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Hydrodynamic and sediment transport modeling of deltaic sediment processes

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HYDRODYNAMIC AND SEDIMENT TRANSPORT MODELING OF DELTAIC SEDIMENT PROCESSES

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ABSTRACT

This dissertation tests the hypothesis that the early phases of deltaic bar and distributary channel formation and sediment transport on an adverse slope could be simulated with a 2D finite element sediment transport model. The models used were RMA2 and SED2D modules of the TABS-MD model suite. A finite element mesh of the lower Atchafalaya River and the delta was developed, using the Surface Water Modeling System (SMS) software package. Calibration and validation of the model were performed, using data collected during field surveys and from available Atchafalaya River archived flow, suspended sediment, and dredging records.

In a test simulation in which adequate flow and sediment supply were provided in large quantity, sub-aqueous distributary mouth bar formed at the end of the feeder channel. As simulation continued, a more prominent distributary channel and sub-aerial levees were developed. When the model was changed to impose a no flood conditions on high points, formation of new distributary channels was observed.

The same model was used to determine a self-sustainable adverse slope or sediment ramp that could be used to divert sediments efficiently in a deltaic setting similar to the Atchafalaya Bay. A test slope of 1V to 51H was used in the model. After several simulations, the model tends to produce a much milder slope close to 1V to 412H. Five adverse natural slopes observed in the Wax Lake Outlet delta were compared with the model-suggested slopes. Adverse slopes at the Wax Lake Outlet delta varied from 1V to 340H – 850H, with 1V to 543H as the average.

Finally, a calibrated model of the Atchafalaya River and the delta was applied to develop a set of sedimentation/erosion curves that could be used by the engineers to

estimate scour and deposition for proposed artificial feeder channels. These curves suggest that a discharge of at least 11,325 cms (400,000 cfs) at Morgan City is necessary to transport sand into the delta. It was observed that even for a very high flood, sand deposition should be limited to within 1000 m (3281 ft) of distributary channel mouths.

CHAPTER 1. INTRODUCTION

The Mississippi River is perhaps the best studied and the most engineered of all the world's large river systems (Schumm and Winkley, 1994). In earlier times, the major engineering challenges were to control the tendencies for unpredictable channel migration and catastrophic flooding. These problems have largely been alleviated by the United States Army Corps of Engineers (USACE), at least in the lower reaches south of Cairo, Illinois, through a combination of continuous levees, artificial meander cut-offs, bank stabilization measures, and controlled release floodways (Schumm and Winkley, 1994). Where once the Mississippi entered the Gulf of Mexico through a number of distributaries, now there are only two: the Lower Mississippi that flows past New Orleans and discharges at the edge of the continental shelf, and the Atchafalaya that takes a shorter route to the west through an inland swamp basin and debouches into a system of shallow coastal lagoons (Figure 1.1).

The evolution of the Lower Mississippi and Atchafalaya system under the 70-year-old Mississippi River and Tributaries Project (MR&T) continues to present the USACE each year with new challenges. The river adjusts to management measures, such as a long-term reduction in sediment available for transport (Figure 1.2; Kesel, 1989), or an artificial control of water and sediment flow between the Mississippi and Atchafalaya Rivers. The artificial control may be regulated by relatively slow changes in bottom slope, conveyance, or flood flow line (Schumm and Winkley, 1994). The more interesting problems are not a function of river hydraulics, however, but of man's changing vision of what the river should do.

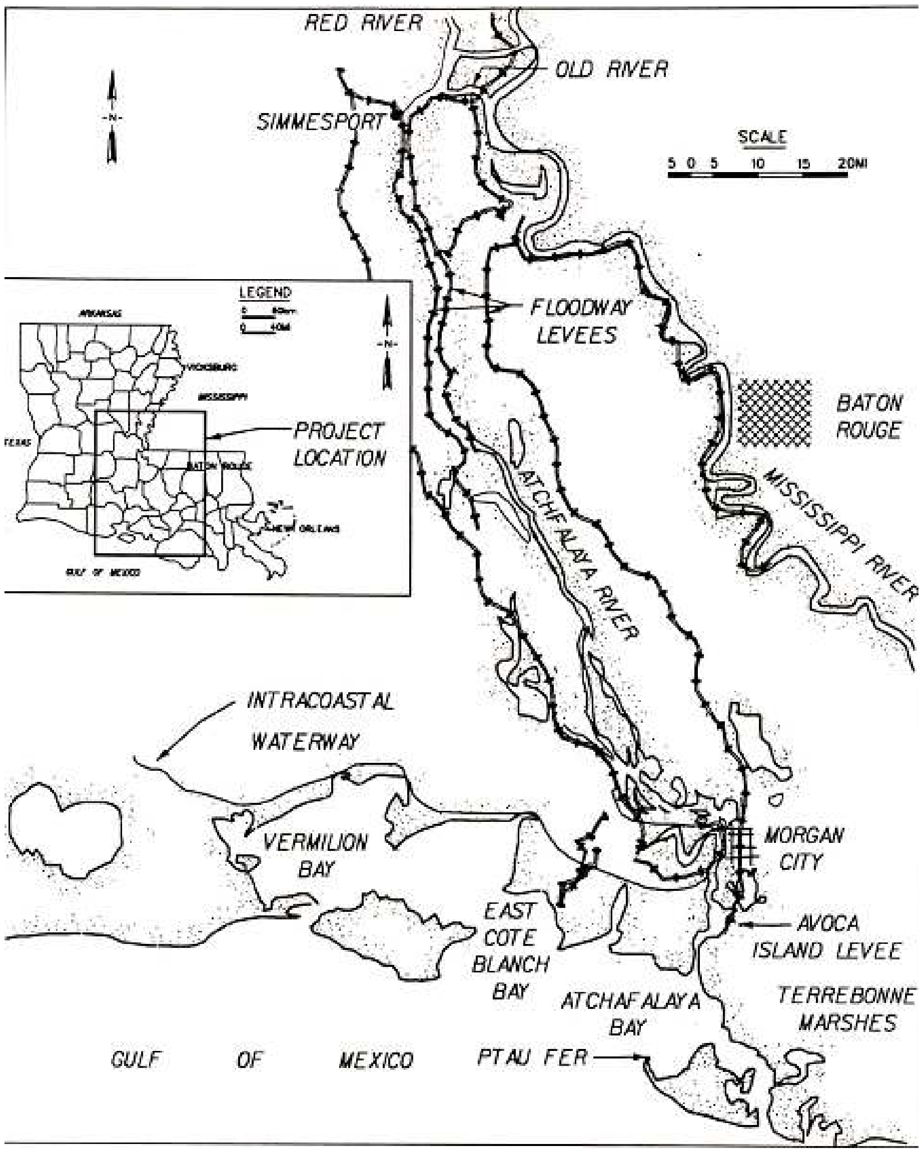


Figure 1.1. General location of the study area (modified from Donnell and Letter, 1992).

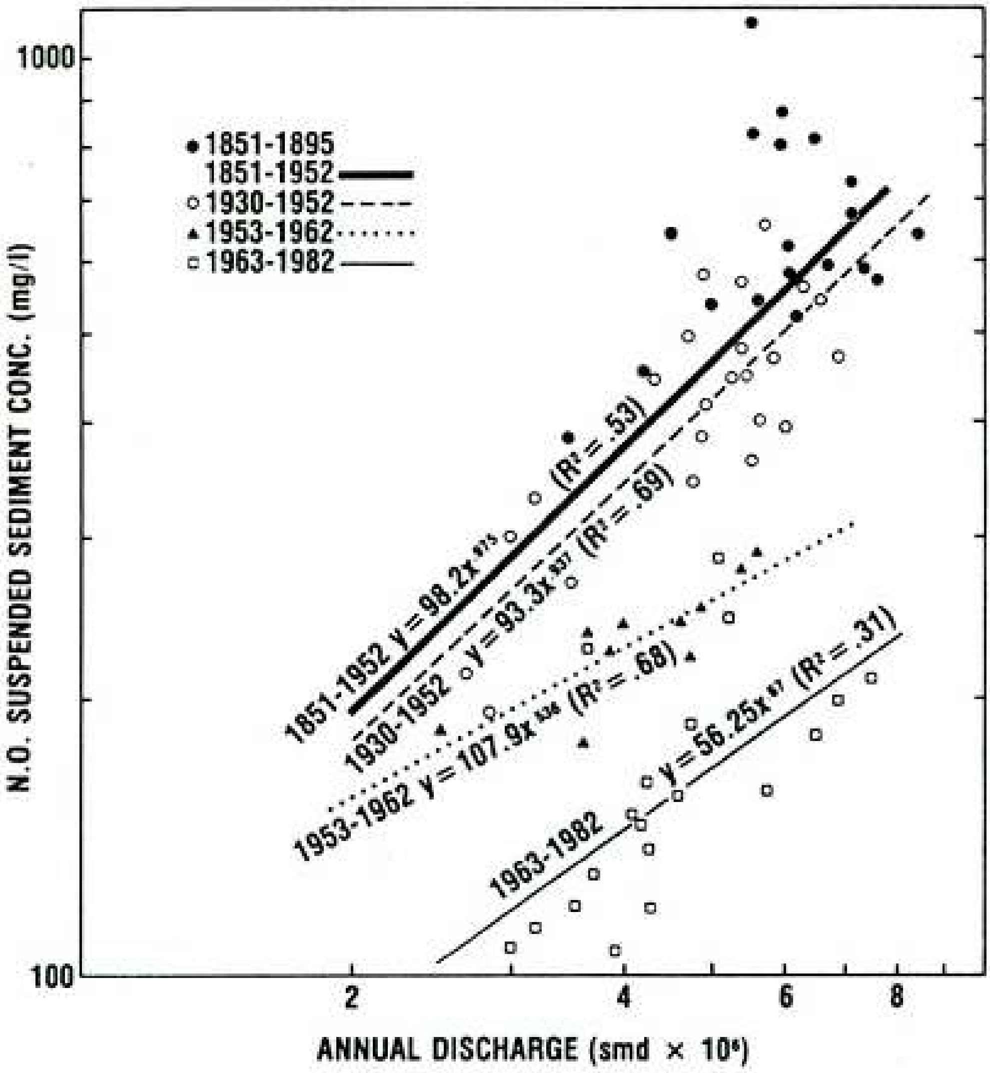


Figure 1.2. Relation between average annual sediment concentration and annual discharge at New Orleans (from Kesel, 1989).

Two of these changes set the stage for the work described here. First, world economics and commerce dictate that in order to remain competitive, ports on the Lower Mississippi and Atchafalaya Rivers must be able to accommodate ever larger and deeper-draft vessels in the future. Second, recreating a number of newly artificial distributaries, or diversions, downstream of Baton Rouge on the Lower Mississippi River is considered necessary to forestall a catastrophic loss of coastal wetlands within the deltaic plain south of New Orleans and Baton Rouge.

Sound and feasible engineering approaches to building new wetlands and nourishing deteriorating coastal ecosystems will require a cadre of engineers, well-trained in the use of highly sensitive predictive tools. Engineering for the environmental protection of generations to come should create confidence, avoid environmental consequences, and eliminate costs to life and property. For engineers, the ready availability of low-cost, high-speed desktop computers introduces a practical means by which complex numerical models originally developed as research tools may address real river management problems. These advantageous computer technology advancements were applied in this study to models developed over many years at the USACE Waterways Experiment Station. The models dealt with a classic river management problem, that of predicting the dynamics of sediment transport and associated land building in a deltaic setting over a full flood hydrograph.

Actively growing deltas associated with the Mississippi River system have been building in two locations within Atchafalaya Bay at the mouths of Atchafalaya River and Wax Lake Outlet (Figure 1.3). The Atchafalaya River, a controlled distributary of the

Mississippi River, currently is managed to carry 30% of the combined flow of the Mississippi and Red Rivers, measured at the latitude of 31 degrees north (USACE, 1993; Wells et al.,1984). Over the last several decades, deposition of Atchafalaya sediments has filled much of the large, inland lake system, and created two sub-aerial delta systems in the Atchafalaya Bay (Roberts et al., 1980; Tye and Coleman, 1989; Shlemon, 1972).

Development of the deltas carries benefits as well as liabilities (McAnally et al., 1991). The primary deltaic benefit represents an addition of new land that may possibly manifest the finest wildlife habitat in North America, situated on Louisiana's coast (van Heerden and Roberts, 1988; Shlemon, 1972). On the other hand, there is serious concern regarding a change in the river's hydraulic regime, causing siltation in the navigation channel and back-water flooding in the low-lying coastal areas (McAnally et al., 1991).

In the 1990s, managers actively sought to maximize the size and quality of both naturally created wetlands and those wetlands created by placement of dredged material (Figure 1.4) in the Atchafalaya delta. The past five years have brought newly constructed lateral channels in an attempt to restore lower Atchafalaya River efficiency in building natural deltaic wetland. Some of these channels fill rapidly, and so provide a minimal long-term stimulus to deltaic development. Others work efficiently to divert water and sediment from the artificial navigation channel that bisects the delta. Engineers seek a reliable modeling tool that will not only optimize designs for diversion channels, but also will predict ancillary effects on sedimentation within the navigation channel.

From an engineering standpoint, the Atchafalaya and Wax Lake deltas provide a natural laboratory for investigating the effects of two conditions that affect sediment

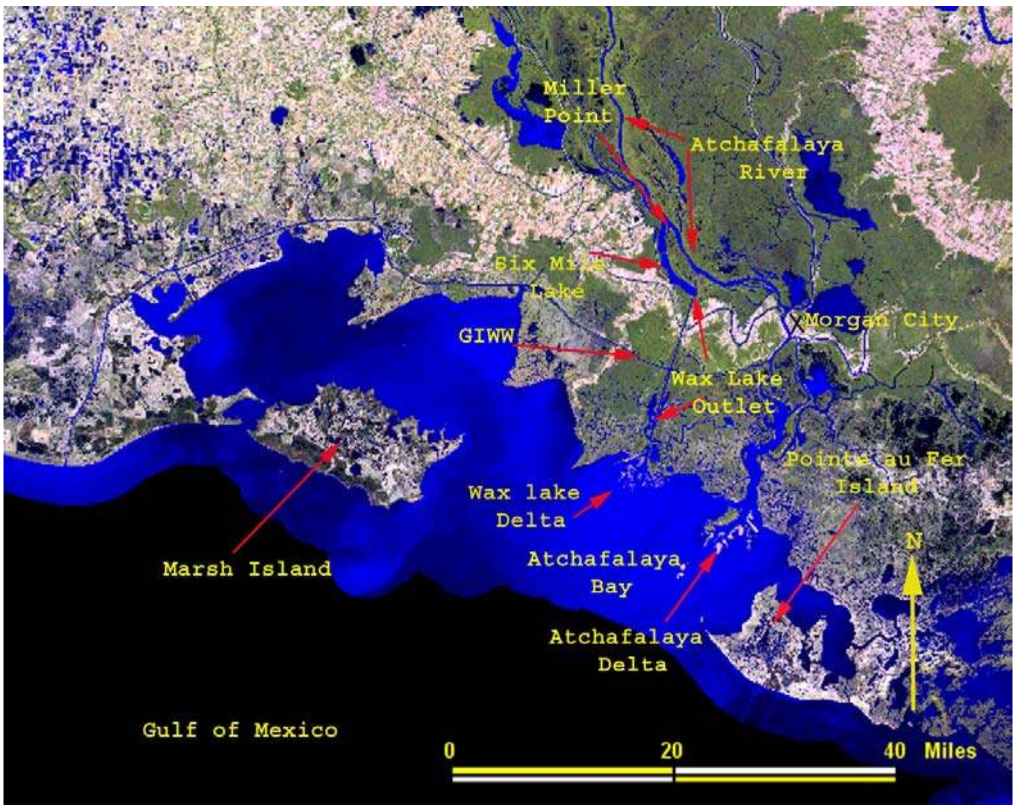


Figure 1.3. General location of the Atchafalaya River, Atchafalaya delta and Wax Lake delta.

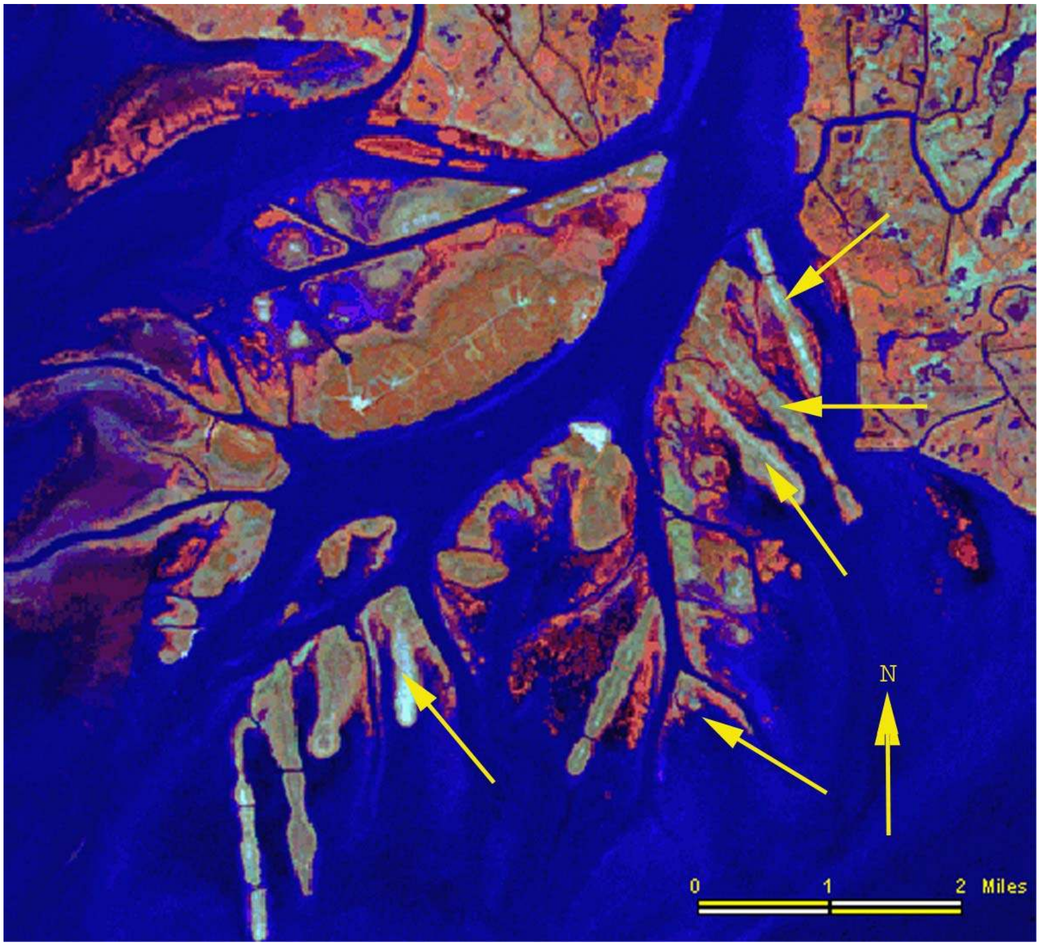


Figure 1.4. New delta lobes created by the dredged material in the Atchafalaya delta. Arrows indicate new delta lobes from dredged materials. (Source: 2000 TM Image from USGS).

transport in other settings: flow through a branching channel network, and flow over an adverse slope. A two-dimensional numerical sediment transport model was applied to investigate these problems.

Engineers working in the Atchafalaya system are interested in predicting the rate of delta growth, as well as the effects of channel elongation and bifurcation on navigation and flooding. In particular, researchers seek answers to the following questions (McAnally et al., 1991):

1) To what extent does delta growth increase backwater flooding?

2) Will navigation dredging requirements increase due to new delta formation?

3) Is there an optimal configuration for self-maintained lateral distributary channels?

4) Might delta creation be sustained without reducing the navigability of the river?

Several research and engineering studies have been conducted that provide tools for predicting the evolution of the Atchafalaya delta and its consequences (van Heerden, 1980 & 1983; McAnally et al., 1991). The approaches demonstrate one or more of the following types: field investigations, analytical solutions, numerical models, and physical models.

Long-term observations of the Atchafalaya delta by geologists revealed key information about processes involved in the formation and growth of a delta lobe. As observed by van Heerden (1980), the Atchafalaya River, upon entering Atchafalaya Bay, transfers from a confined to an unconfined flow state. At this transition, the depth of the river thalweg decreases significantly, resulting in an adverse slope at the river mouth. Further downstream, intermediate-sized silt particles can no longer be transported and are laid down at the delta front, an area seaward of the distributary bar (van Heerden, 1980). With each variance of river discharge seasonally and annually, zones of deposition shift back and forth, and boundaries of differing sediment types overlap.

The pattern of growth and resulting morphology of the Atchafalaya delta is also similar to the Mississippi subdeltas studied by Welder (1959) in the Mississippi bird foot delta. Both van Heerden (1980) and Welder (1959) observed that when flow enters an open area at the channel mouth, the deep central portions of the stream can no longer support the original high suspended load. This leads to deposition of the coarser fractions in mid-channel. Due to the pool of larger sediment, available suspended sediments are deposited by greater amounts in the center of the channel mouth, rather than on the edges. Once initiated, shoaling seaward of the mouth causes friction-induced deceleration and effluent spreading, which in turn increases the shoaling rate (Bates, 1953; Wright, 1977). The overall effect of such differential sedimentation is formation of the distributary mouth bar and branching of the channel (Figure 1.5).

The hydraulics of open channels have long been the focus of engineering studies, mainly for the purpose of designing irrigation, drainage, flood-control and navigation structure (Chow, 1964). Sediment transport also has been of interest, primarily because of its effect on scouring of bridges, siltation of reservoirs, and shoaling of navigable rivers and estuaries (Chanson, 1999; Chow, 1964).

As engineers become involved in managing rivers for new environmental purposes, reliable prediction tools, or models, must be available to compare the expected performance of various design options. When something more than an analytical solution

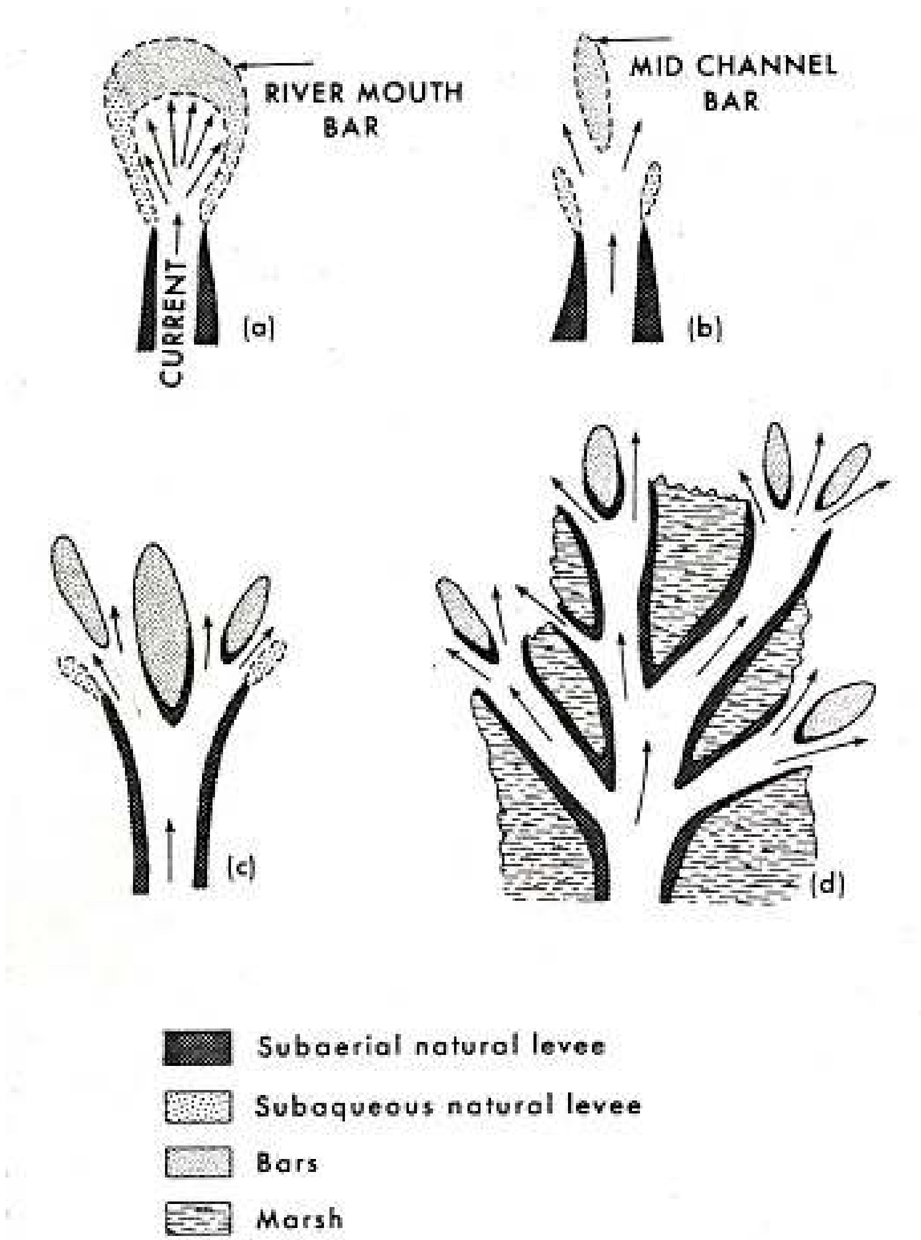


Figure 1.5. Schematic diagram showing the formation of the new delta lobes and distributary channels in the Atchafalaya delta (from van Herrden, 1980).

is required, physical and numerical models have been used to address sediment transport problems (Chanson, 1999). A physical model is a scaled-down representation of the prototype geometry, fabricated and investigated in a laboratory under controlled conditions. Physical models are commonly used to optimize structure design or ensure that a structure can operate safely (Chanson, 1999). Numerical models are computer programs that solve basic fluid mechanics and sediment transport equations (Martin and McCutcheon, 1999; Abbott, 1992).

Fluid mechanics equations can be solved in one-dimensional (1D), two-dimensional (2D), or three-dimensional (3D) spatial schemes (Martin and McCutcheon, 1999; Abbott, 1992). Solving these equations in their three-dimensional forms for flow and sediment transport is extremely difficult and has become feasible only as increased computer power makes numerical solutions practical (Martin and McCutcheon, 1999). One- and two-dimensional solutions have been possible for some time, and are used wherever they are appropriate. All real-world situations are 3D, but a model is an apropos simplification of the real-world. In a 1D model, averaging is done along the cross-section, but the method is generally considered to be inappropriate for complex estuary flow and sediment transport.

Delta growth in a river mouth is a long-term process driven by the flood cycle. To predict delta growth by a numerical model, the model should be capable of simulating at least one entire flood cycle. Temporal complexity of the flood and its effect on the sediment transport must be more important than the spatial complexity obtained by a 3D model. Although a detailed geometry could be represented with a 3D model, only a portion (few days) of the flood hydrograph could be simulated, due to higher computational requirements. Thus, the characteristics of the main forcing function of the delta building process would be unrepresented.

Forcing functions and transport processes observed in estuaries such as turbulent flows, tidal mixing, wind stress, wave action, thermal stratifications, and coastal currents and storm surges, are generally three-dimensional in nature (Martin and McCutcheon, 1999). In that sense, sediment transport and delta building processes in the Atchafalaya Bay can be considered a 3D problem.

Valid assumptions and simplifications can be made to reduce the problem to a 2D sediment transport in the Atchafalaya delta (Donnell et al., 1991). In the 2D model, only important prototype properties would be included in the model formulation.

Hydrodynamic and sediment transport models have been widely used in the engineering community to understand deposition and scour in rivers. Donnell et al. (1991) used a 2D model (TABS-2) to study sand bar development and delta front deposition. These models allowed engineers and managers to quantify the benefit of increasing flow and sediment on the ecosystem of an estuary with some success (Donnell et al., 1991; Donnell and Letter, 1992; USACE, 1999).

This dissertation uses a numerical model for sediment transport to test the hypothesis that delta lobe formation observed at the river mouth can be simulated quantitatively. In a 2D parameterization, two experiments were designed to test the ability of the model to (a) simulate development of a distributary mouth bar when flow enters an open area from a confined channel, and (b) predict the sediment transport capacity of the Atchafalaya River over a full flood cycle.

The hypotheses tested in this dissertation are:

1. Sedimentary features associated with delta development, specifically river mouth bar formation and channel bifurcation, can be simulated quantitatively using a two-dimensional, depth-averaged, finite-element hydrodynamic coupled to a sediment transport model.

2. The lobe becomes an obstruction in front of the feeding channel, forcing water to flow around it. Consistent flow around the lobe initiates the creation of two channels.

3. There exists an adverse slope configuration that will maximize sediment transport to the bay.

4. Sedimentation or erosion at critical locations in a river delta can be predicted for engineering purposes over a full flood cycle, using a set of model-generated graphs for various discharge and sediment supply regimes.

CHAPTER 2. LITERATURE REVIEW

2.1 Sediment Transport Processes and Bar Formation

Methods of predicting sediment transport and bar formation range from numerical to empirical with physical modeling lying between these two end points.

2.1.1 Computational Sediment Transport Modeling

Flows in open channels are described by a set of partial differential equations for computer simulation of hydrodynamic and sediment processes (Chaudhry, 1993; Martin and McCutcheon, 1999). Analytical solutions for these equations are not available, except for simplified, one-dimensional cases. Therefore, these equations are solved using numerical methods. Mathematically represented simulations are an efficient way to estimate the time and space-dependent sediment processes (van Rijn, 1989). There are numerous mathematical models available to simulate sediment transport and depositions in one-dimension (1D), two-dimension (2D), and three-dimension (3D) (Martin and McCutcheon, 1999; Abbott, 1992).

