as discrete areas. GIS not only generates inputs for USLE, but also displays outputs. Spatial variation of soil loss correlated with land use can be observed. Based on USLE and the average intensity rainfall of 1990, erosion value was found to be 5.41 tons/acre/year. This erosion rate represents the average annual erosion rate for the entire basin. This erosion value was used for sediment TMDL calculations under steady flow conditions. The MUSLE model which is a single event model was used to determine the soil erosion for the TMDL calculations under unsteady flow conditions.

(3) The flow parameters for both the steady and unsteady flow conditions were computed using the HEC-RAS. The calculated flow velocity and discharge of the Amite River vary in the range of 0.34 – 2.4 m/s and 285 – 771 $m^3/sec$, respectively.

(4) The 1-D model predicts a maximum sediment concentration of 114 mg/L and ranges between 3 mg/l to 114 mg/l. The average concentration of SSC in the Amite River was calculated as 25 mg/L.

(5) The suspended sediment concentration along the Amite River was not uniform and it varied along the river. Such a variation can be attributed to variation in major flow parameters.
(6) The tributaries had a lot of SSC and as a result of that, there were formation of concentration peaks at some sections.

(7) Based on the EPA’s and LDEQ’s water quality standard of 50 NTU, the modeling results predicted Sediment TMDL for the Amite River to be 4665.235 tons/day. The daily reduction in this case was found to be approximately 220 tons.

(8) The TMDL accounted not only for waste load allocation, but also for margin of safety (MOS) and future growth.

(9) Unsteady flow is found to have a significant effect on TMDL calculation. Event based rainfalls produce unsteady flows in the river and a higher erosion rate and sediment concentration in the river, resulting in a higher TMDL for the Amite River. The TMDL value for unsteady flow was found to be 5418.357 tons/day. The median daily reduction was found to be 2653.29 tons. Future development of sediment TMDL should be taken into account for the influence of unsteady flow on TMDL. The upper portion of the Amite River Basin is a major sediment producing area. The sediment coming from the upper watershed and in stream processes, however, impacts lower part. Although most of the major tributaries are prominent sources, sediments are also the result of channel and bank erosion. An erosion control plan can be suggested. For example, some land forming practices such as terraces on the steep slopes to reduce the slope lengths (LS factors), which will slow down the runoff velocities. If appropriate best management practices (BMPs) such as buffer zones are implemented at the sites that are near the stream’s drainage network and the TMDL targets can be met.

The results can be further improved by conducting more detailed field surveys to find out the different land use, water quality and current best management practices being adopted. The RUSLE i.e Revised Universal Soil Loss Equation (Renard et al, 1991), would be far more accurate and appropriate than USLE. More detailed and latest land use data in which agricultural
land use are subdivided into specific crops could be used. Comparing simulation results with measured data to calibrate the model for this specific study area would give a better idea about the fitting parameters. Furthermore, the usage of a one dimensional model for suspended sediment transport assumes that the sediment is well mixed in depth and lateral direction. Natural systems are so heterogeneous, that this assumption might not be fully valid. For numerical solution of advection dispersion equation; usage of appropriate grid size is essential, which if not factored in can result in numerical oscillation. To prevent this, a small grid size could be chosen. With the advent of computers and availability of high computing facilities working with a very small grid size could be possible and would yield better model results.
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Appendix A: Cross Sectional Profile of Amite River

Cross Sections:

Fig 31: Cross section 1

Fig 32: Cross section 2
Fig 33: Cross section 3

Fig 34: Cross section 4
Fig 35: Cross section 5

Fig 36: Cross section 6
Fig 37: Cross section 7

Fig 38: Cross section 8
Fig 39: Cross section 9

Fig 40: Cross section 10
Fig 41: Cross section 11

Fig 42: Cross section 12
Fig 45: Cross section 15

Fig 46: Cross section 16
Fig 47: Cross section 17

Fig 48: Cross section 18
Fig 49: Cross section 19

Fig 50: Cross section 20
Fig 51: Cross section 21

Fig 52: Cross section 22
Fig 53: Cross section 23

Fig 54: Cross section 24
Fig 55: Cross section 25

Fig 56: Cross section 26
Fig 57: Cross section 27

Fig 58: Cross section 28
Fig 59: Cross section 29

Fig 60: Cross section 30
Fig 61: Cross section 31

Fig 62: Cross section 32
Profile Plot:

Fig 63: Depth Profile Plot
Fig 64: X-Y-Z perspective plot

Fig 65: Top view of the Amite River Cross-sections
Appendix B: Parameter Variations and Advection Algorithm

Fig 66: Dispersion variation in Amite River

Fig 67: Depth variation in Amite River
Advection algorithm

for each i, j do

$F_{i,j} = 0; \ G_{i,j} = 0$

Calculate the flux based on the interfaces in x-direction and update increments.

for each i,j do

# Considering interface between cells $C_{i+1}$ and $C_{i-1}$

$U = u_{i,j}^{n+1}$

$V = v_{i,j}^{n+1}$

$R = c_{i,j} - c_{i-1,j}$
if \( U > 0 \) then \( I = i-1 \) else \( I = i \)

\[ F_{i, j} = F_{i, j} + Uq_{i, j} \]

# If method = 1 then end loop here if \( U > 0 \) then \( I = i \) else \( I = ij1 \)

if \( V > 0 \) then \( J = j+1 \) else \( J = j \)

\[ G_{i, J} = G_{i, J} - \frac{1}{2} \frac{k}{h_x} UVR \]

# If method = 2 then end loop here

\[ R = \text{Limited version of } R \text{ (Apply one of the four flux limiters)} \]

\[ S = \frac{1}{2} |U|(1 - \frac{k}{h_x}|U|)R \]

# If method = 3 then end loop here

\[ G_{i, J} = G_{i, J} + \frac{h}{k} VS \]

\[ G_{i-1, J} = G_{i-1, J} - \frac{k}{h_x} VS \]

# If method = 4 then end loop here

# To update increments and fluxes based on interfaces in Y direction, follow the same above steps with roles of \( i \) and \( j \), \( u \) and \( v \), \( F \) and \( G \) switched and replace \( h_x \), length of cell in X-direction by \( h_y \), length of cell in Y-direction. # Update the value of \( c \)

for each \( i, j \) do

\[ c_{i, j}^{n+1} = c_{i, j}^{n} - \frac{k}{h_x}[F_{i+1, j} - F_{i, j}] - \frac{k}{h_y}[G_{i, j+1} - G_{i, j}] \]
Vita

Pradeep Kr. Mishra was born on March 1st, 1979, in Orissa, India to Mrs Shanti Lata Mishra and Mr Uma Shankar Mishra. He graduated from National Institute of Technology, Silchar, with a Bachelor of Engineering degree in Civil Engineering in the year 2001. He came to the United States to join the graduate school at Louisiana State University Agricultural and Mechanical College in the Department of Civil Engineering in spring 2004. He defended his master’s thesis on 17th of October 2005.