2.1.1.1 One Dimensional (1-D) Models

To achieve a practical solution of the governing equations in one-dimension, model parameters are horizontally and vertically averaged over a cross-section of the water body (Martin and McCutcheon, 1999). In this representation, model parameters have the same value over the entire width of the cross section. A detailed review of one-dimensional models, together with their numerical solution methods and applications, can be found in Cunge et al. (1980), Jansen et al. (1979) or review of De Vries et al. (1989). Widely used one-dimensional models such as Mike 11 (DHI, 2003) and HEC-6 (USACE,

1993) have been used to study sediment transport, scour and deposition in large and small rivers, particularly as affected by engineered channels and structures (DHI, 2003; USACE, 1982).

One-dimensional models are virtually the only numerical tool available to simulate morphological changes occurring over years in rivers (van Rijn, 1989). They are relatively easy to set up and calibrate quickly on desktop computers. Assumptions of 1D flow may not be valid in many situations. Flow in a channel along varying cross-section, changing alignment, or complex tidal flow in the estuaries are some of these examples.

2.1.1.2 Two Dimensional (2-D) Models

Two-dimensional models can be laterally or vertically (depth) integrated. A laterally integrated model solves the laterally integrated momentum and continuity equations for the fluid and the sediment phases (Smith and O'Connor, 1977). Appropriate applications for a laterally integrated model are in the design of pipelines, tunnel trenches and settling traps for irrigation canals (van Rijn, 1989; Celik and Rodi, 1988).

Two-dimensional (depth integrated) sediment transport models are based on the depth-integrated equations of motion and continuity linked to a depth-integrated sediment transport model (Boer et al., 1984; McAnally et al., 1986). The water surface elevation, velocity, sediment concentration, deposition or scour is computed at each of many points across the cross-section. The model parameters, however, are assumed to be uniform through the water column at each computational point.

Examples of the depth-integrated models are Struiksma et al. (1984) and Wang (1989). Struiksma et al. (1984) computed bed evolution in a river bend using the sediment transport formula of Engelund and Hansen (1967). Wang (1989) studied sediment distribution in a partially closed channel with steady flow. The two sediment transport models most widely employed in engineering practice are MIKE 21 (DHI, 2003) and TABS-MD (Thomas and McAnally, 1990). MIKE 21 (DHI, 2003) was developed by the Danish Hydraulic Institute and is a finite difference sediment transport model that is increasingly gaining acceptance inside the United States. Similarly, TABS-MD (Thomas and McAnally, 1990) has been widely used by the engineering community since its development in the early 70s by the USACE Waterways Experimental Station.

A 2-D model is necessary if the problem involves complicated circulation patterns and unsteady flows within the model domain. However, these models are more time consuming to set up than 1-D models, and require much more computer time to run. Therefore, careful planning and analysis is needed to develop the optimum trade-off between the density of the computational mesh or grid and the resulting run times, requirements for computer memory, and storage (Moffatt & Nichol Engineers, 2000).

2.1.1.3 Three Dimensional (3-D) Models

Three-dimensional models are based on the 3D-mass balance equations or the convection diffusion equations for suspended sediment transport (van Rijn, 1989). In most three-dimensional models, the flow field and sediment concentration computations are integrated and computed for each time step. In three-dimensional models, both the horizontal and the vertical components of the sediment transport processes are considered.

Three-dimensional models provide the most complete, quantitative representation of any hydrodynamic system. The calibration data requirements are more extensive and expensive (van Rijn, 1989), because a comprehensive field program is required to capture the complexities of flow in three directions (Moffatt & Nichol Engineers, 2000). Often those data are approximated from literature, rather than from field observation. Three-dimensional models can provide insight into the short-term effects of a proposed structure on a particular river management option, but lengthy simulations that model morphological evolution are not currently feasible.

Three-dimensional models should be used when flow and sediment transport are stratified (Martin and McCutcheon, 1999). An example might be where freshwater flows over a salt-water wedge, or warm water overrides colder waters (van Rijn, 1989). Many 3D-models have been applied most frequently in the laboratory (O’Connor and Nicholson, 1988) to small area field sites (van Rijn et al., 1989). Computations for larger model domains in estuaries or the continental shelf are typically lumped to a single day or one tidal cycle (O’Connor and Nicholson, 1988). The application of a 3D model is most necessary near or around a hydraulic structure where flow separation and vortex characteristics are truly three-dimensional, and sedimentation processes are complex (van Rijn 1987; van Rijn et al., 1989). Examples of some of the most widely used 3D models are RMA11 (Resource Management Associates, Inc., 2003), ECOMSED (HydroQual, Inc, 2003), CH3D-SED (Chapman at el., 1996), Delft-3D (Delft Hydralics, 2003). When CH3D-SED was recently applied to the Mississippi-Atchafalaya System at Old River, it was used only to qualitatively verify a proposed sediment rating curve. It was concluded that more work would be needed to mature the model as a professional engineering tool (Louisiana Hydroelectric Limited, 1999).

Whether the problem is solved in 1D, 2D or 3D, either a finite difference (FD) or finite element (FE) method will be used. In the finite difference method, a typical solutions grid is a network of equally spaced orthogonal lines with computational nodes at the intersections or in the center of each square formed. Generally, finite-difference techniques lead to faster solutions than the finite-element method. However, in a finite difference model, the boundaries of channels and other water bodies are approximated by stair-step edges following the grid. A very fine grid is required to represent land elevation, water edges, and the bottom of the water features in detail. If a high resolution depiction of the geometry is required, as in the current application, then the computational advantage of FD over FE may be diminished.

The configuration of a water body can be represented more accurately in a FD model by using the boundary-fitted coordinate or finite difference curvilinear method. This discretizes along boundaries and contours, then uses transform relations to map the discretizations to a rectangular grid for solution. The basic equations are modified to represent currents and tides in the transformed system. Again, the added complexity introduced reduces the FD advantage over FE.

FE solutions discretize the area of a water body into triangular or rectangular elements. The elements need not be the same size or shape, and their edges may be curved. Nodes are placed at vertices and midway between vertices. As described above, the FE technique offers a capability to precisely represent the geometry of a river, estuary, or marsh. Small elements may be used in areas where detail is desired, and larger elements in areas of less interest, allowing some efficiencies in computer usage. The numerical solution schemes for FE models are not as efficient as those used for FD models. Although the individual time step for FE models can be substantially longer than those required for finite difference models, a trade-off in the length-of-time step required for numerical stability offers some advantage. Overall, however, calculation time can be significantly greater for a finite element model if a large number of fairly small elements is used to describe the geometry.

2.1.2 Geomorphological Modeling

Geomorphological conceptual models for the growth of sand bars or deposition of sediments at river mouths are based on field observations or experiments (Welder, 1959; van Heerden, 1980 & 1983; Hatanaka and Kawahara, 1989). They provide the calibration data for numerical approaches, but are of limited utility from an engineering perspective.

Based on the observation of the delta growth and abandonment processes, Welder (1959) first proposed the delta growth model for the Mississippi River delta. Later based on Welder's (1959) research, similar observations were made in the Atchafalaya River delta by van Heerden (1980). Based on his research in the Atchafalaya River, van Heerden (1980) proposed a four-step delta growth and channel bifurcation model. Ashworth et al. (2000) studied the initiation and evolution of a large sand bar, by means of successive bathymetry surveys over a 28-month period. The sand bar was located in one of the largest rivers in the world, the Jamuna River in Bangladesh. Based on repeated bathymetric surveys, combined with bar top surveys, Ashworth et al. (2000) proposed a six-stage model for mid-channel bar growth in the large, sandbed river.

2.1.3 Physical Modeling

Physical models frequently offer an alternative approach to sediment transport problems that are difficult to simulate computationally. According to Dalrymple (1985), physical models integrate the appropriate equations governing the processes without the simplifications that are required for analytical or numerical models. Fixed and movable beds have been utilized for river and coastal studies. Scaling effects are reasonably understood for fixed-bed models (Dalrymple, 1985; Hudson et al., 1979). Less understood are the scaling effects inherent in the material used to represent sediment in a movable bed physical model.

A common scaling problem arises when the prototype grain-size is diminutive; geometric scaling of the sediments results in selection of a model bed material below the diameter boundary between cohesive and non-cohesive sediment (about 0.065 mm) (Dean, 1985; Dalrymple, 1985; Hudson et al., 1979). Distortion of the scale model, i.e., stretching vertical or horizontal length scales, has been suggested as a means for overcoming the inability to reduce the sediment to model scale.

Although many scaling laws have been suggested that require model distortion, this practice is still viewed with skepticism by some. Dean (1985) reviewed several studies, and concluded that the state of knowledge on movable bed models was largely qualitative. There is a potential that artifacts of the laboratory setting can influence the process being simulated to the extent that suitable representation of the prototype is not possible. Other laboratory effects arise from the impact of boundaries on the process being simulated, resulting in an inability to reproduce realistic forcing conditions (Dalrymple, 1985). Atchafalaya delta sediments are fine sand with a mean diameter close to 0.1 mm, which approaches the cohesive boundary; this characteristic suggests that sediment scaling might introduce significant problems.

2.2 Background Information of the Study Area

The Atchafalaya River is a major distributary of the Mississippi River; consequently, the river is analogous to artificial diversions now constructed, or planned to restore, the coastal wetlands through a broad basin defined by flood protection levees before the river discharges into Atchafalaya Bay through the lower Atchafalaya River (LAR) and Wax Lake Outlet (WLO) (Figure 1.1). Atchafalaya Bay leaves the artificially leveed river delta system at the latitude of Morgan City and in the Wax Lake Outlet. The Wax Lake Outlet is located in the lower Atchafalaya River basin, approximately 10 miles west of Berwick. The Wax Lake Outlet is an artificial flood conveyance channel constructed in 1941. The Outlet extends south from Six Mile Lake, across the Teche ridge, then 20 miles to Atchafalaya Bay (US Army Corps of Engineers, 1938; Latimer and Schweitzer, 1951).

Although a monk accompanying La Salle's expedition documented the Atchafalaya as a distributary of the Mississippi in 1542, the river was considered an insignificant stream, choked with debris from the Mississippi and Red Rivers, until the nineteenth century (Latimer and Schweitzer, 1951) (Figure 2.1). After the successful

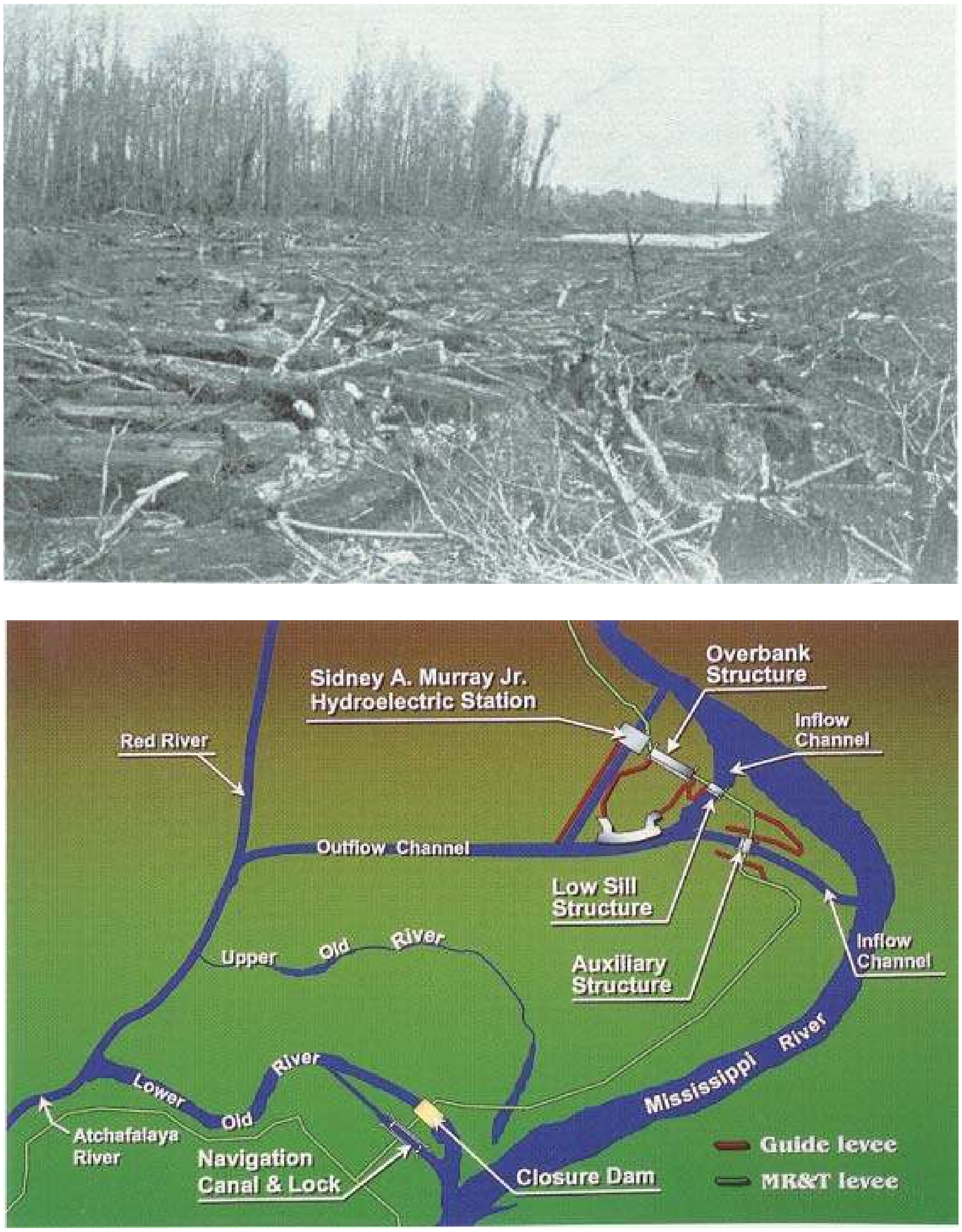


Figure 2.1. Picture of the massive log jam (top) at the mouth of Atchafalaya River and modern water control structures at the confluence of the Mississippi, Red, and Atchafalaya River (bottom); (Modified from Pozzi, 1998 and USACE, 1993).

clearance of log jams in the mid-1800s, the river gradually increased its discharge over the next century; now the river is poised to become a new Mississippi main course to the Gulf (Fisk, 1952).

From its junction with the Old River segment of the Mississippi River, the Atchafalaya flows 141 miles before entering the sea. In contrast, the Mississippi River winds 301 miles to Head of Passes and 332 miles to the mouth of Southwest Pass (Fisk, 1952). This inherent difference in gradient created a condition conducive to the abandonment of the parent channel capture of the full flow of the Mississippi River through its Atchafalaya distributary (Fisk, 1952). To prevent this, a control structure was built at Old River in 1963 (Figure 2.1). The Atchafalaya has since been limited to approximately 30% of the combined Red and Mississippi River flows at the Old River Cutoff (Wells et al., 1984; Wu, 1987).

The period from the 1800s to the early 1950s is generally considered to have contributed to insignificant deltaic sedimentation in Atchafalaya Bay (Morgan et al., 1953; Shlemon, 1972). During this period, the Atchafalaya River increased its discharge, capturing up to 25% of the Mississippi's flow. The major portion of the river's sediment load, however, was deposited in lakes and other catchments in the basin (Roberts et al., 1980; Tye and Coleman, 1989). Prior to the early 1950s, virtually all the sediment discharged into Atchafalaya Bay were clays, which were resuspended by waves and carried out past the Point Au Fer shell reef (Cratsley, 1975).

The decade of 1952 to 1962 marked the beginning of increased sedimentation, which was initially observed in the vicinity of the lower Atchafalaya River mouth

(Shlemon, 1972). Rapid deposition of upper prodelta sediments began at this time, consisting of parallel laminated clays and silty clays (van Heerden and Roberts, 1988). Variations in the thickness of the upper prodelta unit and other clues gleaned from sediment cores indicate that a sub-aqueous distributary channel system was established at this time (van Heerden and Roberts, 1988).

In 1972, small shoals became sub-aerial around the mouth of the lower Atchafalaya River. Those on the western side were composed primarily of dredged material generated from navigation channel maintenance, but those on the eastern side were the product of natural deltaic aggradation (Roberts et al., 1980). The following year, 1973, brought an exceptionally high and early flood. For seven months of that year, record levels of water and sediment were delivered to Atchafalaya Bay (Roberts et al., 1980). As a result, well-developed sub-aerial delta lobes became evident on each side of the navigation channel (Roberts et al., 1980). Above-normal discharges also occurred in the following two years. Scour in the lower reaches of the distributary channels, due to those three flood seasons, nearly doubled the suspended sediment carried by the river and most significantly, increased the amount of sand available for rapid delta growth (Roberts et al., 1980). By the end of the 1976 flood season, well-developed distributary mouth bars were evident at the mouths of both the LAR and WLO (Roberts et al., 1980).

The U.S. Army Corps of Engineers constructed a rock weir inside the basin about 15 miles north of Morgan City/Berwick in 1987. It stretched across the western branch of the river, flowed through Grand Lake, and into Wax Lake Outlet (Kemp et al., 1995). In 1994, the weir was removed to reduce flooding in Morgan City. Creation and removal of the weir significantly changed the hydrodynamics and sediment transport capacity of the system for the seven years it was in place, but its long-term impact, if any, is unknown (Kemp et al., 1995; FitzGerald, 1998).

Although the lower Atchafalaya River delta has been heavily manipulated by dredging to enable navigation, the Wax Lake Outlet delta has been infrequently dredged since creation of the Wax Lake Outlet. FitzGerald (1998) estimated the rate of lower Atchafalaya River and WLO delta growth, based on digital terrain modeling. For areas above –2.0 NGVD, the LAR delta was growing at the rate of 3.2 km^2/yr; WLO delta growth was 3.0 km^2/yr (FitzGerald, 1998).

2.3 Previous Atchafalaya Studies

Numerous research studies have been published from a wide range of perspectives to investigate the behavior of the Atchafalaya River and the development of the deltas in the Atchafalaya Bay. Early studies on the Atchafalaya River have been mainly from geologic and geomorphologic points of view. Research works published on the Atchafalaya system can be categorized as follows: 1) geologic and geomorphologic studies; 2) generic analyses of delta growth; 3) quasi-two dimensional modeling; 4) analytical solution of the Atchafalaya flow; 5) one-dimensional modeling; and 6) two-dimensional modeling.

2.3.1 Geologic and Geomorphologic Studies

Fisk (1952) showed that the Atchafalaya River was a distributary of the Mississippi River by 1542. Since then the hydraulic efficiency of the Atchafalaya has improved; both channel width and depth increased with each downstream extension of the levees. Bank caving was common, and coarser grained sediment deposited progressively farther downstream (Fisk, 1952).

Based on historical documents, Fisk (1952) showed that maintenance of navigational channels upstream through Grand and Six Mile lakes demanded continuous dredging. Consequently, competence of the lower Atchafalaya River has increased and coarse-grained sediments have been carried into Atchafalaya Bay at a greater rate than expected for wholly natural delta progradation (Fisk, 1952; Shlemon 1972).

According to Thompson (1951), the bottom topography of Atchafalaya Bay, until 1950, had changed little from the configuration mapped in the bathymetric survey of 1858. Similarly, in 1953 Morgan et al. (1953) found that almost no filling along the shore had occurred from the 1935 hydrographic survey to the time of their soundings in 1952. It thus appeared likely that significant filling of Atchafalaya Bay did not commence until after 1952 (Shlemon, 1972).

Shlemon (1972) reported that the greatest change in bottom topography between 1952 and 1962 occurred in the eastern part of Atchafalaya Bay; the change reflects the dominant contribution of sediment from the lower Atchafalaya River. Rapid deposition of upper prodelta sediments began at this time, consisting of parallel laminated clays and silty clays (van Heerden and Roberts, 1988).

Garrett (1971 in Shlemon, 1972) outlined the probable future configuration of the Atchafalaya delta from sediment load measurements in the Wax Lake Outlet and the lower Atchafalaya River. Van Heerden (1980 & 1983) investigated the developmental mechanisms and natural depositional facies of the Atchafalaya delta. The focus of those studies was the eastern portion of the delta, which at the time was relatively undisturbed by human modifications. van Heerden (1980 & 1983) determined that development of the area was caused mostly by the processes of channel bifurcation, seaward extension of distributary channels, upstream accretion of delta lobes, and lobe fusion by channel abandonment.

Roberts and van Heerden (1992) observed that, like the Atchafalaya delta, the delta at the mouth of the WLO also began its sub-aerial development with the flood of 1973, yet its growth pattern took a much different shape. Prior to 1980, Wax Lake and surrounding water bodies upstream of the bay were acting as sinks to the Outlet's sediment supply. In contrast to the eastern Atchafalaya delta, the processes of channel elongation, lobe fusion and upstream growth occurred simultaneously (Roberts and van Heerden, 1992). This indicated a more efficient retention of sediments by the WLO delta system (van Heerden, 1994).

Mashriqui et al. (1997) documented the statistical relationship between suspended sand supply at Morgan City and dredging in the bay, and conducted additional relevant historical analyses. Mashriqui et al. (1997) concluded that the proportion of sand in the suspended sediment supply would continue to increase as channels in the basin matured further. Accordingly, dredging volumes will also increase for a given flood magnitude until the Wax Lake Outlet captures a volume of sand proportionate to discharge.

<u>2.3.2 Regression Analyses of the Delta Growth</u>

In this method, observed historical phenomenon relative to deposition within the bay was connected to future delta growth with regression equations (Letter, 1982). Future delta growth was predicted from an initial bathymetric condition. The regression model (Letter, 1982) predicted a nearly-linear trend of delta growth with 49.2 km^2 (19 mi^2) at year 10 and 225 km^2 (87 mi^2) at year 50 (Figure 2.2).

The regression analyses was based on the field data, provided predictions, and permitted a simple form of sensitivity analyses. The limitation of this approach is that it is a statistical tool, rather than a dynamic composition-and-supply model; consequently, the approach was incapable of addressing changes in the sediment composition and supply that control the delta-building processes.

2.3.3 Generic Analyses

Wells et al. (1984) performed a generic or comparative analysis of the Atchafalaya River delta that drew on geomorphological information available from other deltas formed under similar environmental conditions. Wells et al. (1984) concluded that the delta will grow at a rate of 4.1 km^2/yr (1.6 mi^2/yr) and the WLO delta will grow at a faster rate than the Atchafalaya River delta, at least until 2030 (Figure 2.3).

The generic method is somewhat subjective with respect to the choice of deltas for comparison (Shlemon, 1972; Wells et al., 1984). The approach cannot address modifications made to the river for management reasons. Since the Atchafalaya River is highly manipulated by human needs, long-term generic models may not predict future trends if future management of the river is significantly different from that of other deltas, or from the way it was managed in the past.

2.3.4 Quasi-two Dimensional Numeric Model

A quasi-two dimensional numeric model was developed as a modification of the 1D HEC-6 model (Thomas et al., 1988). This modified model provided for the lateral

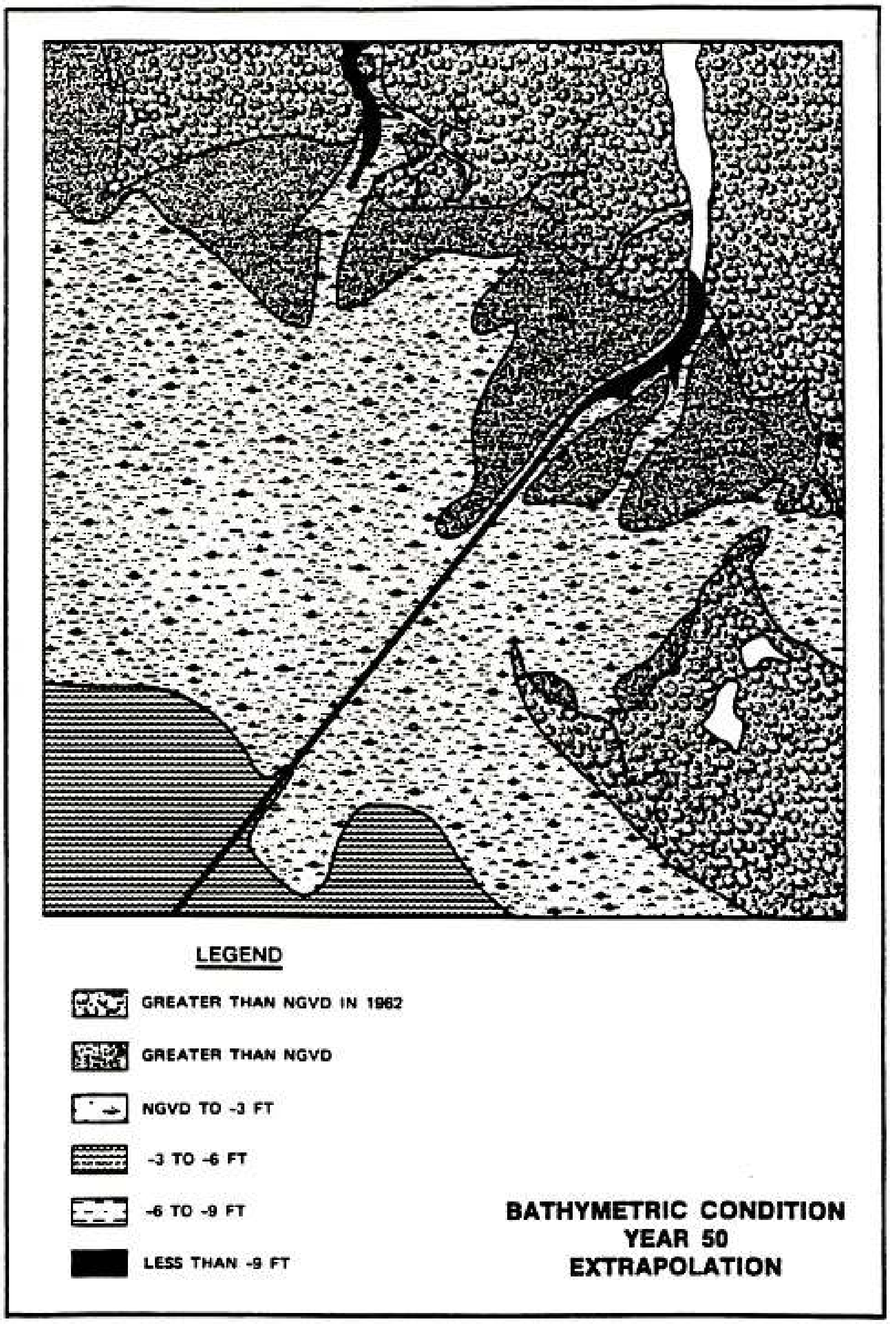


Figure 2.2. Predicted year 50 bathymetry for the selected extrapolation sequence from regression analysis (from Letter, 1982).

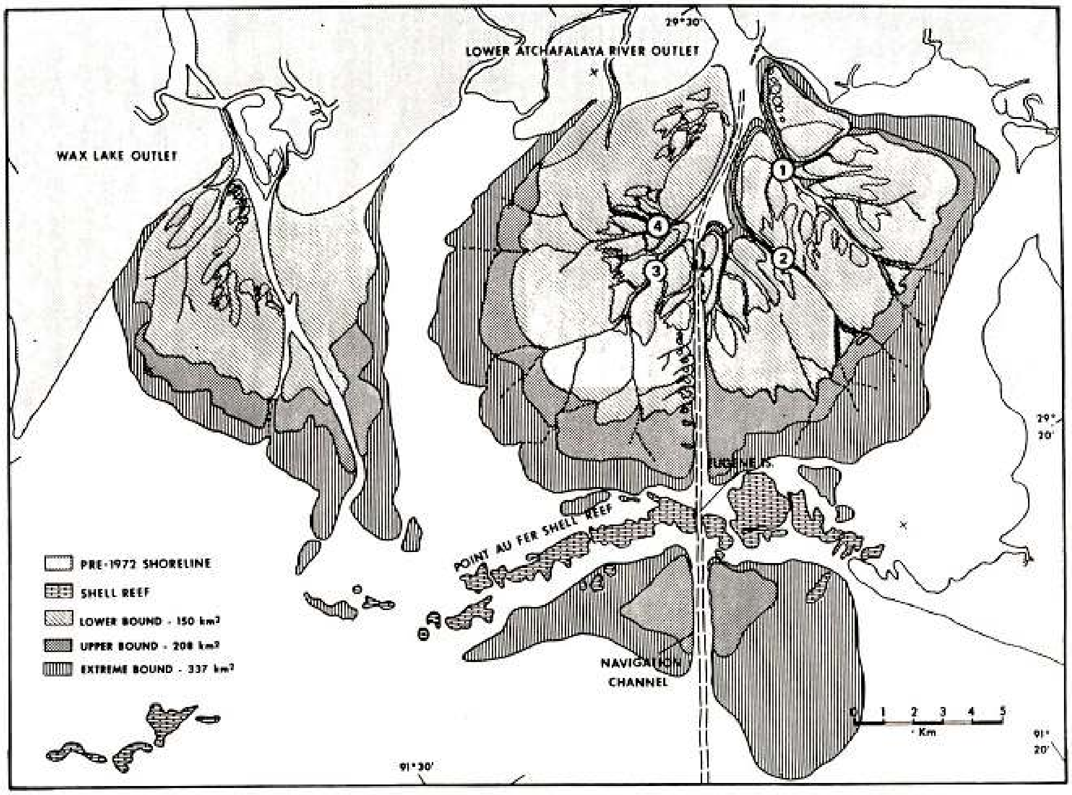


Figure 2.3. Configuration of the sub-aerial land in Atchafalaya Bay predicted in the year 2030 (from Wells et al., 1984).

transport of sediments. Continuity of sediment mass was conserved during simulations. Future prediction of the delta was made at 10-year intervals from 1977 to 2030. It was observed that the peak growth would occur at year 40, with a predicted 33 mi^2 sub-aerial delta growth (Figure 2.4). Delta growth is a balance between constructive fluvial processes and destructive marine influences (waves).

Limitations of this model were its 1D assumptions. Shell reef had never been removed from this model; consequently the model was incapable of incorporating wind and wave effects.

2.3.5 Analytical Model

An analytical prediction of the future delta growth was developed by Wang (1985), based on the assumption that Atchafalaya flow may be compared to turbulent jets issuing from river outlets to a quiescent bay. The analytical model used a solution of the hydrodynamic equation coupled with an advection-diffusion mass transport equation. This research resulted in a predicted delta growth rate of 7.8 km^2/yr (3.0 mi^2/yr) (Figure 2.5).

Although this method used an exact solution of the problem domain, the major shortcoming was that like the 1D approximation, it did not account for the wind or wave action on the system. It also never accounted for effects of the sub-aerial delta as it developed, as the jet theory breaks down when depths become sub-aerial.

2.3.6 Numerical Modeling

The most recent studies of the Atchafalaya River and deltas have been done under the auspices of the Atchafalaya River re-evaluation study conducted by the U.S. Army

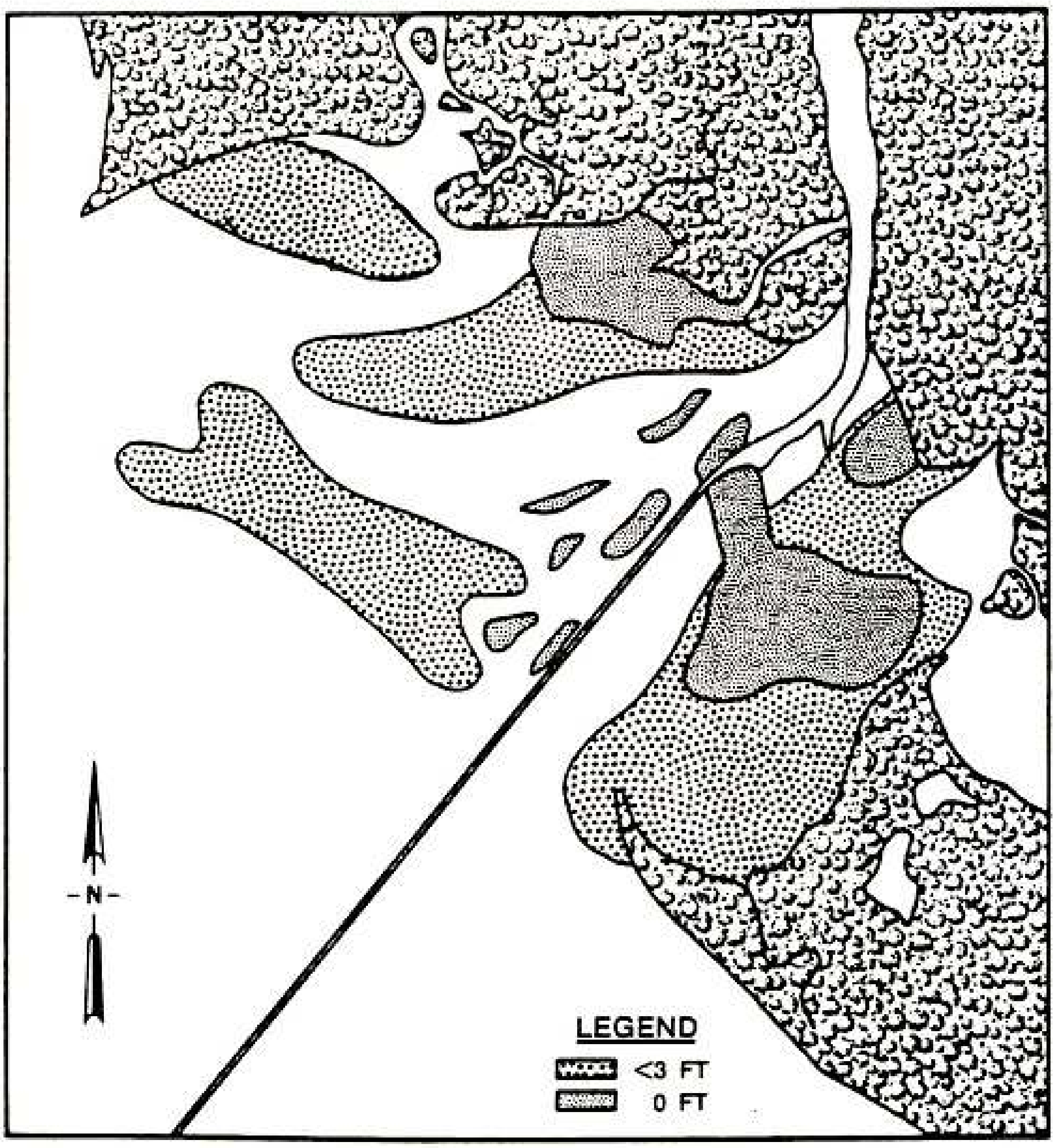


Figure 2.4. Quasi –2D calculated 50-year delta configuration (from Thomas et al., 1988).

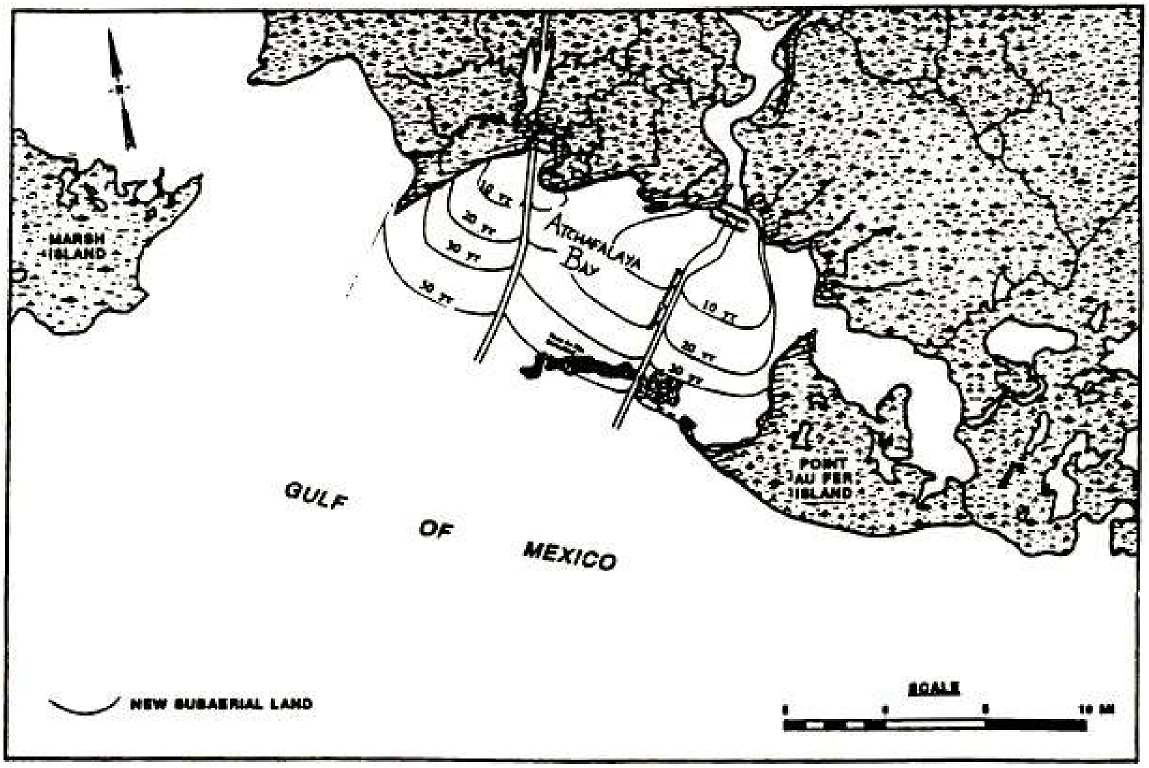


Figure 2.5. Predicted 50-year delta configuration by the analytical method (from Wang, 1985).

Corps of Engineers, New Orleans District (USACE, 1999). This study employed the HEC-6 model to simulate the expected water surface elevation or flowline of the river for project flood conditions. Since HEC-6 is a one-dimensional model, three separate reaches were created to model the branching system. HEC-6TR, a special version of a HEC-6 model, was used to investigate sediment transport capacity and delta growth. The HEC-6TR program was set up to simulate 50 years in the future; it was designed to predict the erosion and deposition in the river, as well as dredging in the navigation channel.

A spatial model with a coarse 1 km^2 FD 2D hydrodynamic scheme was developed in Louisiana State University to predict long-term sedimentation and habitat changes for the Atchafalaya Bay region (Martin, 2000). This model was capable of simulating the progradation of the River deltas, 50 to 70 years into the future. It was used as a tool to evaluate marsh management and delta development plans for the Atchafalaya River. However, this model was developed using a finite difference algorithm with a spatial cell resolution of 1 km^2. It was observed that the bottom topography of the bay, represented by 1 km^2 cells, removed all small-scale delta lobes and delivery channels. Therefore, a much finer resolution cell grid is necessary to accurately represent the delta in the model for engineering purposes.

2.3.7 Earlier Atchafalaya TABS-2 Model

A series of TABS-2 models have been developed for the Atchafalaya River by the Waterways Experiment Station for the New Orleans District (Donnell et al., 1991; USACE, 1999). Donnell et al. (1991) used both Cray-1 and Cyber 205 supercomputers for the earliest work, while a workstation cluster was employed for later work. A portion of the hydrograph was used to predict sediment transport potential. Later, that information was extrapolated to estimate the delta growth. Although the model predicted delta growth up to 50 years in the future (Figure 2.6), those predictions were based on extrapolation of short-term runs. The model was used to suggest rate and distribution of sedimentation.

Later, another TABS-2 model was developed to investigate delta growth and salinity in the bays and adjacent marshes (USACE, 1999). In the south, this model was extended well offshore into the Gulf of Mexico (Figure 2.7). The model contained about 48,000 nodes and had to describe the geometry of the model domain, which covered a much wider area than Atchafalaya Bay. Due to its high mesh density, the simulation took 1 day of CPU time for each 7 days of real time. As a result, only a portion, generally one week, of a flow hydrograph could be simulated in a single run. A copy of the model, developed by Waterways Experiment Station (WES) (USACE, 1999), was provided and studied. This model was reconfigured in a PC windows environment to review the processes included in the model, and to determine the zones most influenced by the Atchafalaya River, including an approximate alignment that could be used as the Gulf boundary for the Atchafalaya River model.

2.4 Rationale and Objective of this Research

Few studies have been conducted using a numerical engineering model to simulate bar formation and channel development in a real river. Most engineering research has narrowly focused on an assessed performance of alternative structure configuration. Engineers used one-dimensional models to determine the likelihood of

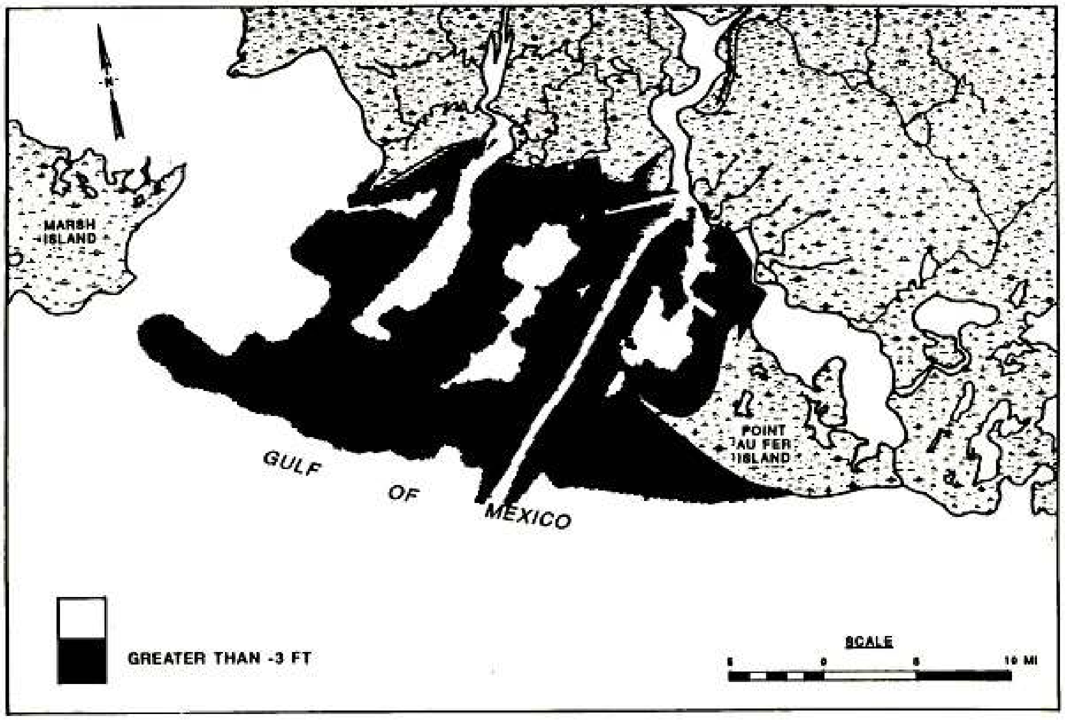


Figure 2.6. Predicted 50-year delta configuration by the TABS-2 method (from Donnell et al., 1991).

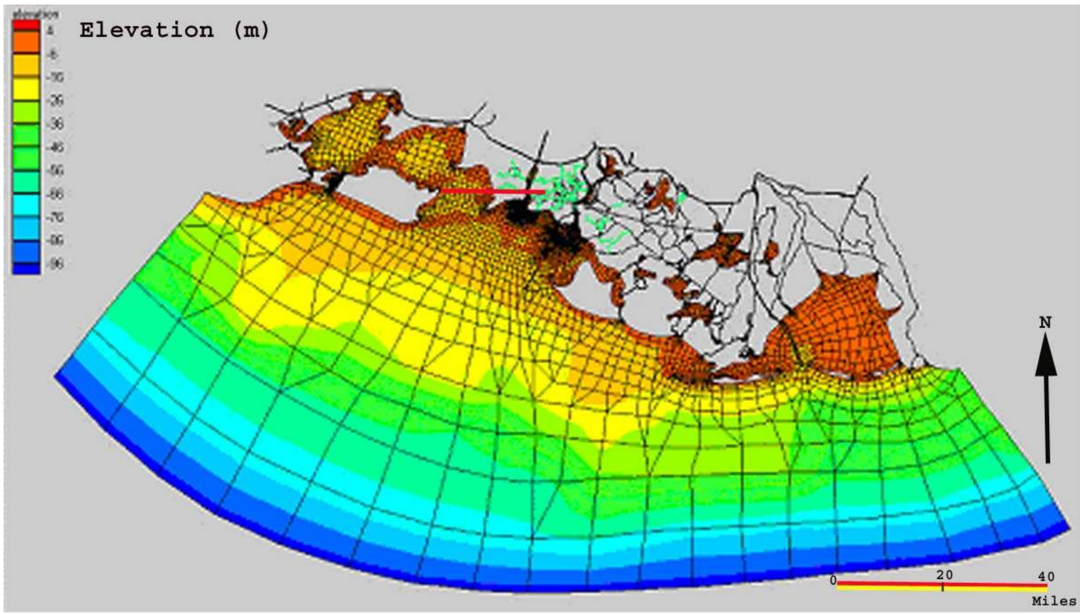


Figure 2.7. TABS-2 model domain developed to investigate the delta growth and the salinity of the Atchafalaya Bay (from USACE, 1999).

scour and deposition in rivers and streams with slowly changing cross-sections. Three-dimensional models have been applied in a few cases to explore sediment transport behavior in the immediate vicinity of structures on the scale of hours or days. On the other hand, field studies have demonstrated that that it takes months or years to develop a delta lobe or sand bar at a river mouth. Because of the practical restriction to short-range simulations, three-dimensional models are not ready for deltaic development. Earlier modeling experience, on the other hand, has shown that a two-dimensional sediment transport model could be used to perform continuous simulations extending for months in the prototype.

River mouth bar formations have traditionally been studied using geomorphologic conceptual models that do not explain bar growth quantitatively in terms of flow or velocity, but provide data that can be used to calibrate numerical models. It is difficult to predict time and space-dependent processes using geomorphological models. The approach taken here is to study the details of incipient delta formation and channel development using an engineering model. The higher level of detail that this must require has become possible as computers have become more powerful and computationally efficient; hydrodynamic and sediment transport models such as TABS-MD have become available. This study uses the knowledge gained from numerous geomorphological studies to calibrate a continuous simulation sediment transport model. Geomorphology indicates the spatial and temporal scale of features to be reproduced by the model. Nomographs developed from model runs can be generally applied by engineers toward the management and understanding of the delta.

Graphical representations of design parameters are common in engineering (Chow, 1964). The most widely used hydrologic relationship shown in a graph is the stage-discharge graph or rating curve (Singh, 1992). In sedimentation engineering, deposition or erosion is computed for specific project needs (Simons and Sentruk, 1992). When model simulations are made, sedimentation or erosion is reported in a tabular form (Stoschek et al., 2001). Some predictions on deposition are posited for management of a reservoir system (Chow, 1964). In sedimentation engineering, when a design question arises or a project has to be managed, a new model is developed and simulations are made.

Development of a working sediment transport model is often expensive and time consuming. Yet, for engineering applications, the concepts of sedimentation and erosion curve are relatively new. This research proposed that the rate of deposition or erosion by a river could be represented graphically by developing a set of curves. Later, these curves could be used for engineering purposes to estimate long-term deposition or erosion. Use of this technique will reduce the need for model simulations and would support planning for new projects.

CHAPTER 3. METHODOLOGY

3.1 Archived Data on the Atchafalaya River

An effort was made to collect all existing background data pertaining to the study area.

3.1.1 Bathymetry Data

The major source of recent bathymetry data was the 1998-1999 hydrographic survey of the Atchafalaya River System by the U.S. Army Corps of Engineers (USACE), New Orleans District (NOD). Original survey data was provided in the form of MicroStation design files in a CD ROM. Horizontal coordinates were in the state plane coordinate system, Louisiana South, NAD 1983. All elevations were expressed in feet National Geodetic Vertical Datum (NGVD) 1929. This survey also included rectified aerial photography, compiled shorelines, and cross-sections and over-bank transacts of the lower Atchafalaya River deltas (Figure 3.1). Depth configurations of the Atchafalaya Bay were available as well, from a 1994 terrain model developed at LSU (FitzGerald, 1998; Figure 3.2). Additional bottom elevation data were incorporated from the TABS model developed for the Atchafalaya River Reevaluation Study by the Waterways Experiment Stations (WES). More recent aerial photography was collected to incorporate the most recent dredging projects into the model. The latest information on the Atchafalaya River navigation channel was acquired from the survey performed by the New Orleans District (NOD) (Figure 3.3). The elevation information posted in these navigation data was in Low Mean Gulf datum and was modified to NADV 83 vertical datum.

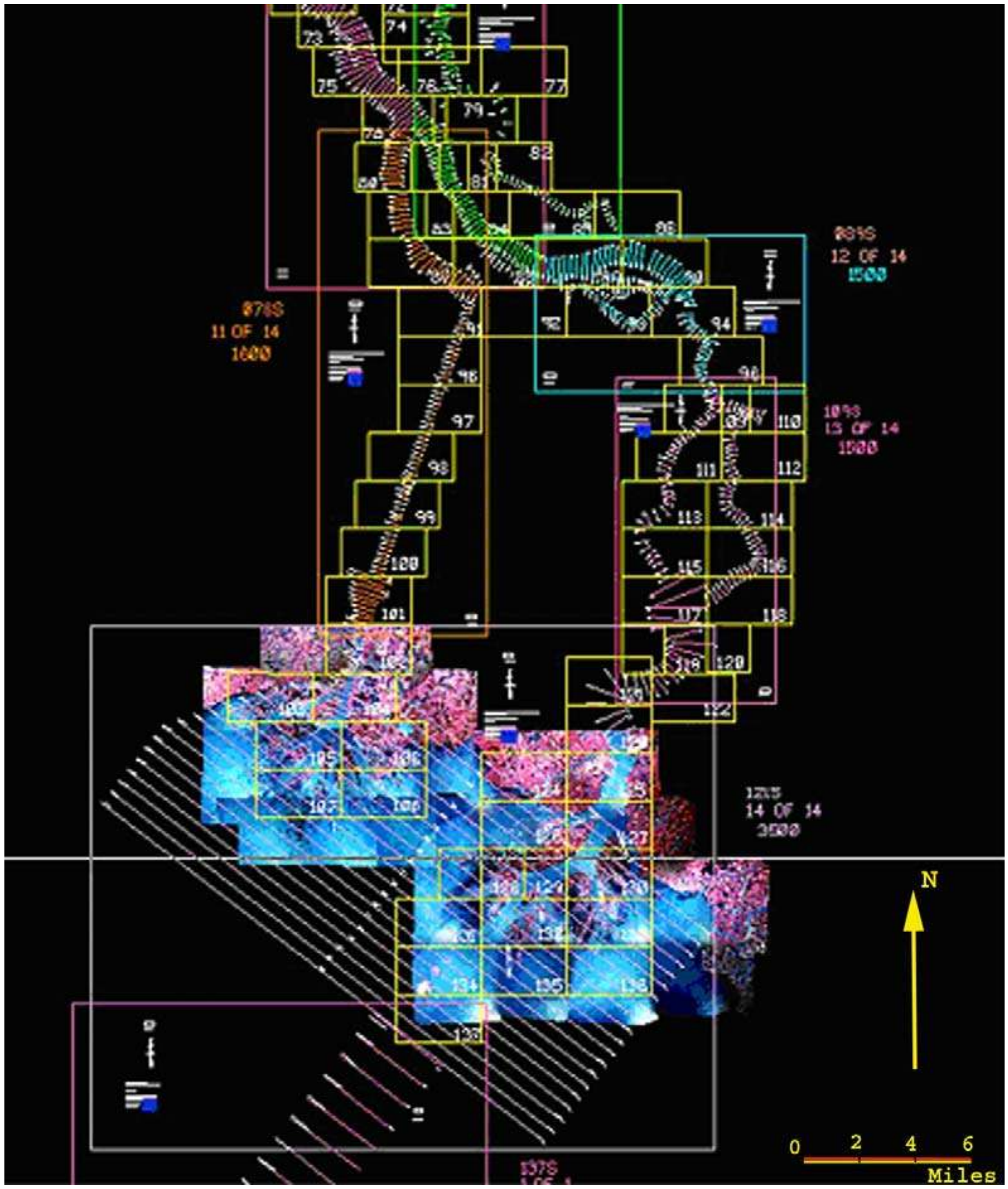


Figure 3.1. USACE 1998-99 Bathymetry survey locations.

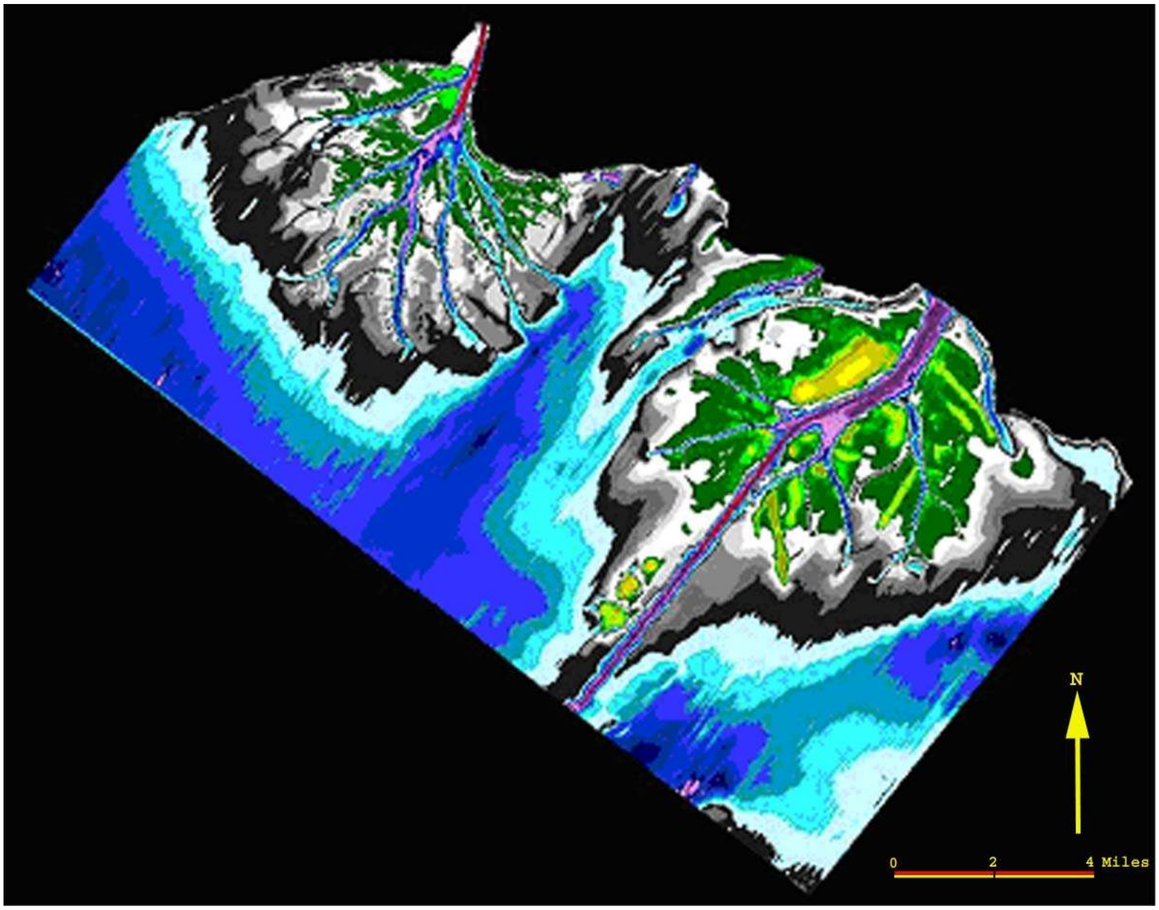


Figure 3.2. Digital Terrain model of the Atchafalaya delta (from FitzGerald, 1998).

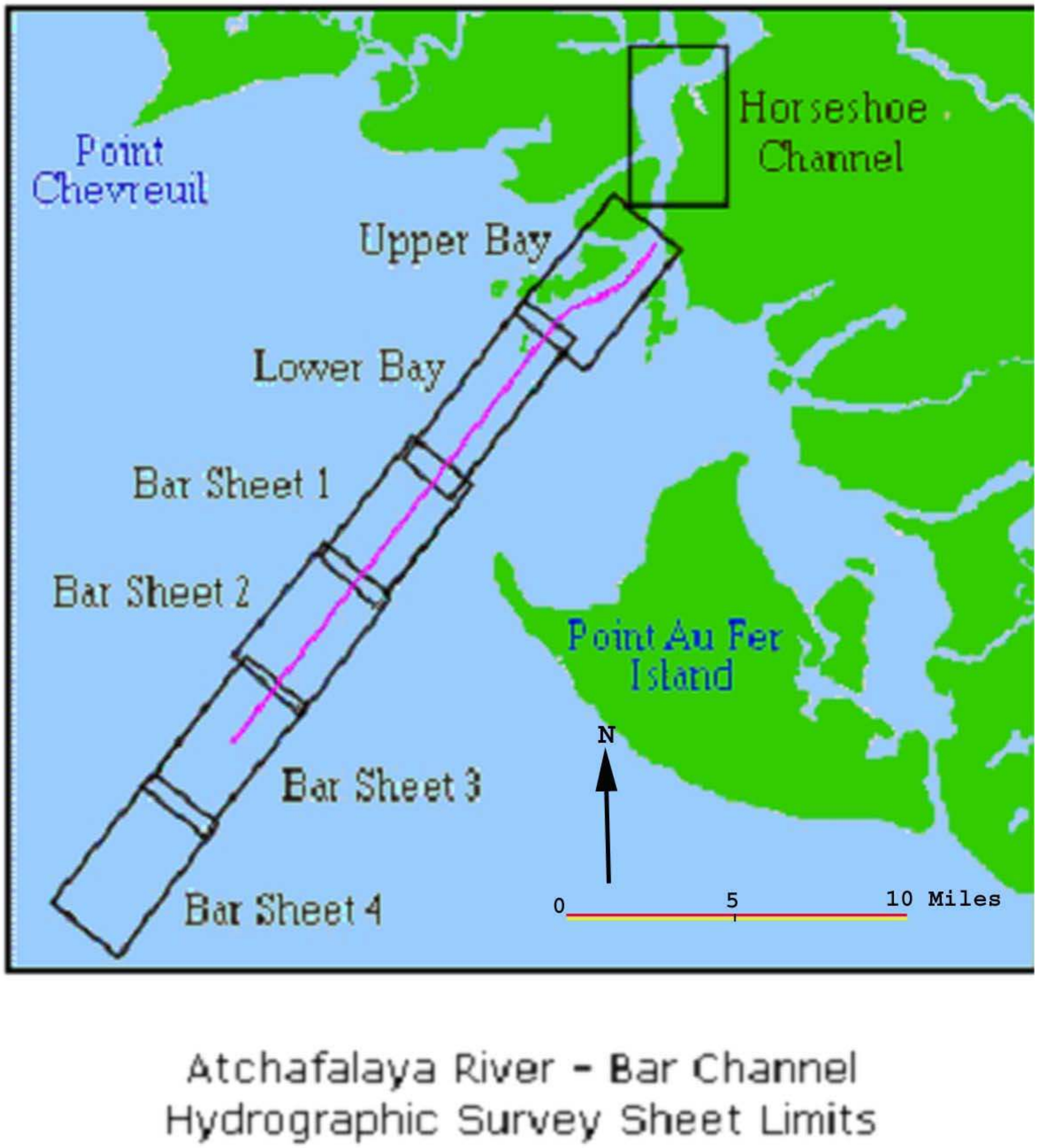


Figure 3.3. Locations of the USACE hydrographic survey for navigation (from http://www.mvn.usace.army.mil/ops/odt/atch.htm).

3.1.2 Hydrologic Data

Hydrologic and sediment data for the Mississippi Atchafalaya system were obtained from many sources. Periodic flow, stage and sediment data are available from the USACE from Tarbert Landing, MS, for the Mississippi River, and from Simmesport, Morgan City, and Calumet at Wax Lake Outlet for the Atchafalaya River. Stage records acquired at Eugene Island and the Amerada Hess location in Atchafalaya Bay, and at Luke's Landing in the East Cote Blanche Bay, are also available from the USACE (Table 3.1 and Figure 3.4).

Tarbert Landing (USACE ID 1100) is located in the Mississippi River near Mile 306.3 AHP (1962 Survey). Discharge and stage records, once intermittently located at this station since 1937. The Simmesport station is located near River Mile 4.9 (1963 survey) on the Atchafalaya River. Simmesport stage and discharge data, from 1935 to the present time, also are obtainable; bimonthly or monthly sediment data are collected since 1952. The Morgan City station is located near River Mile 117.7 (1963 survey). Available since 1989, stage, discharge, and monthly sediment concentrations are accessible at the Morgan City Wax Lake Outlet station, located at Calumet on the west side of east Calumet Floodgate.

Two tide stations record the tidal information in the Atchafalaya Bay. The tide station in the Atchafalaya Bay at Eugene Island (USACE ID 88600) is the most remote station in the bay, with an hourly tide record recorded every hour since 1973. The second tide station, located at the Atchafalaya Bay near Eugene Island (USACE ID 88550), is located approximately 5.5 miles northeast of that island, and also provides an hourly tide

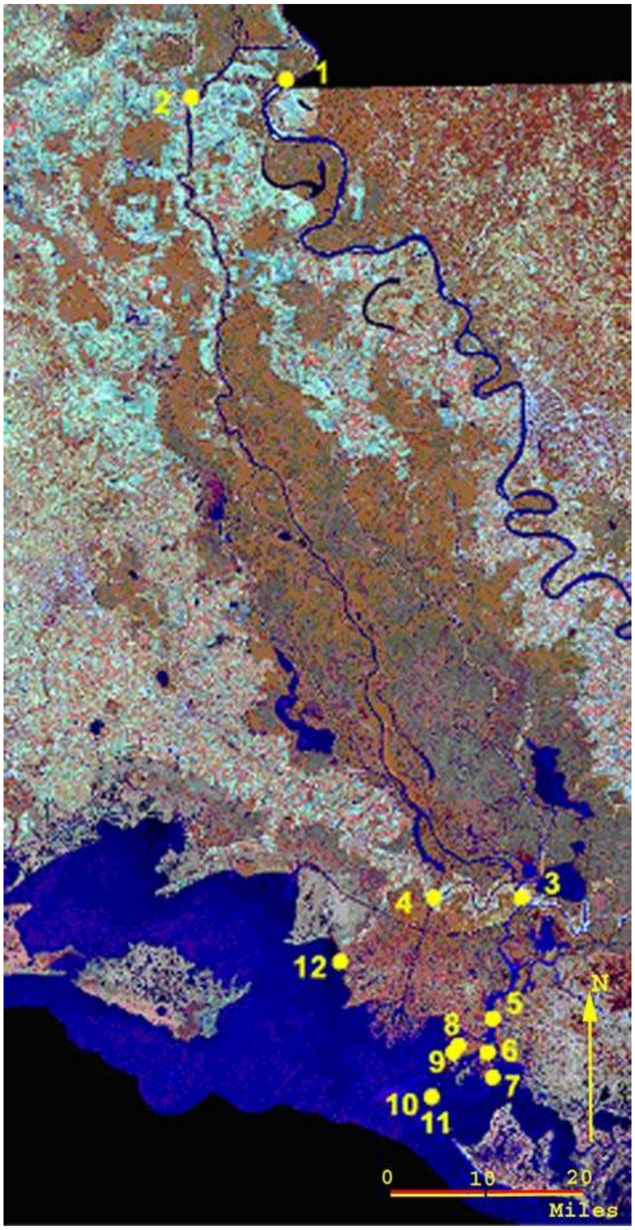


Figure 3.4. Locations of the data collection stations.

Table 3.1. Names and locations of the data collection stations

NO	Name	ID	Latitude	Longitude	Agency
1	Mississippi River at Tarbert Landing, MS	1100	31.008	-91.624	USACE, USGS
2	Atchafalaya River at Simmesport, LA	3045	30.983	-91.798	USACE, USGS
3	Lower Atchafalaya River at Morgan City, LA	3780	29.703	-91.202	USACE, USGS
4	Wax Lake Outlet at Calumet, LA	3720	29.698	-91.373	USACE, USGS
5	AT04-01	AT04-01	29.500	-91.250	LDNR
6	AT04-02	AT04-02	29.450	-91.280	LDNR
7	AT04-03	AT04-03	29.410	-91.270	LDNR
8	AT04-06	AT04-06	29.460	-91.330	LDNR
9	Atchafalaya Bay near Eugene Island, LA	88550	29.458	-91.341	USACE, USGS
10	AT04-04	AT04-04	29.380	-91.380	LDNR
11	Atchafalaya Bay at Eugene Island, LA	88600	29.379	-91.382	USACE, USGS
12	East Cote Blanche Bay at Luke's Landing, LA	88800	29.597	-91.543	USACE, USGS

record for every hour from 1976 to date. A third tide station, East Cote Blanche Bay at Luke's Landing, LA (USACE ID 88800), is located 2.8 miles north of the entrance to Bayou Sale. An hourly tide record also is available for every hour from 1957 to date.

3.1.3 Sediment Data

Suspended sediment data are available from 1952 for Simmesport, from 1980 for Morgan City and for the Wax Lake Outlet. In Simmesport, suspended sediment data are collected bimonthly by the USACE during non-flood years, and weekly during flood season. In Morgan City and Wax Lake Outlet, suspended sediment data are collected monthly during a non-flood year, and bimonthly during a flood year. During severe flood events, sediment data are collected more frequently by USACE and USGS.

3.2 Model Selection and Development

3.2.1 Model Selection

A variety of models with the capability to perform hydrodynamic and sediment transport simulations were reviewed to determine the most appropriate model for this study. Moffatt & Nichol Engineers (2000) performed a comparative analysis to identify the most appropriate engineering model to characterize the hydrodynamics and salinity of the Barataria Basin. All of these models were fairly well known in the hydraulic modeling community and have successful records of accomplishment on numerous applications throughout the world (Moffatt & Nichol Engineers, 2000). Moffatt & Nichol Engineers (2000) evaluated models for capabilities such as the following: node flexibility in describing the bathymetry, computational time, ease of set up and calibrate model, model acceptance and widespread use, the ability to include major forcing functions and the capability to simulate cohesive and non-cohesive sediment transport. Two 2D models, TABS-MD and MIKE21, were recommended as the most suitable models for simulating the complex Barataria system, which is comparable to the Atchafalaya in many respects.

Forcing functions significant to the Atchafalaya system are flow, sediment concentration, sediment size distribution, winds, tides, and waves (van Heerden, 1980 & 1983). Strong northerly and southerly winds substantially affect water levels and the movement of fine sediments in the shallow bay area during the winter and spring (van Heerden, 1980 & 1983). Tides play a major role in the circulation of water and fine sediments when river discharges are low in the summer and fall.

Extremely complex natural and man-made features of the Atchafalaya River and the delta challenge any numerical modeling effort. The prospective model may represent a continuous simulation model with the capability to include discharge, sediment load, high precision bathymetry, wind, tide, wave action and localized subsidence. River water in the Atchafalaya system follows a complicated path as it flows towards the Gulf of Mexico through channels, deltaic passes, sub-aerial and sub-aqueous levees, and open waters. Analyzing this complex system requires a model capable of addressing a wide range of flow conditions over a complex geometry of shallow water bodies, interspersed with intertidal land that alternates between wet and dry.

Delineation of geometry with an FD model requires very small cells, which would lead to a large number of computational domain. This could be accomplished through MIKE21, using a nested modeling technique, computationally as intensive as an FE model like TABS. Simulating a nested MIKE21 model would require more model computational time than a model grid without a nested model.

Moffatt and Nichols (2000) recommends TABS-MD over MIKE21. The industry standard engineering mathematical model TABS-MD (Thomas and McAnally, 1990) is used to explain the dominant processes that control flow, sediment transport, and delta growth in the Atchafalaya delta. The particular use of TABS-MD is due to the model's ability to run on a desktop with Surface-Water Modeling System (SMS) software (EMRL, 2002). The SMS software provides valuable tools for mesh generation, data interpolation, and graphical visualization. The SMS program was developed by Brigham Young University (BYU) in cooperation with USACE-WES. The TABS-MD suite includes separate hydrodynamic (RMA2) and sediment transport modules (SED2D).

3.2.2 The TABS-MD Model

The TABS-MD model, an extremely reliable engineering model, has been used extensively in the university research environment (Barrett, 1996; Freeman, 1992; Roig, 1994). Barrett (1996) used the TABS-MD model for wetland design. Freeman (1992) conducted a review of the model behavior in shallow water, and Roig (1994) used this tool for marsh and wetland modeling.

Three modules (GFGEN, RMA2 and SED2D-WES) of the TABS-MD will be used in this study. The module GFGEN will be used to create the finite element mesh of the study area; the module RMA2 will simulate hydrodynamic conditions of the study area; and SED2D-WES will compute sediment transport, scour, deposition and bed elevation changes within the study area.

The RMA2 program is a two-dimensional depth-averaged finite element hydrodynamic model that is two-dimensional in the horizontal plane. Like all vertically averaged schemes, it is not recommended where vortices, vibrations or vertical accelerations are of primary interest (Donnell et al., 2000). Vertically stratified flows are similarly beyond the capability of this model (Donnell et al., 2000). The TABS-MD model assumes the fluid is well mixed vertically with a hydrostatic pressure distribution; vertical acceleration is assumed negligible.

SED2D-WES can be applied to clay, silt, or sand bed sediments. In regard to suspended sediments, the model considers a single grain of any size may be introduced, but each studied size class must be simulated separately. SED2D does not compute velocities or water surface elevation, which is provided as an input from external calculations (RMA2). An implicit assumption of the SED2D is that the change in bed elevation during simulation does not change the flow field significantly. When bed change becomes significant, and this assumption does not hold, a new flow field must be generated. The Ackers-White formulation for sand transport was used for sediment transport capacity calculations, because it performed satisfactorily in tests of Mississippi/Atchafalaya sediments conducted by WES (Donnell et al., 1991) and others (White et al., 1975; Swart, 1976). In SED2D, clay transport and deposition are calculated by using equations of Krone (1962). The mathematics of RMA2 that is the essence of each component of TABS-MD is summarized from Donnell et al. (2000).

The RMA-2 hydrodynamic module solves the depth averaged two-dimensional equations of continuity and momentum transport (Donnell et al., 2000):

$$h\frac{\partial u}{\partial t}+hu\frac{\partial u}{\partial x}+hv\frac{\partial u}{\partial y}-\frac{h}{p}\left[E_{xx}\frac{\partial^2 u}{\partial x^2}+E_{xy}\frac{\partial^2 u}{\partial y^2}\right]+gh\left[\frac{\partial a}{\partial x}+\frac{\partial h}{\partial x}\right]+\frac{gun^2}{(1.486h^{1/6})^2}(u^2+v^2)^{1/2}$$
$$-\zeta V_a{}^2\cos\psi-2hv\omega\sin\Phi=0$$

---------------------- *Equation 1*

$$h\frac{\partial v}{\partial t}+hu\frac{\partial v}{\partial x}+hv\frac{\partial v}{\partial y}-\frac{h}{p}\left[E_{yx}\frac{\partial^2 v}{\partial x^2}+E_{yy}\frac{\partial^2 v}{\partial y^2}\right]+gh\left[\frac{\partial a}{\partial y}+\frac{\partial h}{\partial y}\right]+\frac{gvn^2}{(1.486h^{1/6})^2}(u^2+v^2)^{1/2}$$
$$-\zeta V_a{}^2\sin\psi+2hu\omega\sin\Phi=0$$

---------------------- *Equation 2*

$$\frac{\partial h}{\partial t}+h\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}\right)+u\frac{\partial u}{\partial x}+v\frac{\partial h}{\partial y}=0$$ ------------------------ *Equation 3*

where

h = Water depth

u,v = Velocities in the Cartesian directions

x,y,t = Cartesian coordinates and time

ρ = Density of fluid

E = Eddy viscosity coefficient,

for *xx* =normal direction on *x* axis surface

for *yy* =normal direction on *y* axis surface

for *xy and yx* = shear direction on each surface

g = Acceleration due to gravity

a = Elevation of bottom

n = Manning's roughness n-value

1.486 = Conversion from SI(metric) to non-SI units

ζ = Empirical wind shear coefficient

V_a = Wind Speed

ψ = Wind direction

ω = Rate of earth's angular rotation

Φ = Local latitude

Equations 1, 2 and 3 are solved by the finite element method using the Galerkin Method of weighted residuals. The elements may be one-dimensional quadrilaterals or triangles, and may have curved (parabolic) sides. The shape (or basis) functions are quadratic for velocity and linear for depth. Integration in space is performed by Gaussian integration (Donnell et al., 2000). Derivatives in time are replaced by a nonlinear finite difference approximation. Variables are assumed to vary over each time interval in the form

$$f(t) = f(t_0) + at + bt^c \qquad t_0 \le t < t_0 + \Delta t$$

------------ *Equation 4*

which is differentiated with respect to time, and cast in finite difference form. Letters *a, b* and *c* are constants.

At the end of simulation RMA2 produces water depth and velocity at each time within the solution domain. Water depths and velocity fields produced by the RMA2 are used by SED2D-WES to solve the two-dimensional advection-dispersion equation. The basic convection-diffusion equation is presented in Ariathurai et al. (1977) and Donnell (2000),

$$\frac{\partial C}{\partial t}+u\frac{\partial C}{\partial x}+v\frac{\partial C}{\partial y}=\frac{\partial}{\partial x}\left(D_x\frac{\partial C}{\partial x}\right)+\frac{\partial}{\partial y}\left(D_y\frac{\partial C}{\partial y}\right)+\alpha_1 C+\alpha_2$$

where

C = concentration, kg/m^3

t = time, sec

u = flow velocity in x-direction, m/sec

x = primary flow direction, m

v = flow velocity in y-direction, m/sec

y = direction perpendicular to x, m

D_x = effective diffusion coefficient in x-direction, m^2/sec

D_y = effective diffusion coefficient in y-direction, m^2/sec

α_1 = a coefficient for the source term, 1/sec

α_2 = the equilibrium concentration portion of the source term, kg/m^3/sec

For clay transport**,** deposition rates of clay beds are calculated with the equations of Krone (1962),

$$S=\begin{Bmatrix}\dfrac{-2V_s}{D}C\left(1-\dfrac{\tau}{\tau_d}\right) & for \quad C<C_c \\ \dfrac{-2V_k}{D}C^{5/3}\left(1-\dfrac{\tau}{\tau_d}\right) & for \quad C>C_c\end{Bmatrix}$$

where

τ = bed shear stress,

τ_d = critical shear stress for deposition,

C_c = critical concentration

3.2.3 Model Coefficients and Suggested Values

The key coefficients or parameters necessary to set up a TABS-MD model input file are the Manning's roughness (n) and eddy viscosity coefficients (Donnell et al., 1991; Moffatt & Nichol Engineers, 2000; Roig, 1994). Manning's roughness is the most commonly used parameter for calibration of the hydrodynamic model (Donnell et al., 2000).

Manning's roughness values, n, are expected to range from 0.020 to 0.035 for channels with sand beds (Chow, 1983). The value of n depends for the most part on water depth, vegetative cover and flow conditions. For a large alluvial river, Manning's values should change during a flood event (Simons and Sentruk, 1992). For an open and tidally influenced estuary, different researchers have used different roughness numbers. In Caminada Bay, Kjerfve (1973) used a Manning's value of 0.030, while Park (1998) used the value of 0.040 for a Barataria basin study. Using a much smaller model grid, Park (2002) found that for the same Barataria area, the value of 0.020 was more appropriate. In the Atchafalaya River and delta, a range of Manning values have been applied in earlier work. Donnell et al. (1991) used lower Manning values in the main deep channel and higher n values in the shallow bays. In the lower Mississippi River, hydrodynamic model values of the roughness varied from 0.015 to 0.020 in the main channel to 0.025 to 0.067 in the distributaries (USACE, 1990).

Turbulence is defined as the effect of temporal variation in velocity, and the momentum exchange associated with their special gradients (Donnell et al., 2000). Donnell et al. (2000) discusses this concept further below:

> Gelerkin methods of FE modeling, like some numerical model formulations, require the addition of a minimum level of artificial diffusion in order to obtain a 'stable' solution that converges in the Newton-Raphson iterative scheme. The Gelerkin method of weighted residuals used by RMA2 did not include any inherent form of stabilization other than the eddy viscosity term and requires a certain amount of added turbulence to achieve stability. However, if taken in excess, the velocity distributions could be smeared in space and time. It is difficult to establish a value for an eddy viscosity for the model being developed. Turbulence exchanges depend on the momentum of the fluid and the distance over which the momentum is applied.

Values for eddy viscosities were determined mainly from the literature and values used in the earlier Atchafalaya studies. Eddy viscosities were assigned to each element type and size. The eddy viscosity coefficient assigned to the Mississippi River was 100 lb-sec/ft^2 (4790 Pascal-sec) in USACE (1990).

Parameters needed to set up the SED2D sediment transport model were inflow sediment concentration, an initial concentration of suspended sediment in the river, the size of the suspended sediment, the size of the bed material, sediment fall velocity, grain size for roughness, diffusion coefficient, and critical shear stress for erosion and deposition (Donnell, 2000).

Four basic properties known to be important for sediment transport predictions are size, shape, specific gravity and fall velocity. The particle shape factor is 1.0 for a perfect sphere ranging to a low of 0.1 for a very irregularly shaped particle. Several recommended shape factor values can be found in the modeling literature. For natural sand, a shape factor of 0.7 could be employed (Yang, 1996).

Fall velocity, defined as the average terminal settling velocity of a particle falling alone in quiescent, distilled water of infinite extent (Simons and Sentruk, 1992), can be determined from formulas found in the literature. Rubey (1933) introduced a formula for computation of the fall velocity of gravel, sand and silt particles. For a given particle size, shape factor and temperature, the U.S. Interagency Committee on Water Resources, Subcommittee on Sedimentation (1957), provides a guide to the fall velocity characteristics of the sediment. A fall velocity curve was developed on this basis for various sizes of sediment (Figure 3.5 and Table 3.2).

A preliminary finite element mesh was developed, using the SMS 8.0 software package. A finite element mesh is defined as a network of triangular and quadrilateral elements constructed from nodes. The creation of a finite element mesh requires the user to provide bathymetric information and to define the study area extremities. The SMS software has the capability to import aerial photographs and satellite imagery as a back drop to delineate water and land features. In this study, the Map Module in SMS was used to define the study area boundaries and water features. Later, SMS automatically generated a mesh or grid network from the map module and then interpolated the bathymetry data onto the mesh.

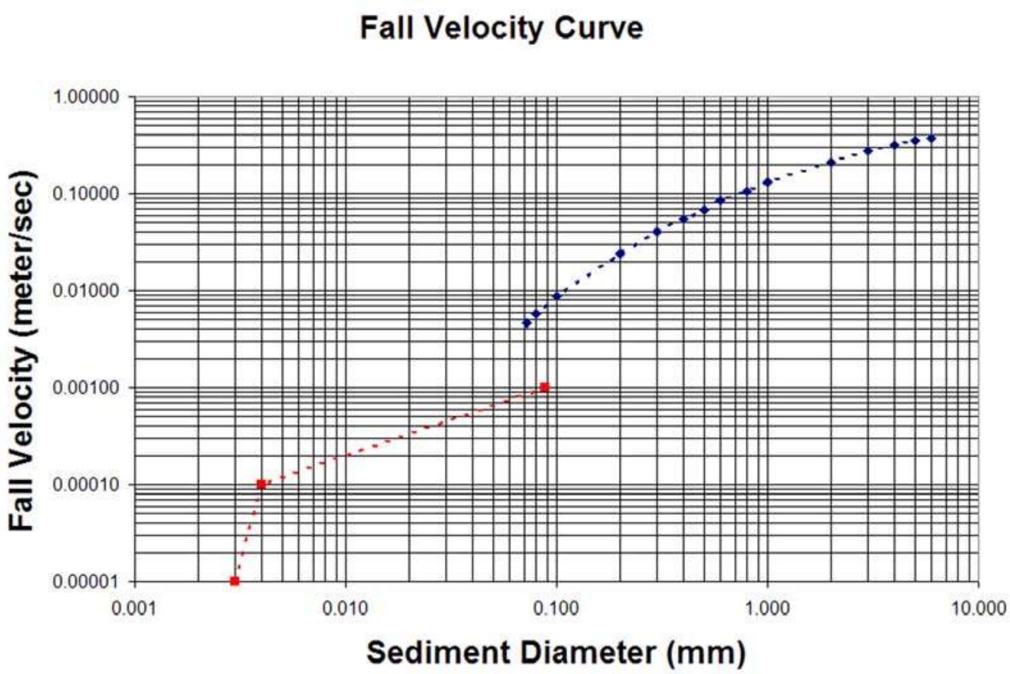


Figure 3.5. Fall velocity diagram. (Source: U.S. Interagency Committee on Water Resources, Subcommittee on Sedimentation, 1957 & Donnell et al., 1991)

Table 3.2. Sediment size and fall velocity. (Source: U.S. Interagency Committee on Water Resources, Subcommittee on Sedimentation, 1957 & Donnell et al., 1991)

Sediment Type	Grain Diameter>	<Grain Diameter	Avg Dia	Fall Velocity
	mm	mm	mm	m/s
Clay	0.0020	0.0040	0.0030	0.00002
Very Fine Silt	0.0040	0.0080	0.0060	0.00006
Fine Silt	0.0080	0.0160	0.0120	0.00019
Medium Silt	0.0160	0.0320	0.0240	0.00050
Coarse Silt	0.0320	0.0625	0.0473	0.00150
Very Fine Sand	0.0625	0.1250	0.0938	0.00350
Fine Sand	0.1250	0.2500	0.1875	0.00700
Medium Sand	0.2500	0.5000	0.3750	0.01700
Coarse Sand	0.5000	1.0000	0.7500	0.02800
Very Coarse Sand	1.0000	2.0000	1.5000	0.04200

In this study, the SMS map module was used to import a satellite image acquired in 2000 to delineate model boundary and major water features. Detailed features of the Atchafalaya delta were delineated from 1 m resolution color infra-red aerial photographs collected by the USGS in 1998. Alignments of the navigation channel and distributary channels were determined using a bathymetric survey performed by the U.S. Army Corps of Engineers. The model developed has a high density of computational nodes in the Main River Channel and in the deltaic passes, with relatively low resolution in the open bays. Key features included in the model are the Atchafalaya River Main Channel, Bayou Shaffer, the Gulf Intracoastal Water Way (GIWW), Bayou Avoca, Bayou Chene, Shell Islands Pass and Atchafalaya Bay (Figure 3.6). The built-in interpolate command in the mesh creator module of SMS was used to assign a depth for each individual node. Later, hand-editing was done to fine-tune the depth information in the model as additional data became available from the field.

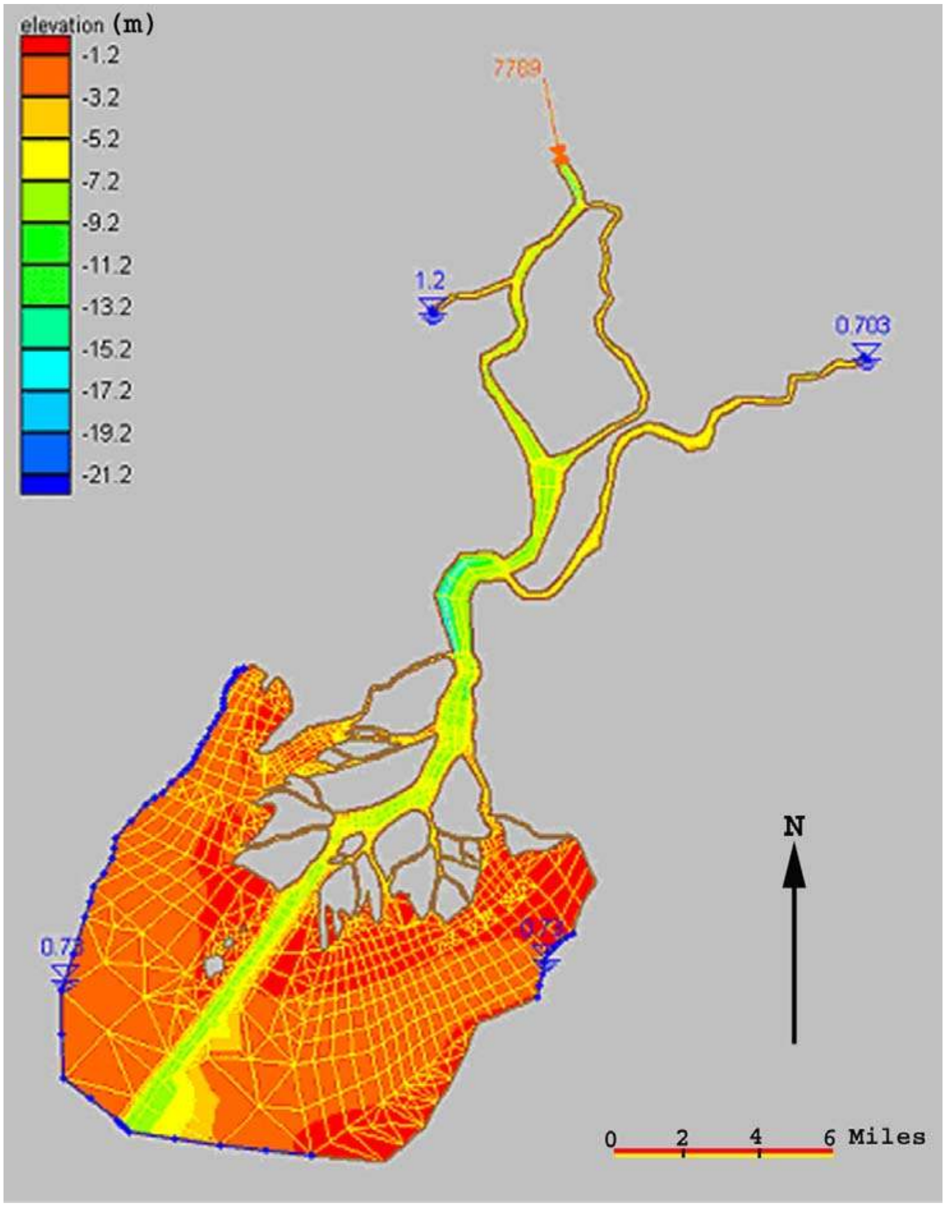


Figure 3.6. Prototype model boundary.

CHAPTER 4. RESULTS AND DISCUSSION

4.1 Field Program

4.1.1 Discussion and Analysis of the Archived Data

Historical daily Simmesport discharge data (1935-2002) showed that the minimum and maximum observed discharge at Simmesport was 311 cms (11,000 cfs) and 22,112 cms (781,000 cfs), respectively (Figure 4.1). The arithmetic average of the observed discharge was 5,776 cms (204,000 cfs). A minimum discharge of 311 cms (11,000 cfs) was recorded on 24 June 1964, while a maximum discharge of 22,112 cms (781,000 cfs) was recorded on 12 May 1973. The average monthly discharge suggested that flow through the Atchafalaya River is distinctly seasonal (Figure 4.2). High flow in the Atchafalaya River generally occurs during winter or spring (December through May) while low flow usually takes place during summer and fall (June through November).

Significant deviation from the average hydrograph may be observed during any flood year. Major floods are defined as high water events that exceed bank-full level (Louisiana Hydroelectric Limited, 1999). The flood of 1927 peaked in the third week of February, while that of 1993 lasted into July and August. A total of twenty-two major floods have been observed to date (Table 4.1) (Louisiana Hydroelectric Limited, 1999). Generally, a major flood can be expected every 7 to 10 years; however, floods have frequently occurred in consecutive years (1912 and 1913; 1943, 1944 and 1945; 1973, 1974, and 1975; 1983 and 1984).

Daily Atchafalaya River discharge (1935-2002) at Simmesport was analyzed and a percent probability curve (Figure 4.3) was developed.

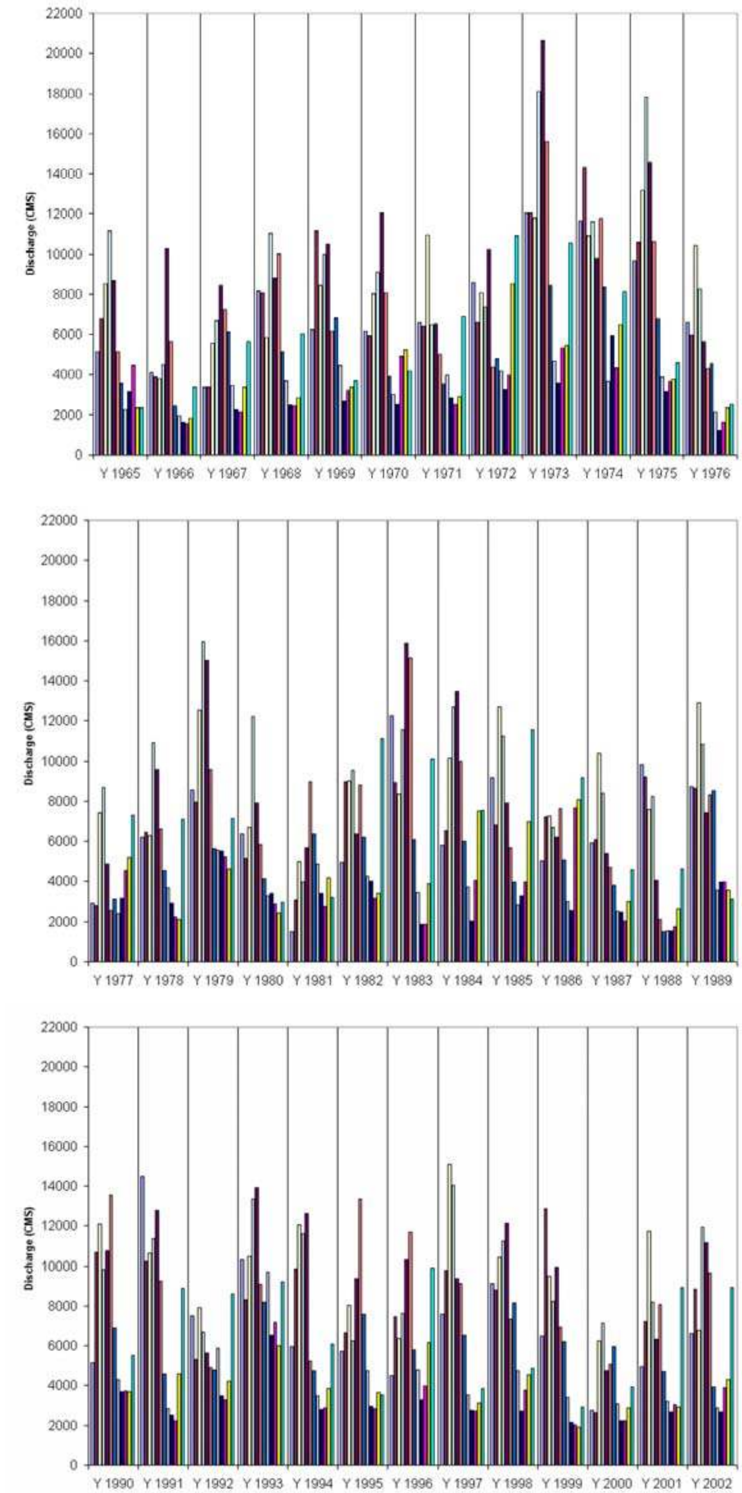


Figure 4.1. Average monthly flow at Simmesport.

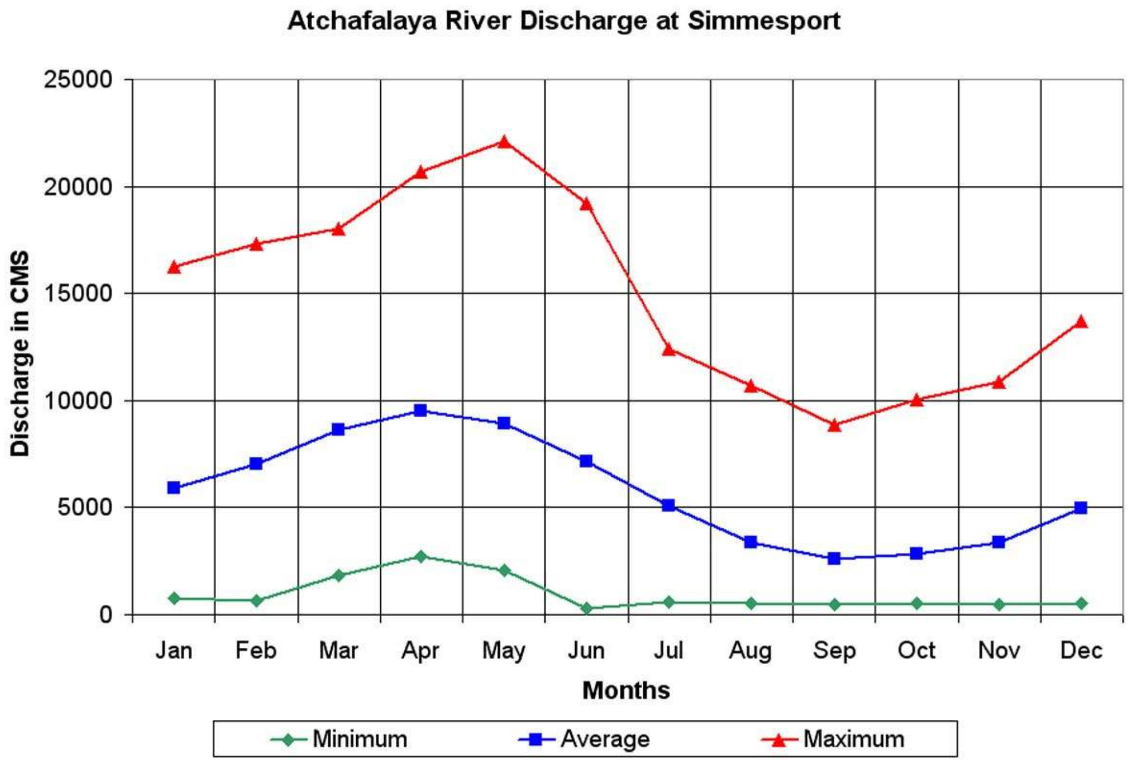


Figure 4.2. Daily discharge averaged over months at Simmesport (1935-2002).

Table 4.1. Historical Floods in the Mississippi River.

		Vicksburg, Mississippi	Days	Morgan City	Wax Lake Outlet
Rank	Year	Discharge (CMS)	Over Bank	Discharge (CMS)	Discharge (CMS)
1	1927	64490**	185		
2	1937	58319	43		
3	**1973**	55544	89	19,591	7,700
4	1945	54412	47		
5	1950	53110	29		
6	**1975**	51864	32	14,466	5,294
7	**1983**	50647	50	11,635	6,172
8	1913	50477	42		
9	1912	50392	72		
10	1897	50307	75		
11	**1997**	50279	NA		
12	1922	49599	70		
13	1929	49288	106		
14	1916	49118	90		
15	1907	48722	73		
16	**1979**	47957	53	12,456	5,973
17	**1991**	47844	14	8,323	4,162
18	1943	47306	9		
19	1920	46683	78		
20	1944	45551	3		
21	1903	45466	82		
22	**1984**	45296	24	8,663	5,436
** This is an estimated peak discharge had levees held.					

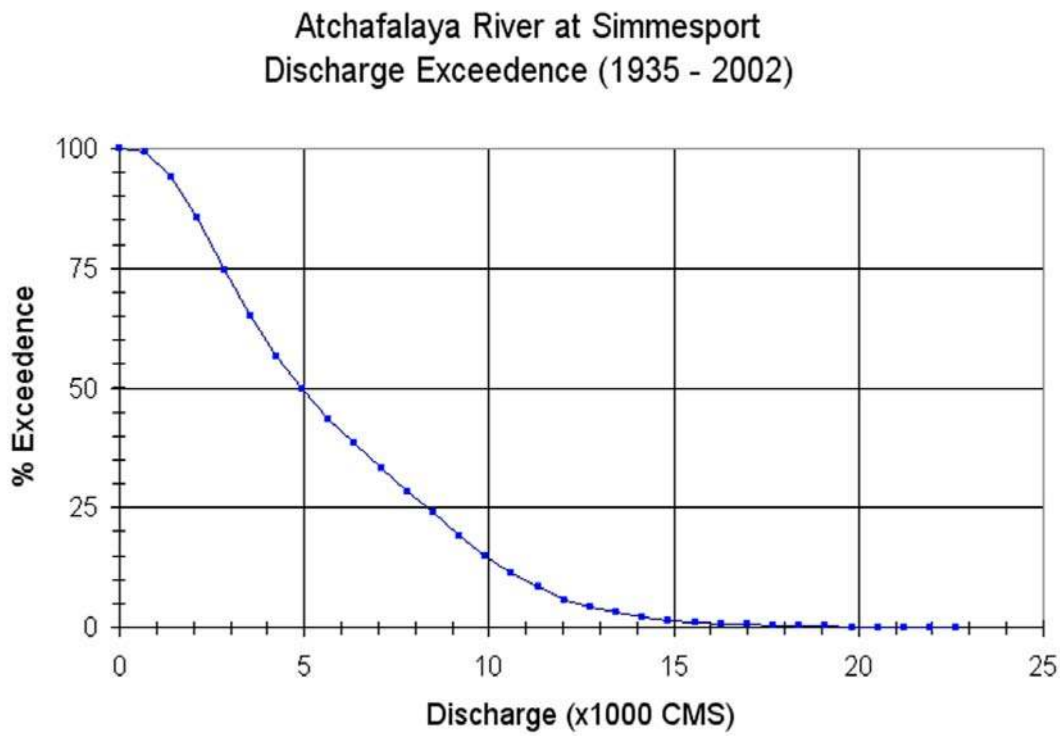


Figure 4.3. Discharge exceedence curve for the Simmesport stations.

This covers pre-control (1935-63) and post-control periods (1964-2002). Discharge hydrographs of major floods vary widely in magnitude and duration.

It was important to classify floods with respect to magnitude and duration to determine which had the capability to carry sediments to the delta. A flow of 11,325 cms (400,000 cfs) at Simmesport was assumed to initiate the flood in the Atchafalaya River. During the 1973 flood, the highest flow documented at Morgan City was on May 29, 1973 but discharge was higher than 11,325 cms for 112 days (3.75 months), and peak discharge was observed 57 days after the beginning of the flood. The most recent significant flood occurred in 1997. The 1997 flood took 23 days to reach peak and 26 days to fall below 11,325 cms. The total duration of flood flow defined in this way was 49 days or 1.6 months. Historical peak flows at Morgan City were compiled from several sources (Louisiana Hydroelectric Limited, 1999; Kemp et al., 1995). These discharges were used in the model to test the sand-carrying capacity of the Atchafalaya River.

The yearly volume of water flowing through Simmesport was calculated (Figure 4.4) from the daily observed flow data. Daily flow volume was calculated using the formula:

Volume = flow rate * duration of flow

Yearly volume was computed by summing the daily flow for each year.

Boundary Gulf stages have a strong, long-term stage hydrograph with seasonal signatures (Figure 4.5). High Gulf elevation is seen during April, May, and June with a relatively low water level during November and December. The concentration of sand entering the Atchafalaya also has a seasonal variation. Like the flow hydrograph,

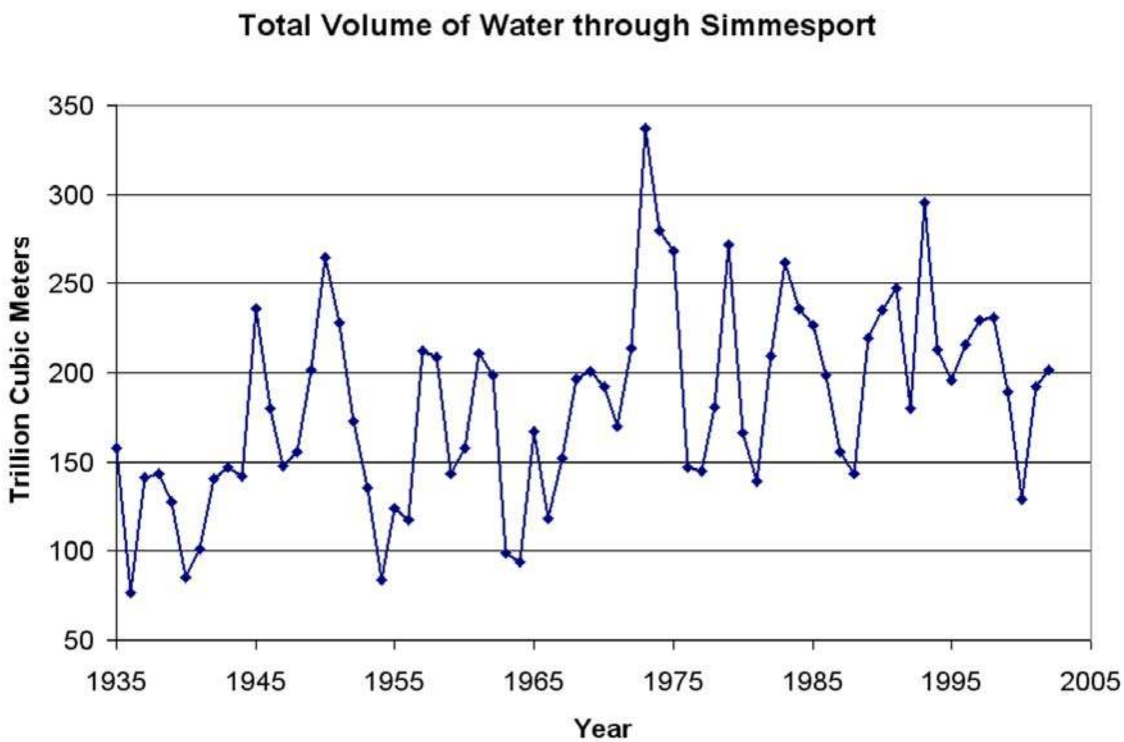


Figure 4.4. Yearly volume of water flowing through Simmesport (1935-2002).

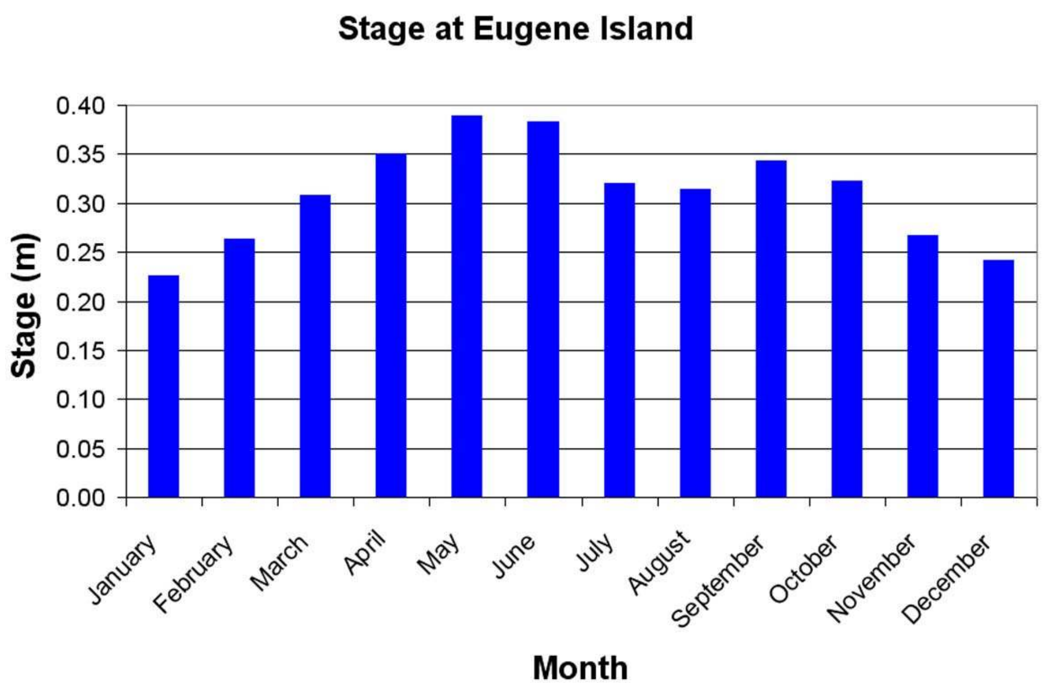


Figure 4.5. Stage hydrograph at the Eugene Island in the Atchafalaya Bay (1987-2001).

suspended sand was available at Simmesport during February, March, and April, while little sand generally was suspended during the August through November period (Figure 4.6). A similar seasonality has been observed at the Morgan City station (Figure 4.7).

A sediment load budget was developed using the formula:

Sediment load = Average sediment load of two consecutive measurements * Time elapsed between those two measurements

A sediment load budget was developed at Simmesport (1952-1996); it was found that the average total sediment flux was 104 million tons per year. The fine sediment passing through Simmesport accounted for 77 percent of the total load, while coarse sediment was 23 percent. Average fine material through Simmesport reached 80.6 million tons per year, while coarse sediment was 23.5 million tons per year. Similar calculations for Morgan City (1980-1996) showed that 86% of the sediment material was fine, and 12% of sediment material was coarse; at Wax Lake Outlet (1980-1996), 87% was fine and 13% was coarse. This data suggested that roughly 10% of Simmesport sand is deposited before it reaches the Morgan City or Wax Lake Outlet stations.

Peak discharge and peak sand concentrations did not occur simultaneously. Suspended sand concentration rose to a maximum many days before the peak water discharge (Figure 4.8). The time lag between peak sand sediment concentration and peak discharge varied from year to year, averaging 27 days (Figure 4.9). This phenomenon was captured at Simmesport, but not at Morgan City and Calumet, presumably because of the more frequent sampling at Simmesport. At the stations in Morgan City and Wax

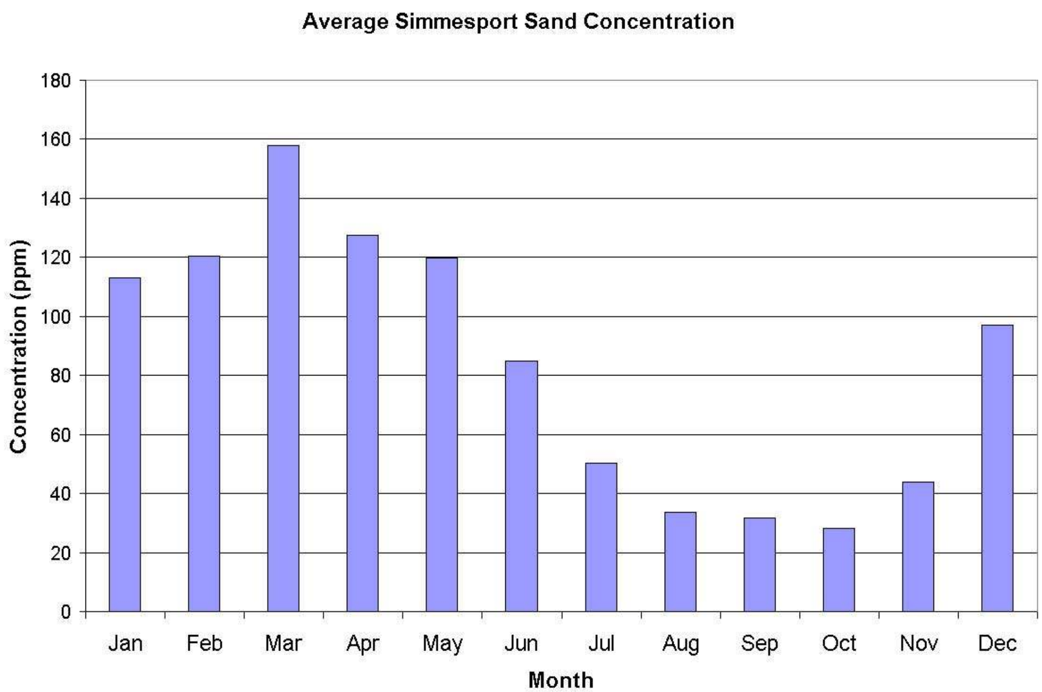


Figure 4.6. Average monthly sand concentration at Simmesport (1952-1996).

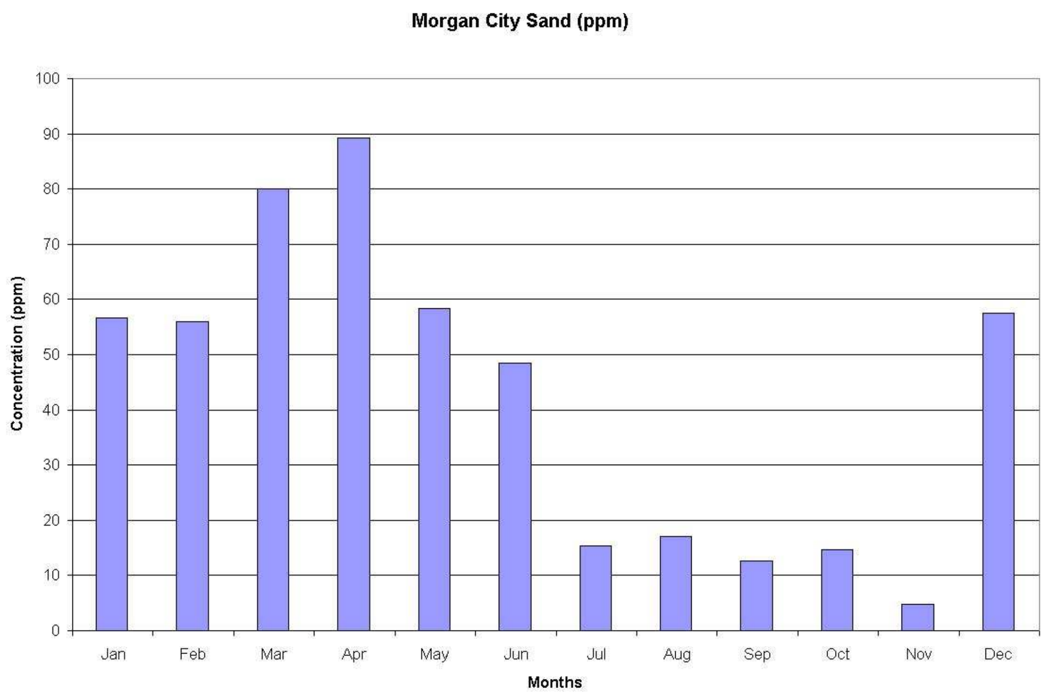


Figure 4.7. Average monthly sand concentration at Morgan City (1980-1996).

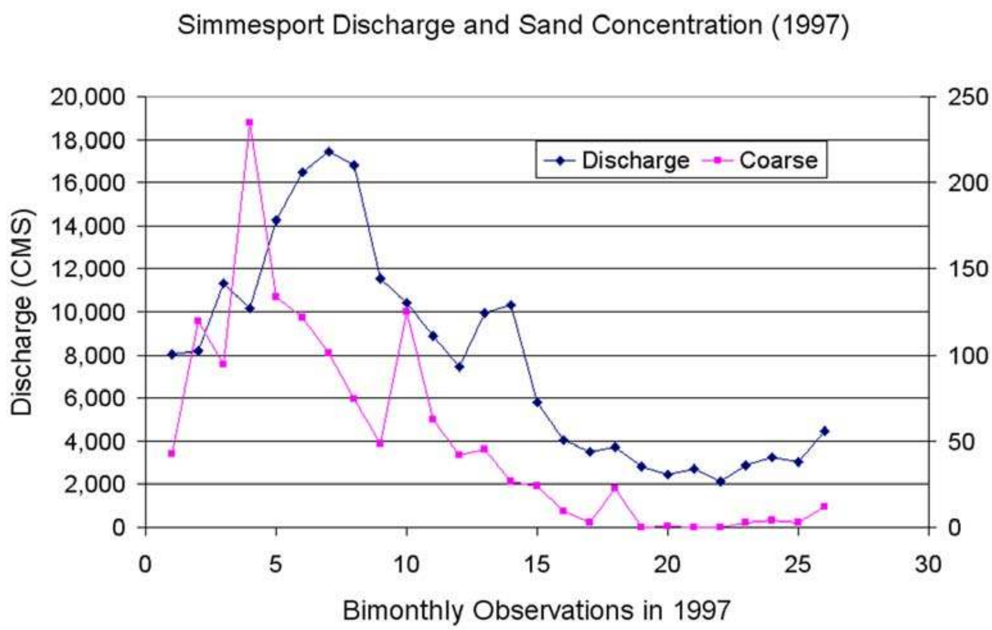


Figure 4.8. Hydrograph of flow and sediment concentration at Simmesport observed during 1997.

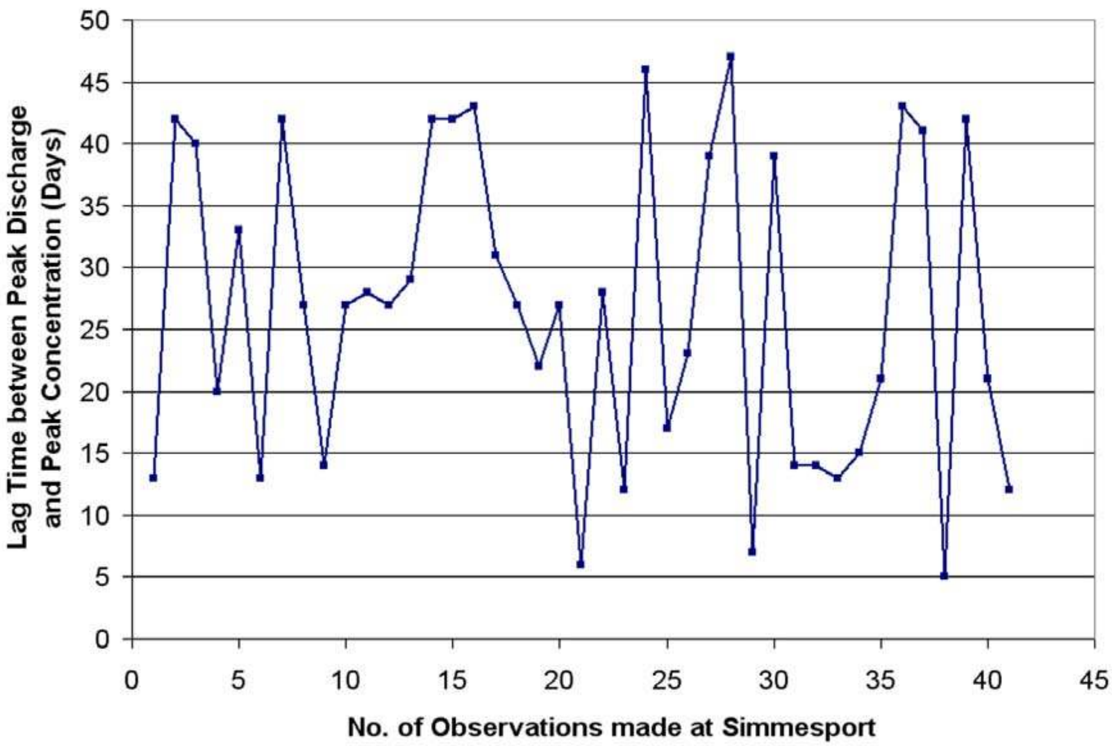


Figure 4.9. Plot of lag time observed between peak discharge and peak coarse sediment concentration at Simmesport (1952-1996).

Lake Outlet, peak sediment and flow appear to be concurrent, because sediment data is collected only once a month.

A preliminary analysis of the existing data and the station locations indicated additional data would be necessary for model calibration and validation. There were only two long-term tidal stations in the model domain. The remote one, at Eugene Island, was at the outer edge of the navigation channel, but only daily 8 a.m. data are available at this station. The tide observed at this station could be used as the model's base level stage boundary. On the other hand, the Amerada Hess gage is located inside of the delta itself. Tides observed at the Amerada Hess can be used for calibration because an hourly record is available. More hourly or higher resolution tide data were needed, however.

Detailed sediment data were necessary to develop and calibrate the sediment transport model. Therefore, a field data collection program was initiated to collect detailed bathymetry, flow, tide and bottom sediment information.

4.1.2 New Tide Station Setup

Information was unavailable to determine the effects of small delta and distributary channels on tide, lag time-to-peak tide, and the effects of tide on sediment transport. Five new tide stations were established by Louisiana Department of Natural Resources (LDNR) around the delta in 2001 to collect water surface elevation data (Figure 4.10). A field survey program was initiated to collect bathymetry flow, velocity, suspended and bottom sediment sample data (Figure 4.11).

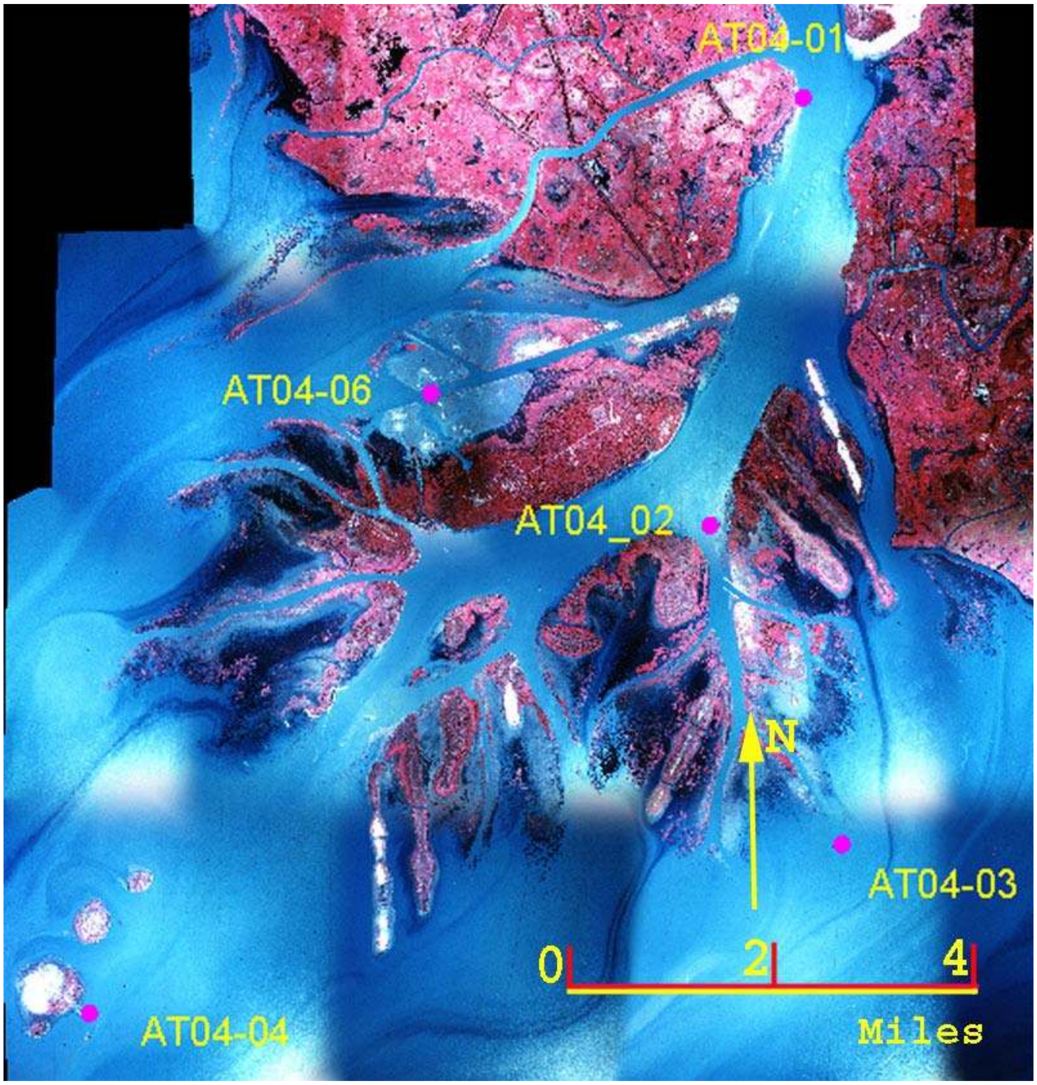


Figure 4.10. Locations of new tide stations in the Atchafalaya delta.

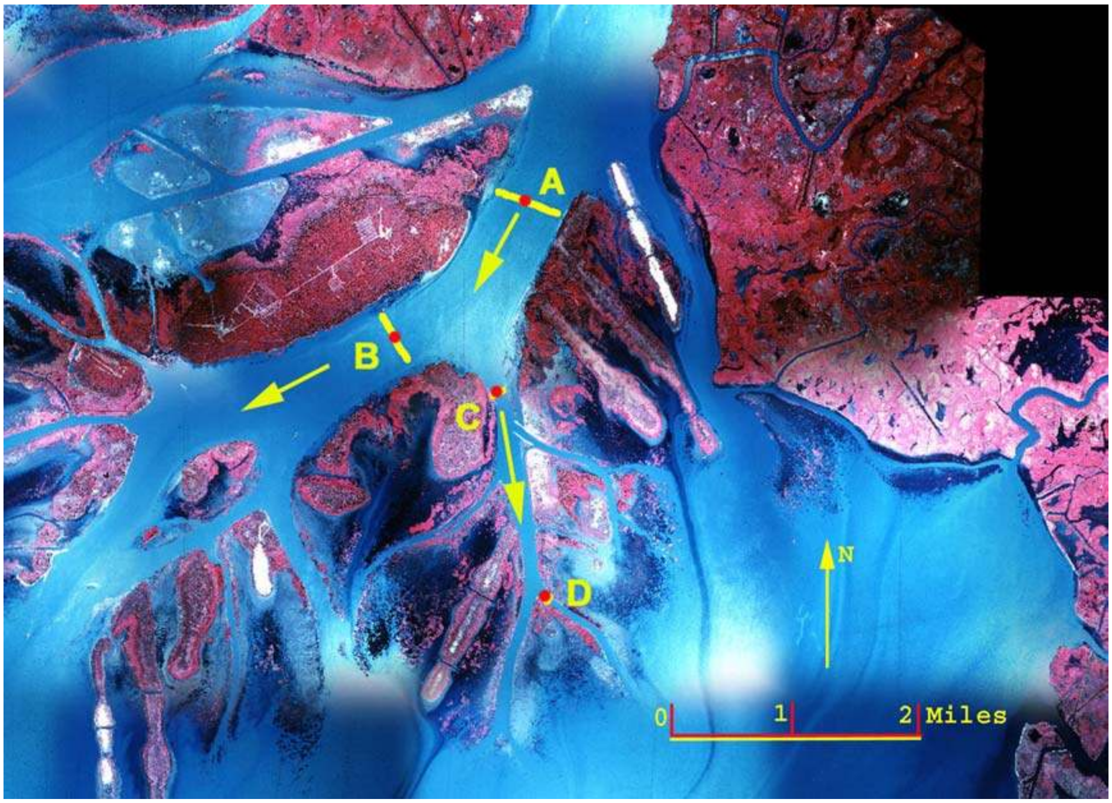


Figure 4.11. Locations of the sediment and velocity sample stations in the Atchafalaya delta.

The location of each tide station was selected such that the Atchafalaya River main channel and the Atchafalaya delta were covered sufficiently to capture the tidal fluctuations with high degree of accuracy. Stations 1, 2 and 4 were located along the main channel. Station 3 was located at the end of the Castille Pass, where the channel shoaled as it entered the distributary channel and the bay. Station 6 was established to the west of the main channel at the head of Breaux Pass. All newly established stations were set to collect data at 30-minute intervals.

4.1.3 Bathymetry Data Collection

The LSU team conducted several bathymetry surveys during June, July, August and October of 2002 to investigate detailed channel and bar morphology. A bathymetry survey was conducted using a GPS equipped echo sounder (Hydrotrac). The depth of water was later corrected to the NAVD88 datum, using the tide gage data. Four separate field trips were made to the eastern side of the Atchafalaya delta to collect bathymetry, suspended sediment, and bottom bed material samples. Most of the survey and sediment data collection effort was focused in the East and Ratcliff Pass distributaries to obtain information in the most active portion of the delta (Figure 4.12).

The first survey was conducted during June 18-19, 2002. One bathymetry line was completed during the June 18 survey. The survey started at 1539 CST and was completed by 1719 CST. The time of survey was carefully noted on the survey notebook and recorded by the Fathometer. The noted time was used to correct water depth for tidal correction and adjusted with a known datum. On June 19, 2002, four survey sets were

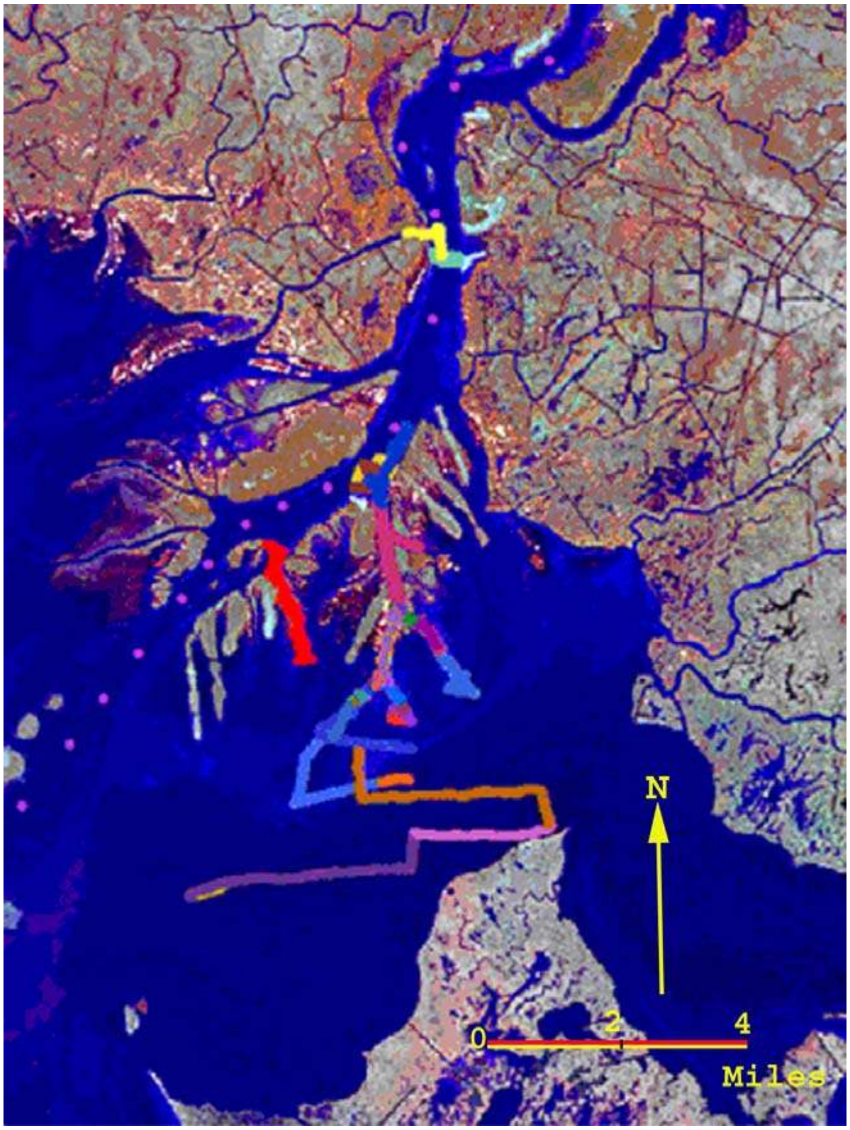


Figure 4.12. Bathymetry survey locations in the Atchafalaya delta.

completed for bottom elevation. The first survey line was initiated on 1018 CST, the second began on 1033 CST, the third on 1630 CST, and the fourth one around 1650 CST.

The second field trip was conducted during July 1-2, 2002. On July 1, three sets of survey lines were completed. The third was conducted during July 23-24, with four sets of bottom survey data lines completed. The fourth survey was completed in August 2002. During the survey trips, bottom sediment and suspended sediments were also collected.

Bathymetry data collected from these surveys then were merged with archived data to form a single point data file in NAVD88 XYZ format (Figure 4.13). Later this file was used in the SMS package to assign depths to each computational node in the model.

4.1.4 Calibration Data Collection

Brown Cunningham Gannuch Inc. (2002) conducted a separate, pseudo-synoptic field survey to collect discharge information from the main river channel, and from the first and second order deltaic passes. This approach was taken to understand how the distribution of discharge and sediment transport among the small channels varied with total discharge measured in the Atchafalaya River. This was accomplished by comparing the spatial variation of the data and relative difference of the discharge. Locations of pseudo-synoptic surveys are shown in Figure 4.11. Cross-sections A-B are in the main channel. Cross-section C is in the first order channel at the East Pass. Cross-section D is located at the second order channel at the Castille Pass. During each synoptic survey, the flow, velocity and suspended sediment data were collected at pre-selected stations. Channel bottom elevation, depth, stage and cross-section area were measured.

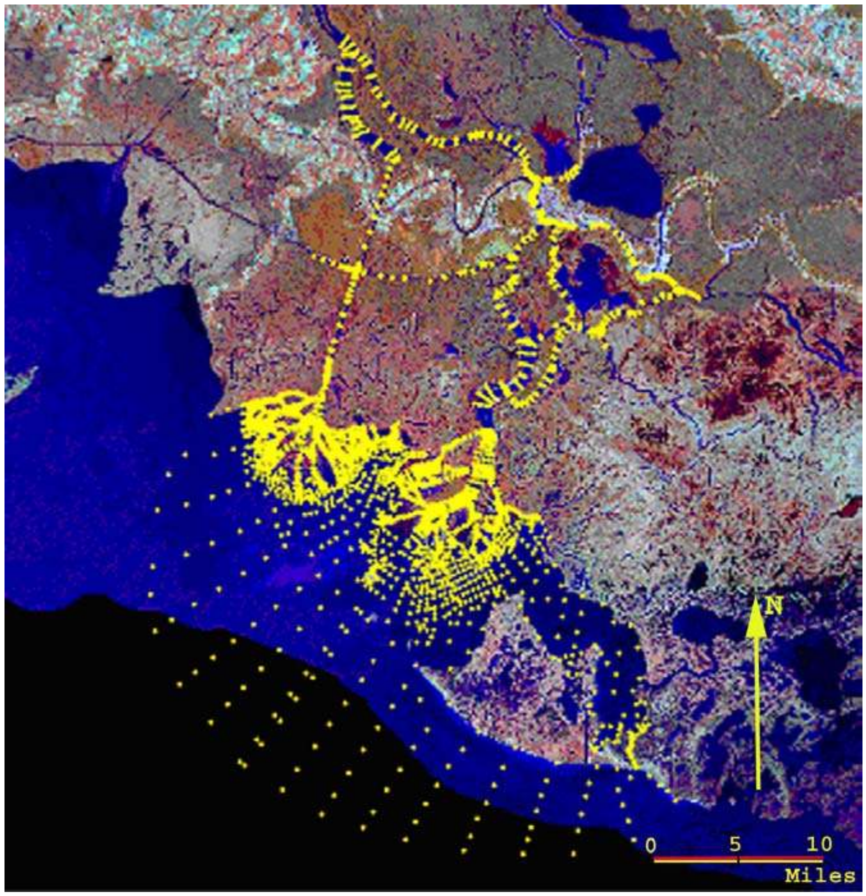


Figure 4.13. Point data in the XYZ format for the Atchafalaya River and delta.

A field survey was made on May 23, 2002, to collect data for the model calibration. During this survey, water surface elevation, distribution of flow through the Atchafalaya delta, average cross-section velocity, suspended and bed samples were collected. Discharge and sediment load at Morgan City were measured by the USGS.

Velocities were measured at 80, 60, 40 and 20% of the depth of water where four measurements were taken, with 80, 60, 20% of the depths measured in the smaller ones. Where two measurements were taken, velocities were measured at 80 and 20% of the depths. The available depth of the channel determined the number of the vertical observations made at each measurement. Some variations were observed in the velocity of depth and in distance from the center of the channel (Figure 4.14). An average of all readings were performed to determine the average velocity of the cross-section.

At each cross-section, suspended sediment and the bottom or bed sediments were collected. From the suspended sediment samples, concentration and percent coarse (sand) and fine (silt and clay) sediments were determined by weight after drying. Tests were done to determine the grain-size distribution of the suspended sediments and bed sediments for each station data taken.

Suspended sediment sampling was conducted at four locations during the May 23, 2002 survey. Several samples were taken from each cross-section at different depths of a vertical location. Collected sediment samples were sent to the laboratory to determine the total sediment concentration, concentration of the coarse and fine sediments, and grain size distribution of the sediment samples (Brown Cunningham Gannuch Inc., 2002). Concentration of suspended sediment and the percent sand was variable even within a

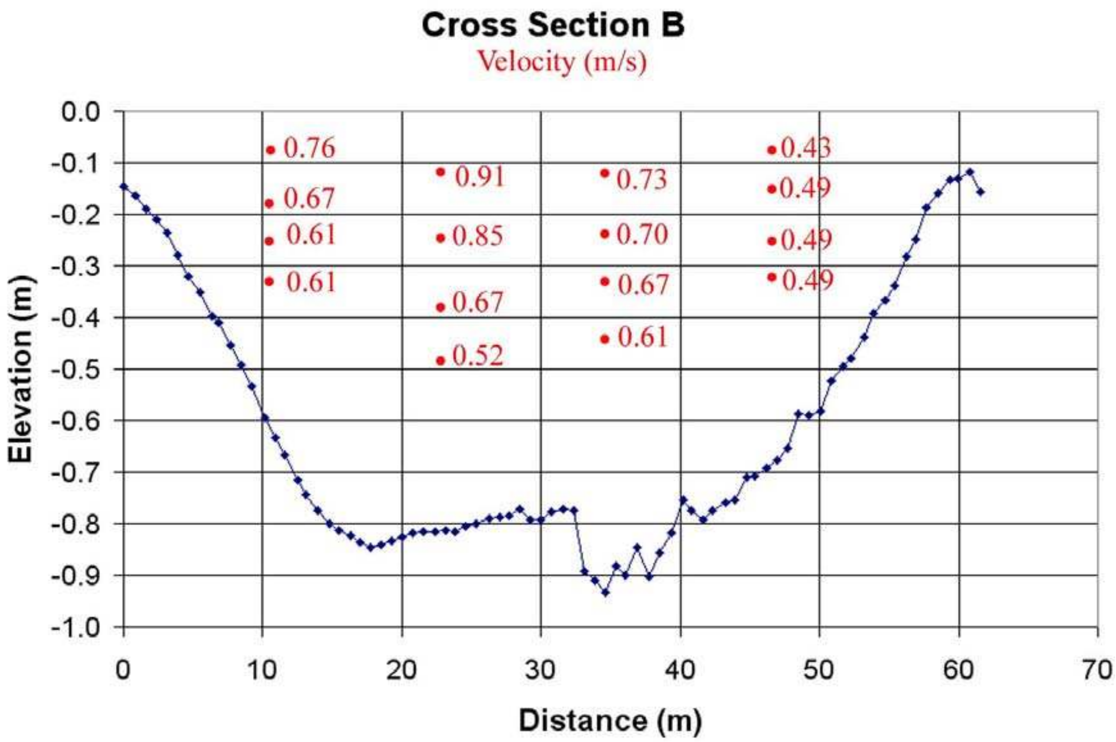


Figure 4.14. Sample velocity data at the cross section B in the Atchafalaya delta.

single cross-section (Figure 4.15). Grain size distribution of the suspended material was shown in Figure 4.16. The grain size distribution of bed sediment collected at the synoptic survey stations was uniform (Figure 4.17).

4.2 Model Calibration

4.2.1 Hydrodynamic Model Calibration

Generally, calibration of the hydrodynamic and sediment transport models were performed for the periods or event when the best observed data were available within the model domain and in the vicinity. Availability of the inflow discharge, tide, distribution of flow through the Atchafalaya delta, and velocity of flow must be considered before selecting a time period for the model calibration.

Data from several existing stations and from newly established stations were used to create input boundary files for the hydrodynamic and sediment transport model. For the hydrodynamic model, upstream boundary conditions at Morgan City were given as the flow boundary. Downstream boundaries of Atchafalaya Bay, Four League Bay, and the distributary channels were given stage boundary conditions, while inflow discharge boundary at Morgan City was obtained from the measured flow at that site. A bay tide boundary of the model was developed, using the observed tide at the AT04-04 station, Eugene Island. This station was very close to the boundary of the model, and assumed to have a highly similar tidal amplitude and phase.

For this study, variable roughness values were used in the model, based on the type of water features and the vegetation type. The Manning's roughness value generally

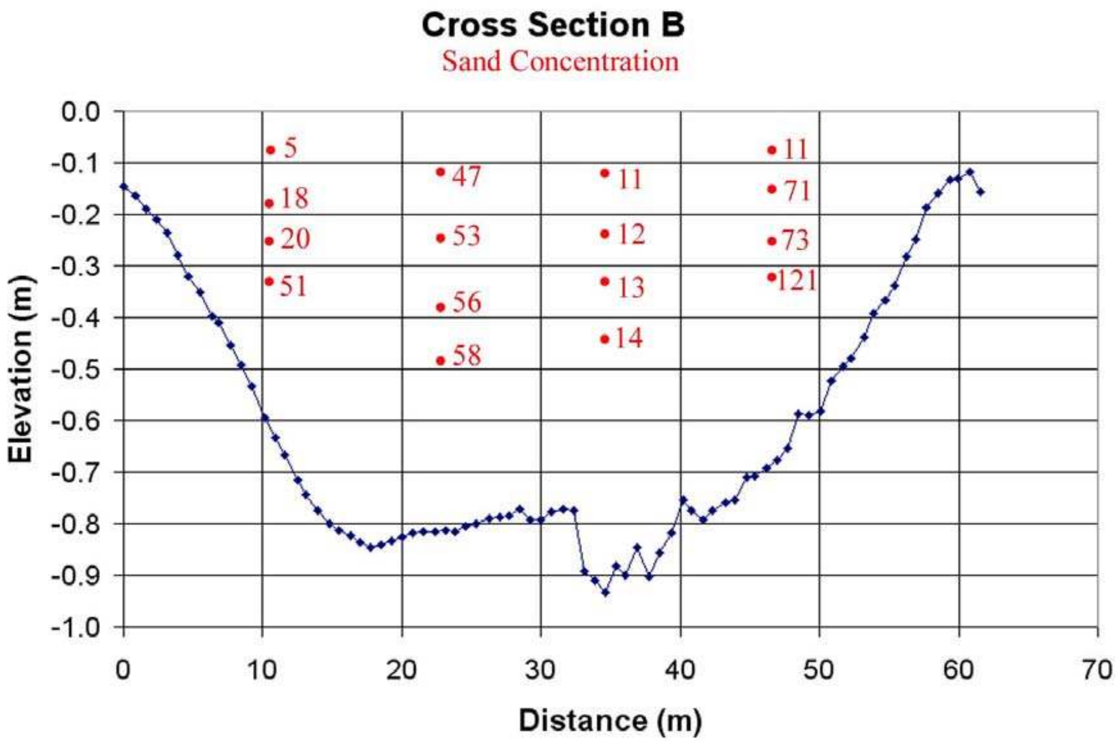


Figure 4.15. Sample coarse sediment concentration (in ppm) at the cross section B in the Atchafalaya delta.

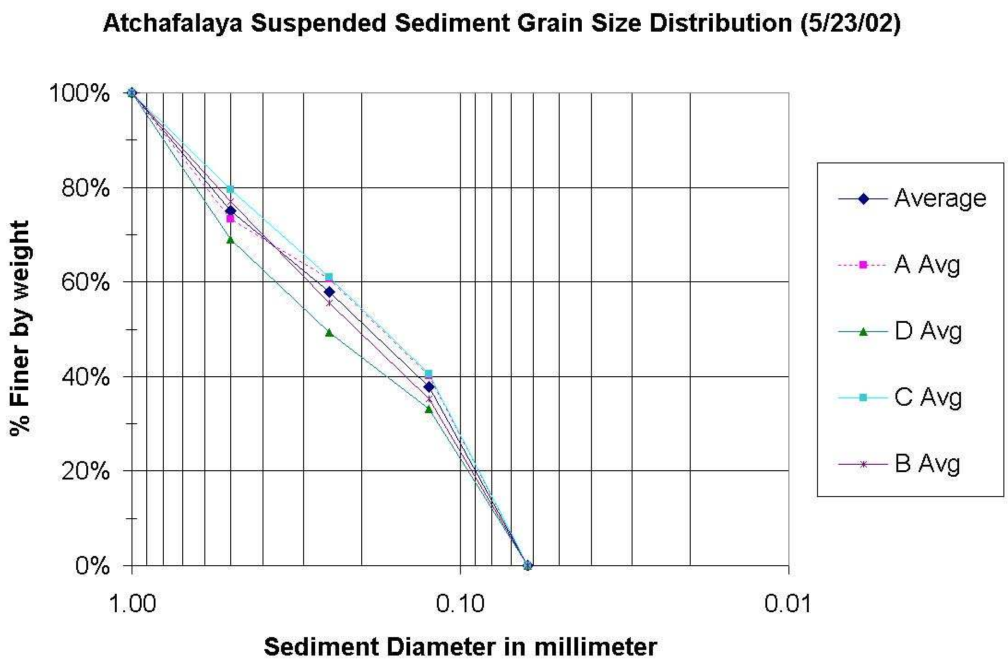


Figure 4.16. Suspended sediment size distribution in the Atchafalaya delta.

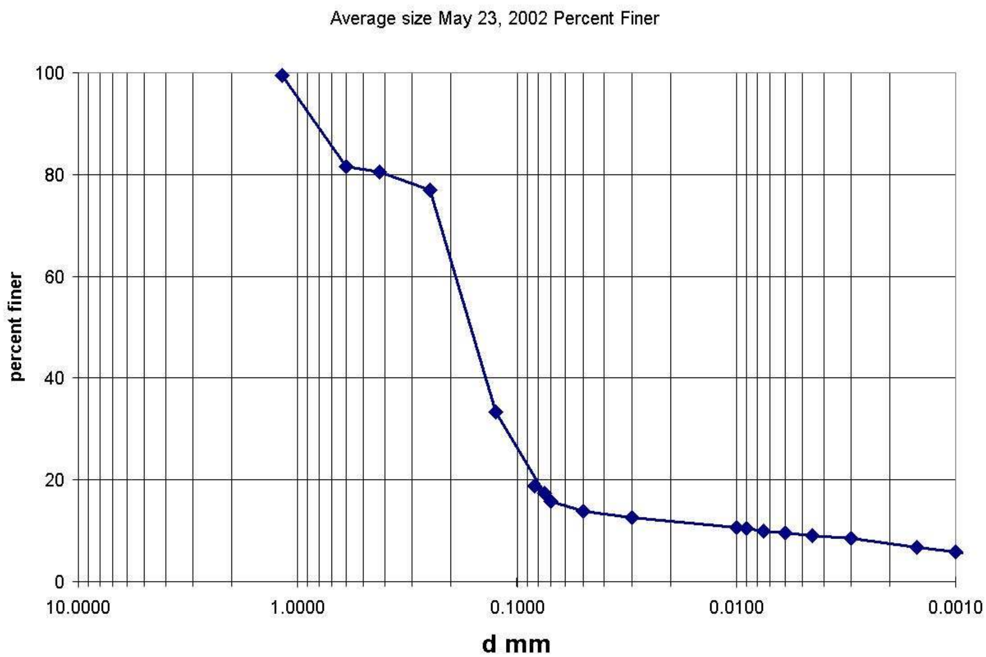


Figure 4.17. Bed sediment size distribution in the Atchafalaya delta.

used was 0.0250. The roughness values were raised in the bay area to 0.030. Roughness values were individually adjusted for the Atchafalaya main channel, Atchafalaya distributary channels and Atchafalaya Bay areas to achieve the closest match to water surface elevations observed at the calibration stations.

Values used for eddy viscosity ranged from 400 lb-sec/ft^2 (19160 Pascal-sec) for smaller elements to 750 lb-sec/ft^2 (35925 Pascal-sec) for larger ones. Higher numbers were used if it was observed that the area of interest displayed higher flow through the element than an area of similarly sized elements.

Several boundary condition files were created to run the hydrodynamic component of the TABS-MD model known as RMA2. The hydrodynamic model was calibrated with continuous tidal data, as well as information obtained from synoptic surveys.

Continuous stages recorded at all four stations were compared to model-simulated stages at the corresponding locations. (Figure 4.18 and 4.19.) Scatter diagrams of observed and simulated stages were also plotted, with a perfect (1:1) match line (Figures 4.20 and 4.21). A time series plot at the station in the eastern side of the Atchafalaya River (ID AT04-03) showed minor amplitude and phase differences between simulated and observed stages (Figure 4.18). The model prediction was slightly high at high tide, and low by a greater margin during low tide. At high tide, the simulated stage was always in phase with the observed tide. The simulated stage lagged to 1 to 2 hours at low tide. Similar trends were observed for the other three stations (Figure 4.18 and 4.19). On the other hand, the model at the upstream end of the delta consistently performed slightly under predicted stages (Figure 4.19 and 4.21).

Several researchers have determined the goodness of fit of hydrodynamic models by computing the absolute difference mean (ADM) and root mean square differences (RMSD) between observed and simulated stage (Liu et al., 2002; Hsu et al., 1999). The absolute difference mean (ADM) is the mean of the absolute values of all differences between simulated and observed values:

$$ADM = \frac{1}{n}\sum_{1}^{n} Abs(Simulated - Observed)$$

The Root Mean Square Difference (RMSD) between observed and modeled data is calculated by summing the square of the difference between the two, then taking the square root of the total and dividing it by the number of records.

$$RMSD = \sqrt{\frac{1}{n}\sum_{1}^{n}(simulated - observed)^2}$$

Both ADM and RMSD provide a measure of variance between observed and simulated stages. Performance of the calibrated Atchafalaya model was evaluated using both methods (Table 4.2). RMS error for the stations in the delta varied from 0.05 to 0.06 meter, while the ADM value was 0.04 at all stations. These numbers are within 10% of the tide range (approximately 0.6 m). RMS error at the upstream River station was 0.12 meter or within 20% of the tide range.

Manning's coefficient is the most important calibration parameter affecting the water elevations. To adjust Manning's higher led to a higher stage; to lower Manning's caused the stage to fall. If a higher n were used to raise the stage to match low tide, then more divergence would have occurred at high tide. Because this modeling effort is designed primarily to simulate sand transport under flood conditions (when stage is high in the delta), the Manning's coefficient was adjusted to reproduce stages with the highest fidelity at the high end of the tide range. Similarly stations within the delta were given higher priority than the station located in the main channel upstream.

In Atchafalaya Bay, stage fluctuation is a complex function of river flow, tide, and wind-driven set-up. Some discrepancies between the model and the observed data should be expected, given that all forcing functions (e.g., wind) were not included in the calibration.

Table 4.2. Root mean square difference and absolute difference mean for all calibration stations.

	Root Mean Square Difference (RMSD) in meter	Absolute Difference Mean (ADM) in meter
Station ID	meter	meter
AT04-01	0.12	0.10
AT04-02	0.06	0.04
AT04-03	0.05	0.04
AT04-06	0.06	0.04

One set of velocity data was collected during the pseudo-synoptic survey for all stations shown in Figure 4.11. Average simulated velocity was compared to measured velocity (Table 4.3). Predicted velocity was somewhat lower than the observed mean velocity at all stations, but only differed significantly (72%) at cross-sections C and D, which are most affected by tidal fluctuations. Clearly, a larger number of observations at these stations would be desirable.

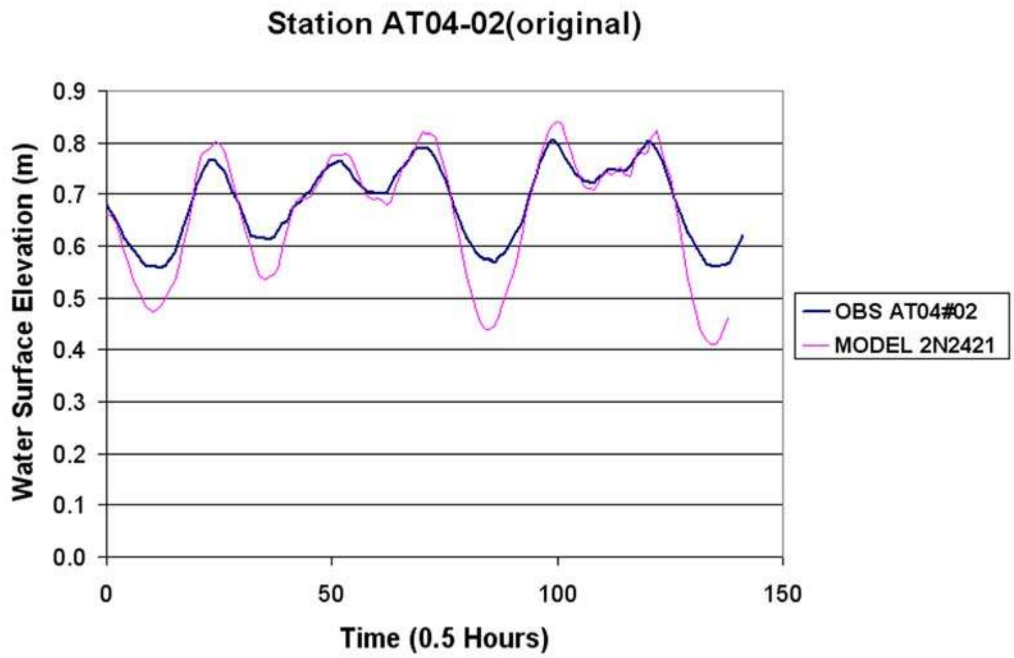
4.2.2 Sediment Transport Model Calibration

The archived Atchafalaya River data shows that sediment concentration varies during a flood event (Figure 4.6). Earlier sediment transport model calibrations for the Atchafalaya River were based on suspended sediment concentrations observed during synoptic surveys and by comparison of simulated sediment transport with historical dredging records (Donnell et al., 1991). The sediment transport model again was calibrated with synoptic sediment concentration data, and compared with historical dredging records. Current model results were examined to ensure that locations of simulated deposition within the navigation channel corresponded with the principal reaches where dredging is typically required. Calibration of the model for suspended sediment concentration with synoptic data provides confidence that the model can reproduce concentration for real-time simulations on a time scale of days or weeks.

For the synoptic survey calibration, inflow sediment concentration was obtained at the Morgan City station. Observed total concentration at Morgan City was 300 ppm during the May 2002 calibration period; the fine part of the total load was 227 ppm and coarse fraction totaled 73 ppm. The gulf boundary was specified for outward flux only, with no return of material permitted to the model domain. An initial sediment concentration was set at 50 ppm. For the calibration simulations of the SED2D model, the effective grain size of the material was assumed to be at 0.012 mm with a fall velocity of 0.002 m/s. The SED2D model was forced by output from the earlier calibrated RMA2 model. Predicted concentrations at the synoptic cross-sections were compared with those observed (Table 4.3). The observed data shows that concentration of sediment is variable, even in the main channel. Concentration was 23 ppm at section A, while at the downstream section B, the average concentration was 52 ppm. At the time of the model calibration, a single set of sediment data was available for comparison. Based on the fluctuations of sediment concentration observed in the Atchafalaya River, this particular simulation shows reasonable agreement with field data in the area of the delta. Additional sediment transport model simulations were performed to compare the model results with the historical dredging record. This validation of the SED2D model will be discussed later.

Table 4.3. Sediment and velocity calibration.

Station ID	Observed	Model	Observed	Model
	Sediment PPM	Sediment PPM	Velocity (m/s)	Velocity (m/s)
A	23	75	0.58	0.52
B	52	67	0.58	0.51
C	51	58	0.61	0.44
D	51	36	0.52	0.18



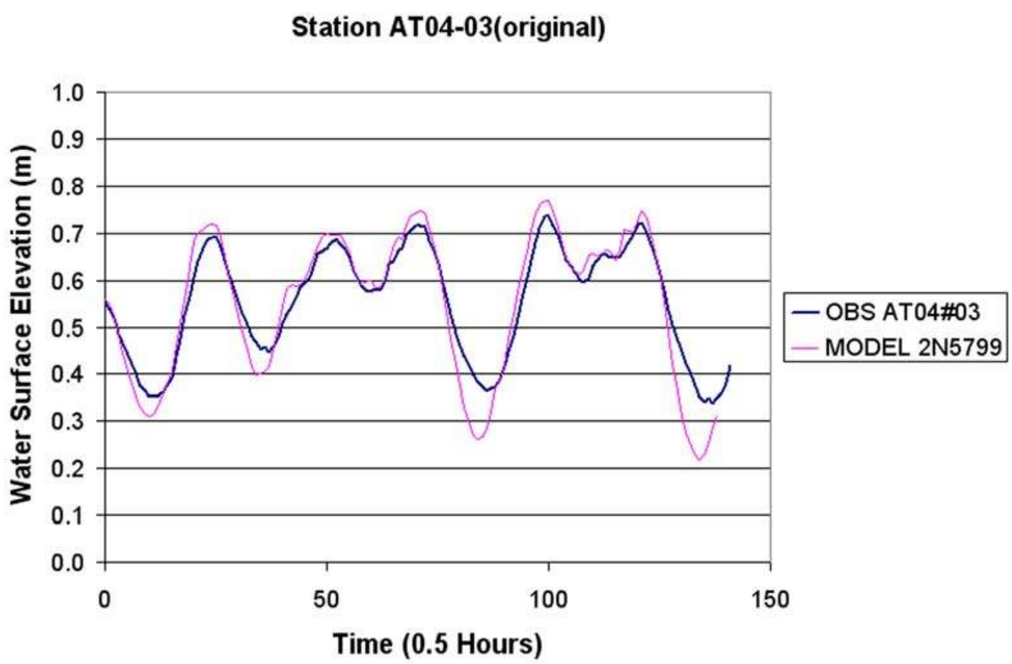
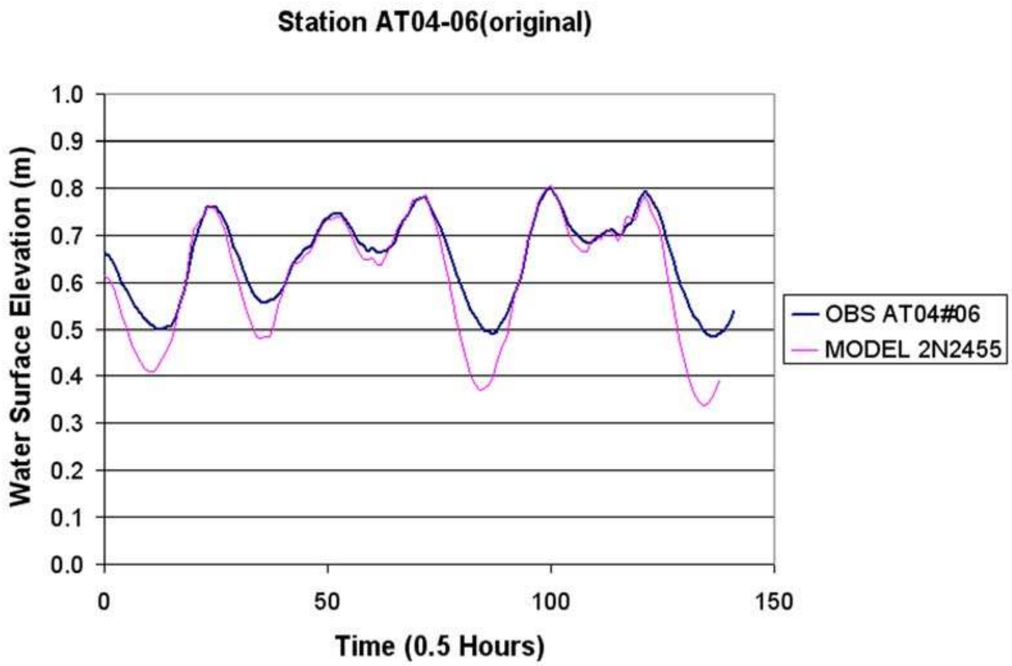


Figure 4.18. Model calibration for tide at station AT04-02 and 03.



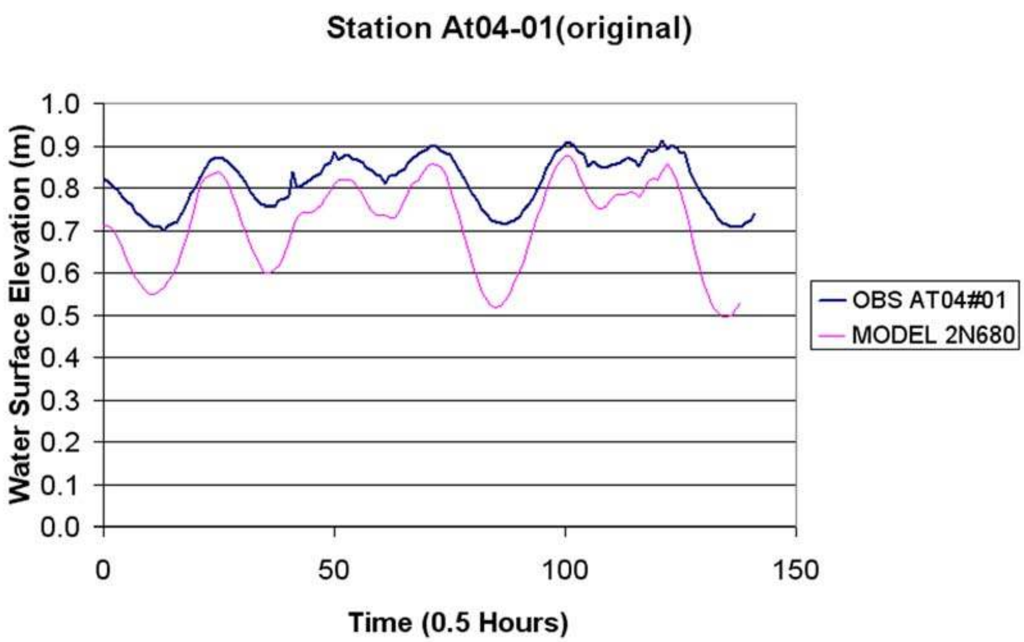
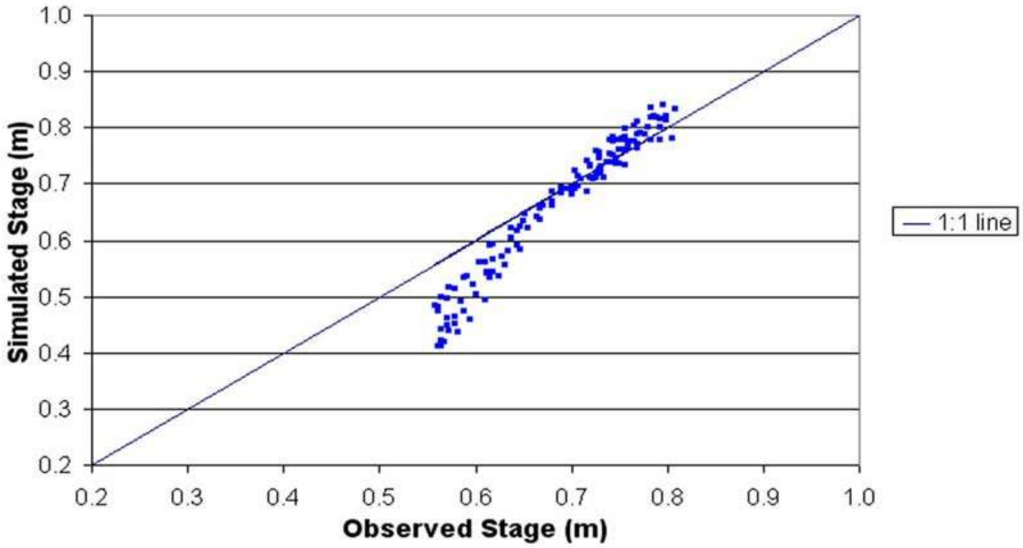


Figure 4.19. Model calibration for tide at station AT04-06 and 01.



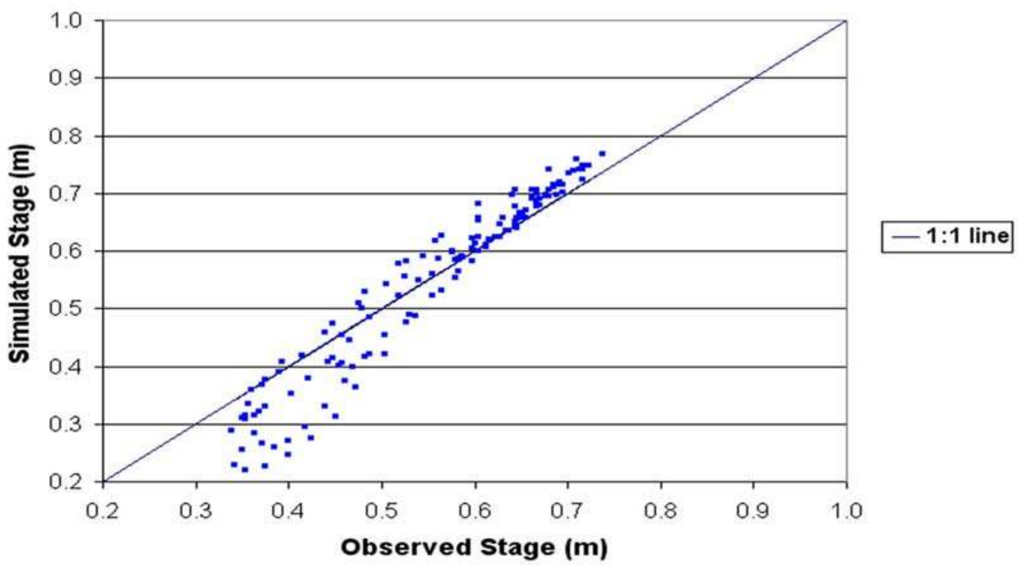


Figure 4.20. Scatter plot of observed and simulated stages at AT04-02 and 03.

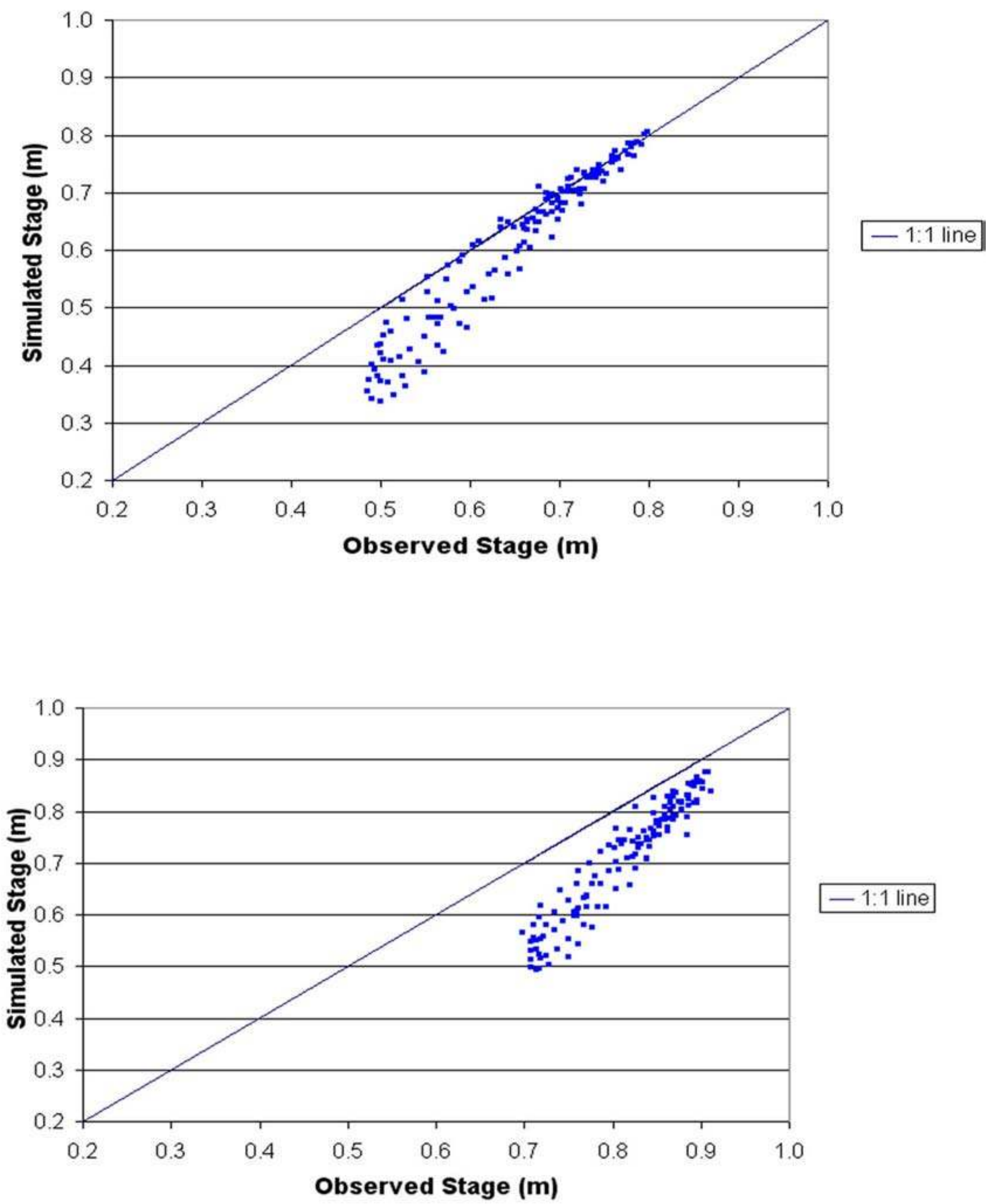


Figure 4.21. Scatter plot of observed and simulated stages at AT04-06 and 01.

4.3 Model Simulations

Two experiments were set up to establish the feasibility of using SED2D, a sediment transport model driven by 2D hydrodynamics (RMA2). The first experiment was designed to determine whether the model could predict the spatial distribution of complex geomorphic features. The second focused on temporal scaling, to show that concatenating results of relatively short runs (two weeks in the prototype) can be used to predict sediment transport on the scale of the flood hydrograph (two to four months).

4.3.1 Experiment 1 – Effects of Geometry on Capacity to Simulate Deltaic Deposition for a Steady Discharge and Sediment Input

Before advancing to the complexity of prototype geometry, it was thought useful to determine first whether a numerical simulation of river mouth bar formation could be acceptably demonstrated for a simple, near-symmetrical mesh depicting the entrance of a relatively deep distributary channel into a shallow bay in idealized form. This preliminary step also provides a basis for comparison for small-scale experiments performed in the laboratory.

Two finite element meshes with similar dimensions were developed based on Ratcliff Pass, one of the main distributary channels on the eastern side of the Atchafalaya River delta, for which detailed bathymetry was available (Figure 22). Ratcliff Pass was selected as the base for both the idealized and prototype meshes, because the geometry was more natural, or less affected, by dredging than other delta. The first simple mesh was symmetric in shape (Figure 4.23 top). Its boundaries and orientation were designed to create symmetric computational nodes and elements that were perpendicular

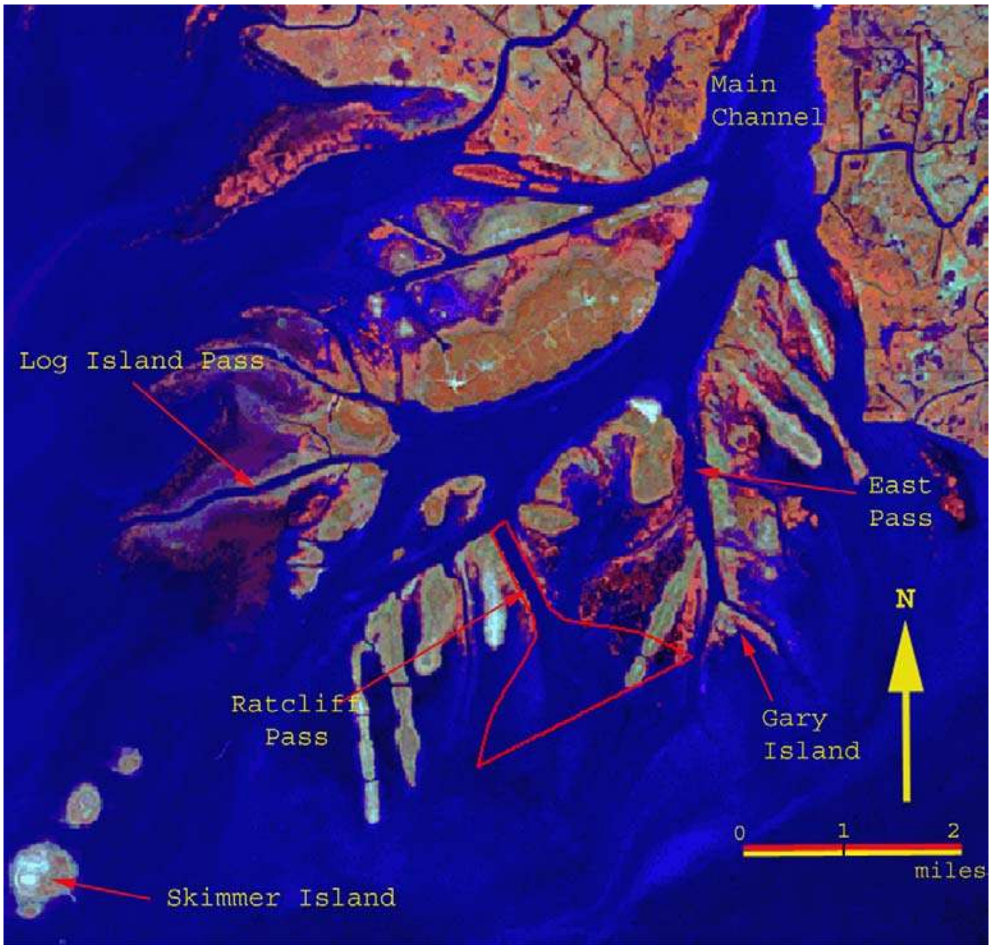


Figure 4.22. Location of Ratcliff Pass and approximate size of the idealized test mesh.

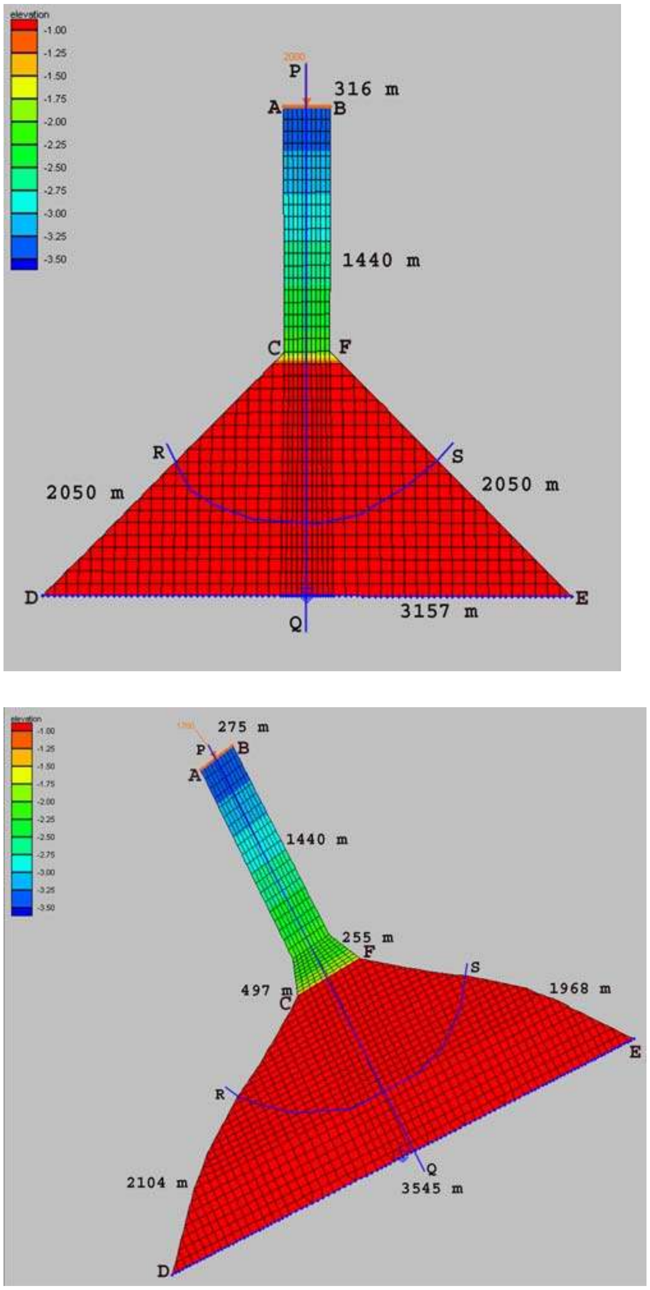


Figure 4.23. Geometry of the idealized test mesh. [Symmetric (top), asymmetric (bottom)].

to the direction of inflow. Channel depth ranged from -3.5 m at the entrance to –1.0 m at the bay boundary. The second mesh more closely approximated the prototype, while retaining the same general proportions (Figure 4.23 bottom).

The Atchafalaya River delta was built by flood deposits of coarse sediment, but not all floods are equal in duration, nor does every flood necessarily result in significant channel elongation or bar formation. Early stages of sub-aerial growth in the Atchafalaya occurred during the high flood of 1973 to 1976 (van Heerden, 1980 & 1983). Deposition of relatively coarse-grained distributary-mouth bar and levee features were associated with rapid channel extension and bifurcation. These features accounted for the majority of sub-aerial growth of the new delta. Accordingly, inflow discharge and sediment concentration were chosen to be similar to observed conditions during a high flood. In this case, inflow and sediment concentrations used in the test models were comparable to what was observed during the flood of 1973.

The adopted hydrodynamic (RMA2) and sediment transport (SED2D) model parameters for the first experiment were unchanged from the calibrated model (Table 4.4). A steady inflow of 70,000 cfs (2,000 cms) was used as the inflow hydrograph in the RMA2 model. This flow was similar to the peak flow that would have passed through the Ratcliff Pass, or a pass similar to the size of the Ratcliff Pass in 1973. A constant supply of fine sand at a constant input concentration of 500 ppm was used in SED2D to ensure a sediment-rich regime.

The hydrodynamic model RMA2 was run for a two-week cycle. Stages and velocities from each cycle were passed to the sediment transport model. At the end of

Table 4.4. Dimensions and input parameters used in experiment 1.

Name	Symmetric Mesh	Asymmetric Mesh
Channel Segment AB (meter)	316	275
Channel Segment AC (meter)	1440	1650
Channel Segment CD (meter)	2050	2104
Channel Segment DE (meter)	3157	3545
Channel Segment EF (meter)	2050	1968
Channel Segment BF (meter)	1440	1650
Channel Segment CF (meter)	316	497
Inflow at AB (cms)	2000	2000
Sediment through AB Inflow (ppm)	500	500
Time Step (hours)	0.5	0.5
Total Simulation time (Week)	2	2
Sediment Size (mm)	0.11	0.11
Fall Velocity (m/s)	0.01	0.01
Manning's Roughness	0.025	0.025

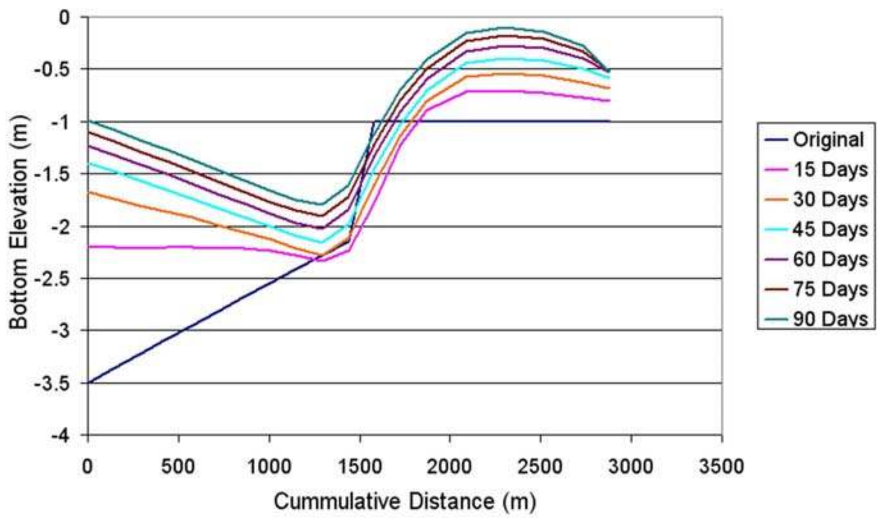
each two-week SED2D simulation, a new bathymetric configuration was produced. This new bathymetry was imposed on the hydrodynamic model for the next cycle. Repetition of sequence was repeated until the desired number of simulations were made (5 to 7).

Changes in bottom elevations generated by the SED2D model at the end of each simulation were plotted for nodes located on longitudinal (PQ) and transverse (RS) sections (Figure 4.23). These lines were selected to compare patterns of deposition or scour.

4.3.1.1 Simulation of the River Mouth Bar

Bottom elevation profiles along the longitudinal PQ section after 6 cycles of simulations (90 days prototype) were compared for the symmetric and asymmetric meshes (Figure 4.24). The occurrence of a bar-like feature at the mouth of the channel is clearly shown for both the simple and complex geometries. The channel thalweg ascends gradually to a bar crest located 730 m downstream of the mouth of the distributary channel. The bar has a steeper slope on the upstream than downstream side. The rate of sedimentation, indicated by deposition within test control volumes, decreases with distance from the mouth of the channel (Figure 4.24).

The model-simulated distributary mouth bar formations can be compared to the deltaic formations described schematically by van Heerden (1983) and Welder (1959) (Figure 4.25). The simulated bar in both the symmetric and asymmetric tests is bisected by an extension of the channel (Figure 4.26). The bar adjacent to the channel shows signs of transformation into sub-aqueous levees that define the sides of the channel. The overall pattern of the sand bar deposition is developed by the SED2D model (Figure 4.26)



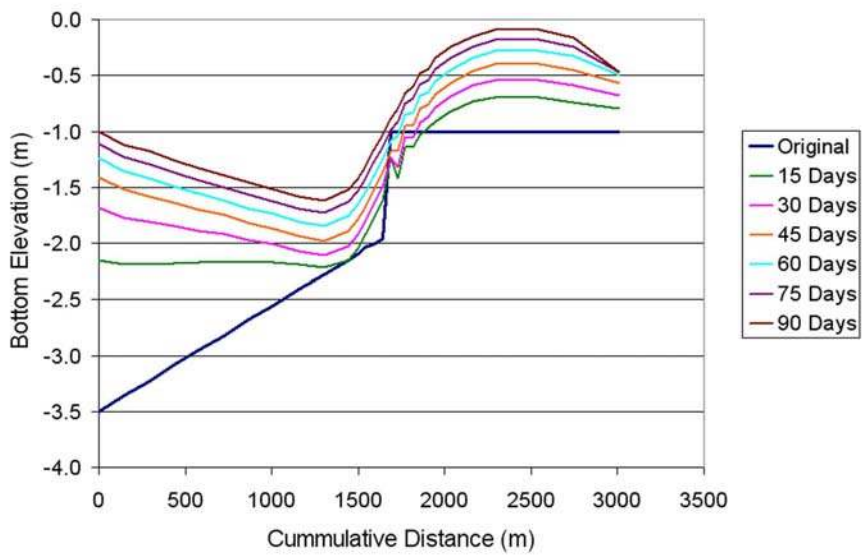


Figure 4.24. Bottom elevation plot after 90 days of simulations.

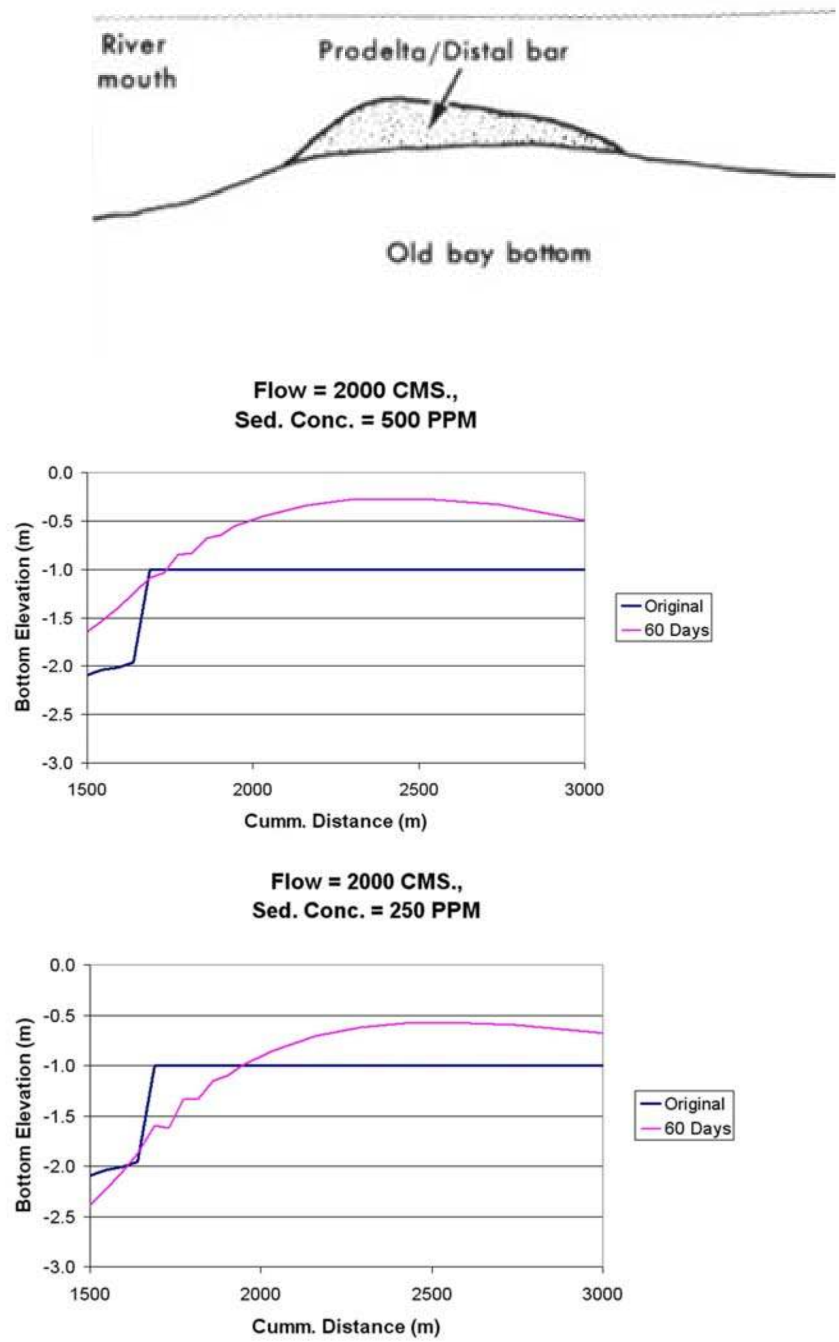


Figure 4.25. Observed delta lobe during inception (top) (Welder, 1959). Simulated delta lobe at the mouth of the feeder channel (middle and bottom).

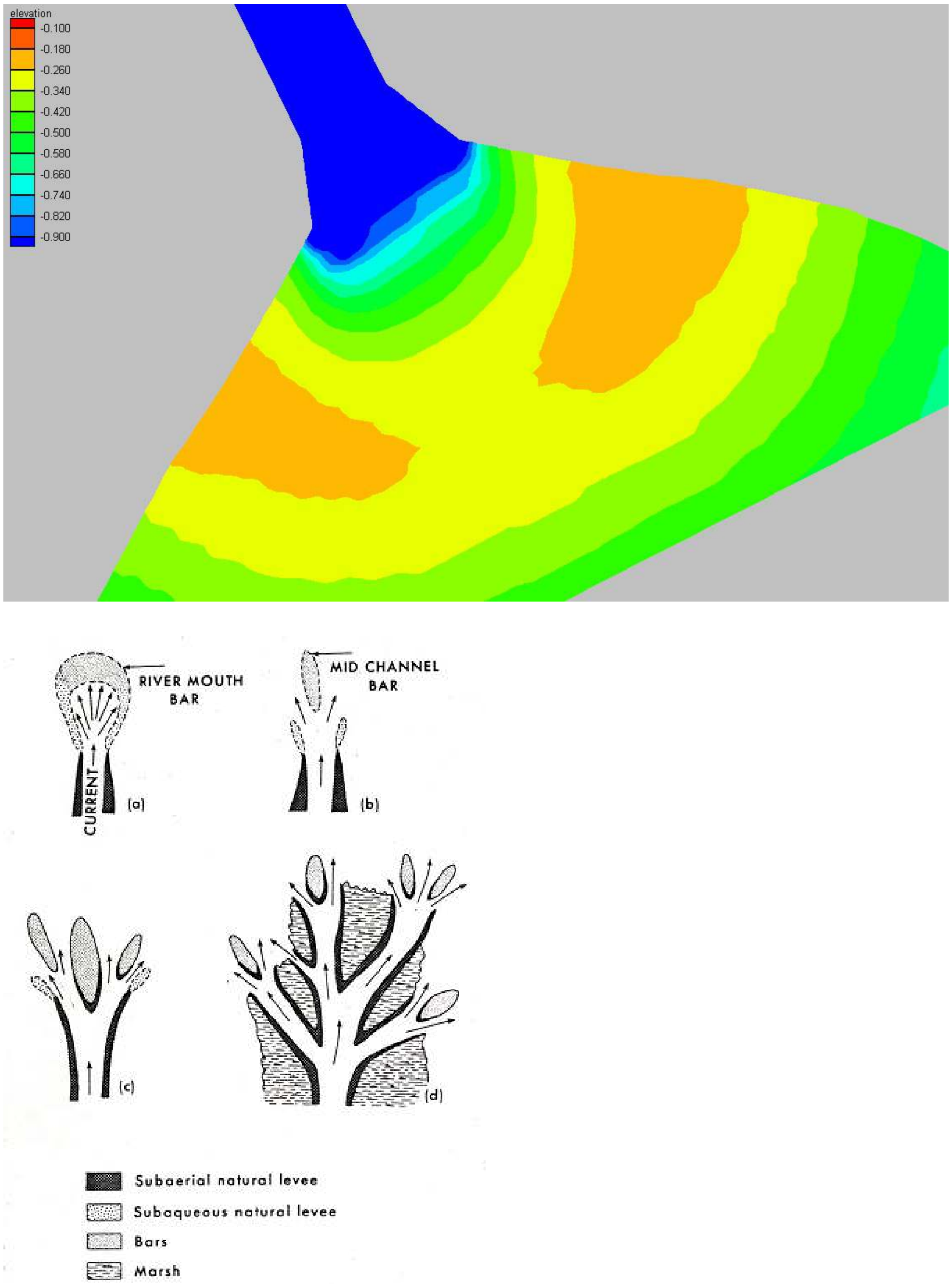


Figure 4.26. Simulated delta lobe at the mouth of the feeder channel (top). Observed delta lobe during inception (bottom a).

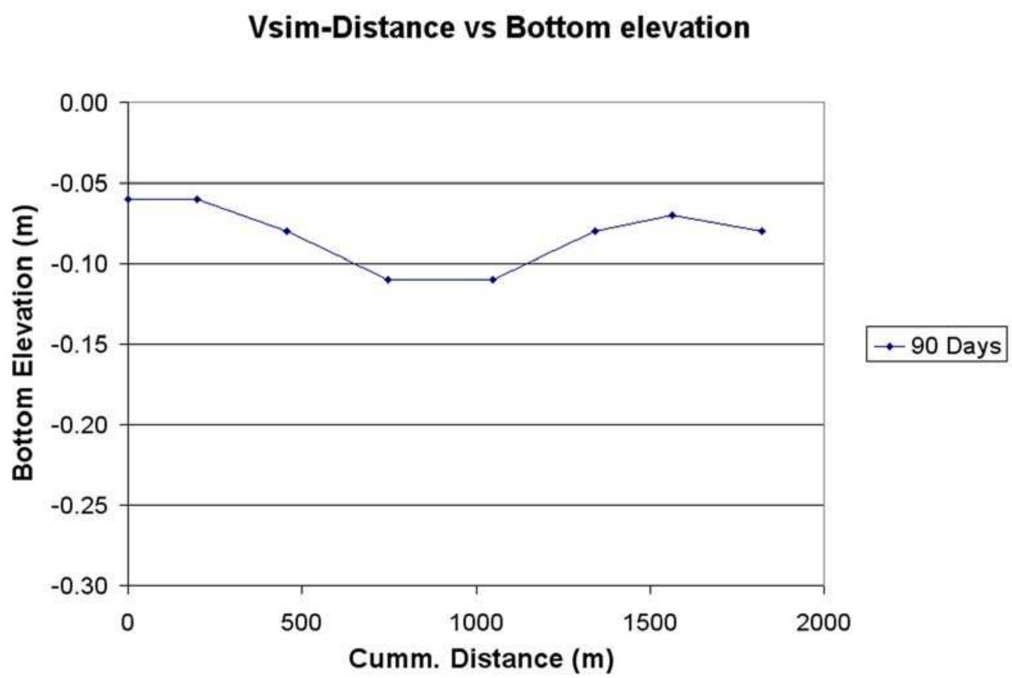
compares qualitatively with mouth bar configuration observed by van Heerden (1983) and Welder (1959). In a qualitative sense, the shape of this deposition is similar to that described by Hatanaka and Kawahara (1989) in the laboratory.

4.3.1.2 Channel Extension and Bifurcation

The bottom elevation profile was plotted along the transverse section RS after 90 days and 60 days of simulation for the symmetric and asymmetric mesh, respectively (Figure 4.27). Although deposition was observed along the entire section, the occurrence of a channel and higher flanking sub-aqueous levees is visible in both simulations. The difference is that the channel thalweg is offset, and levee elevations differ more significantly in the asymmetric version (Figure 4.27), as is observed in nature from surveys.

Flow and sediment concentrations were high in the experiment, as were stages. The levee crests remained sub-aqueous, but real sub-aqueous levees become sub-aerial when the stage falls. When a mouth bar or levee becomes sub-aerial, the flow splits and deflects around the bar, causing scour and erosion leading to channel bifurcation (Welder, 1959).

Bifurcation is a process that takes place over a longer period of time then can be simulated numerically. For example, a large number of delta lobes and new channels were developed during the three large Atchafalaya floods between 1973 and 1975. On the other hand, during the 1976 to 1982 period, no large floods occurred, and no significant seaward extension or channel bifurcation was observed (van Heerden, 1980 & 1983).



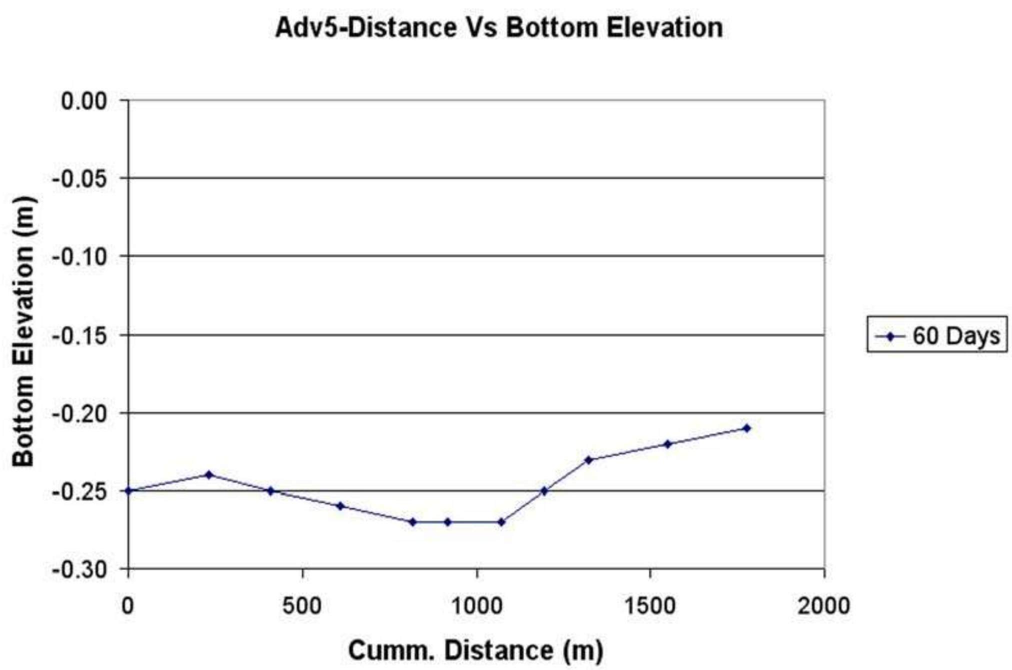


Figure 4.27. Development of the sub-aqueous channel and levee.

A simulation of processes that occur over several years is beyond the capability of existing 2D sediment transport models and present computing technology.

As described earlier, when deposition becomes large enough to create a sub-aerial delta lobe, the flow from the distributary channels passes around the lobe. To mimic this condition in the model, a few elements within the computational domain were removed to create an artificially high ground that could not be flooded. These elements were positioned at the crest of the natural levee deposits so that, as described by Welder (1959), flow would be forced around these features (Figure 4.28 top). New hydrodynamic (RMA2) and sediment transport (SED2D) simulations were made with the modified geometry file, as previously described. Scour to create new channels were observed between the artificially raised levee crests (Figure 4.28 bottom) after two cycles of simulations. These results suggest that mesh manipulations may be reasonably imposed to permit investigations of a process that actually occurs over the course of one or more hydrographs within the current limitations of computational technology.

4.3.1.3 Equilibrium Adverse Slope Development

It was observed in both the symmetric and asymmetric series that the slope of the distributary channel bed changed significantly over the course of a few simulation cycles (Figure 4.29). The final slope is less steep than that initially specified. The initial adverse slope of the mesh was 1V-51H. After the establishment of the equilibrium, the grade of the slope decreased by a factor of 10 to1V-450H (Figure 4.29). A series of sub-parallel equilibrium slopes was developed and maintained as the delta-wedge developed across the river mouth. Model simulated slopes were compared with those observed in various

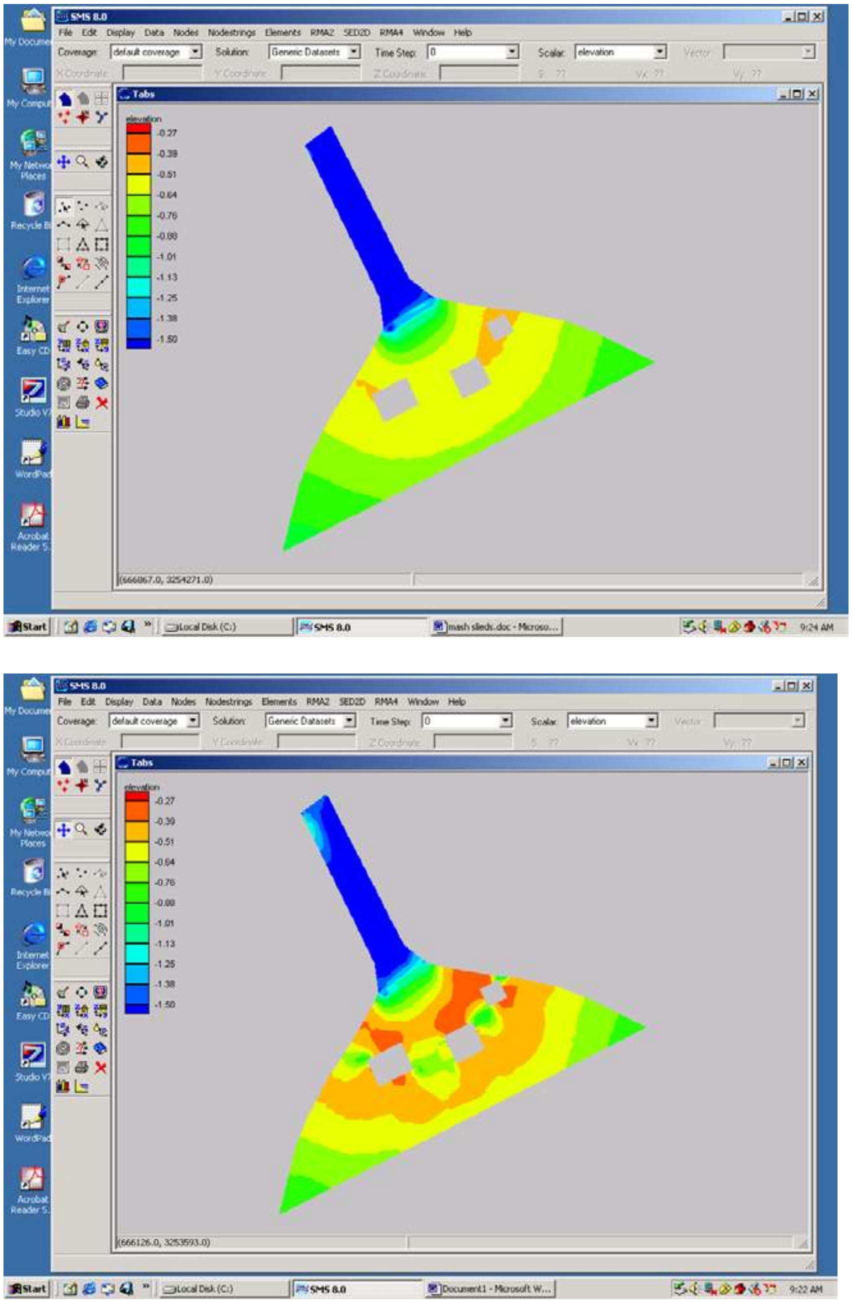
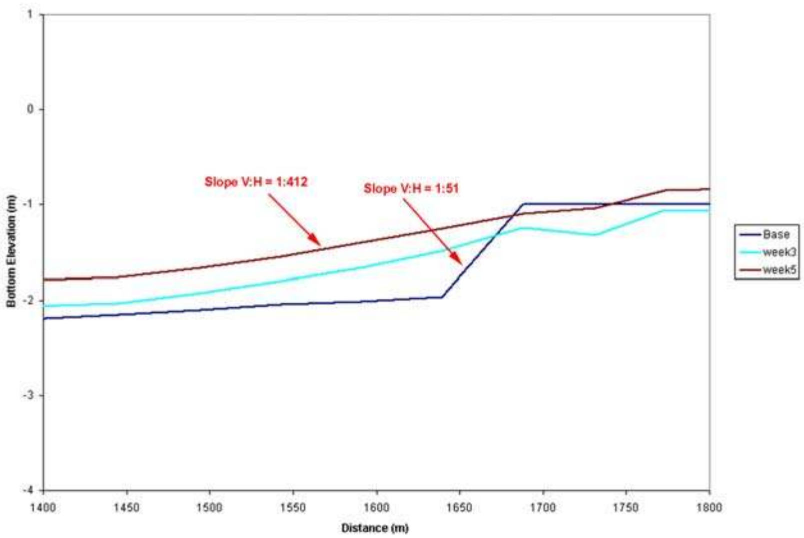


Figure 4.28. Nodes removed from the mesh to mimic sub-aerial delta lobes (top) and formation of channels during simulations (bottom).



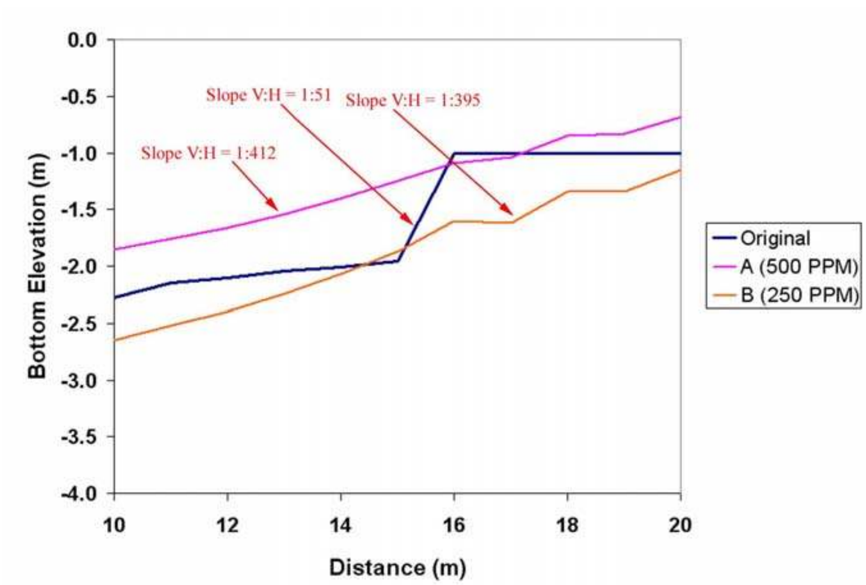


Figure 4.29. Decrease in angle of adverse slope and development of equilibrium slopes under two input concentration regimes.

distributaries of the Wax Lake Outlet delta (Figure 4.30, Table 4.5). The equilibrium slope developed in the experiments falls within the range observed in the Wax Lake Outlet delta (Table 4.5).

Numerical modeling performed with simple but realistic geometry displayed many characteristics of natural delta formations, development of a river mouth bar, channel extension across the bar, sub-aqueous levees, and equilibrium adverse slopes, with features observed in laboratory experiments. Formation of these features maintains consistency with patterns observed by van Heerden (1980, 1983) and Welder (1959).

To test model behavior for lower sediment concentrations that occur more frequently, a new set of simulations was made using a 250 ppm sand inflow, half of the concentration in the earlier test. No other parameters of the model were changed. Bottom elevations in the longitudinal directions were plotted (Figure 4.31). Though less rapid, the predicted bar formation was similar to that of the earlier run (Figure 4.29). The developed equilibrium adverse slope was equivalent to that formed under the 500 ppm input sediment concentration regime (Figure 4.29).

4.3.2 Experiment 2 – Prototype Geometry - Sediment Transport Capacity of the Atchafalaya River with a Constant Sediment Inflow and Different Peak Discharges

Two sets of simulations were made with the calibrated Atchafalaya River model (Figure 3.6). Based on the peak flood discharges observed during the past 30 years, an inflow hydrograph table was developed (Table 4.6). With these constant inflows into the

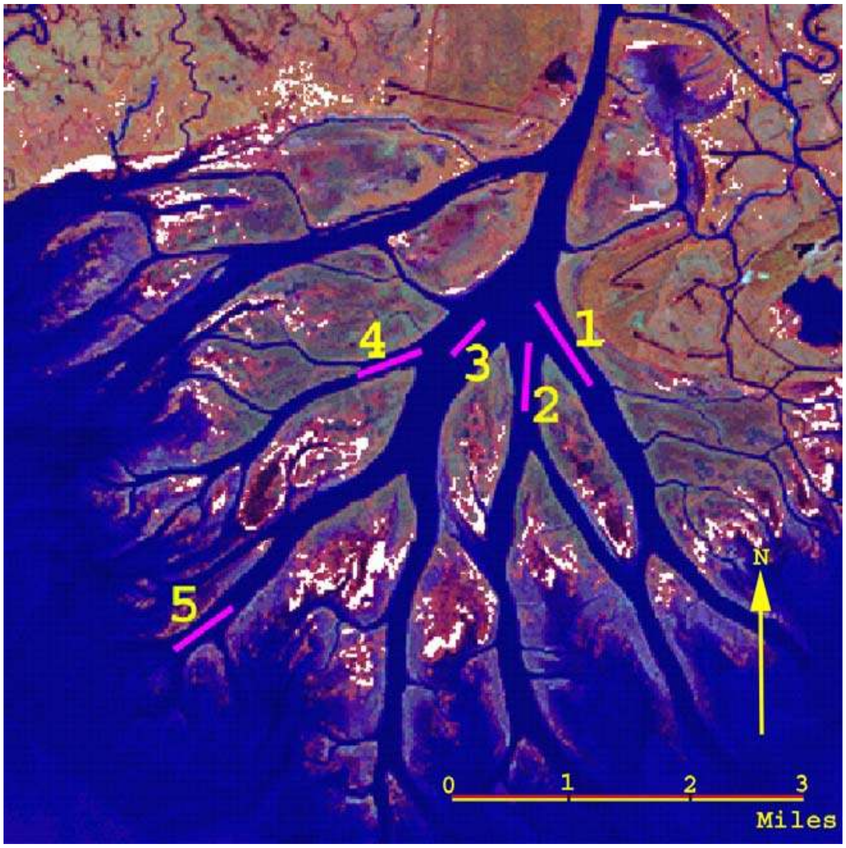


Figure 4.30. Locations of slopes in the Wax Lake Outlet delta.

Table 4.5. Locations and values of slopes in the Wax Lake Outlet delta.

Line	Slope (V:H)
1	1 : 573
2	1 : 850
3	1 : 515
4	1 : 441
5	1 : 340

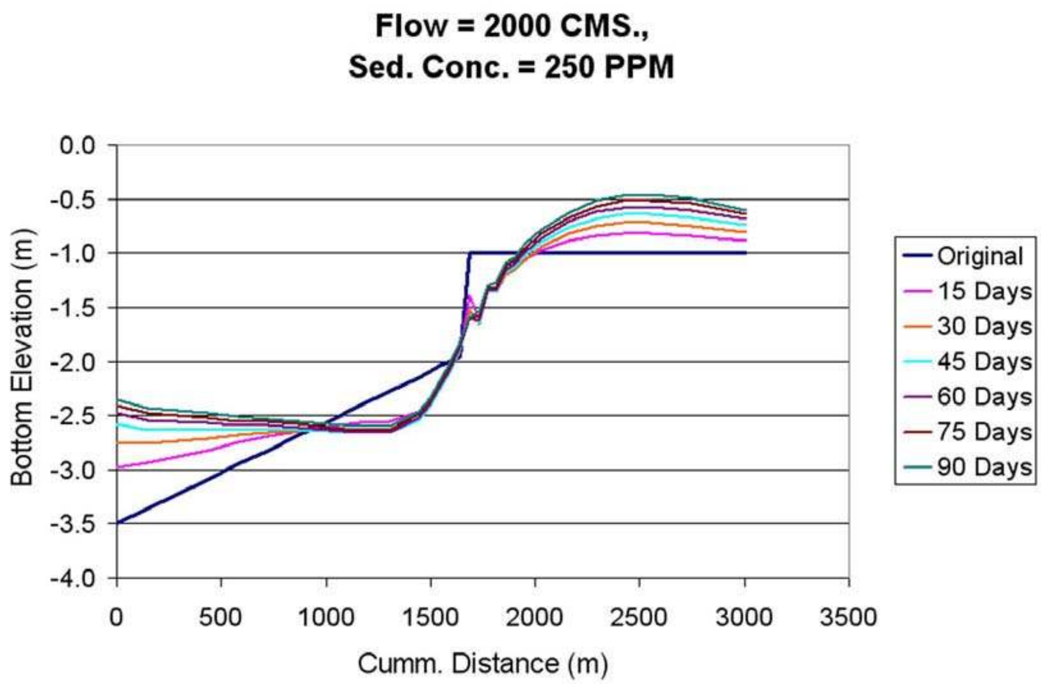


Figure 4.31. Bottom elevation plot after 90 days of simulations.

model, the hydrodynamic model (RMA2) operated for two-week periods. Stage and flow velocity information from these simulations were used in the sediment transport model (SED2D). In the SED2D model, an average inflow sediment concentration of 200 ppm at Morgan City and an initial condition of 50 ppm in the bay were used. One representative tide was used for the gulf boundary, while separate simulations were run for the silt (diameter 0.012 mm) and sand (0.093mm) sediment classes.

To describe the sedimentation or erosion for each set of simulations, nine sites within the Atchafalaya River and open bay mesh were selected for monitoring (Figure 4.32). Three sites (1-3) were chosen along the main channel and six sites (4-9) were selected in the bay at various distances from the mouth of distributary channels.

Sites 4, 5, and 6 were selected in the vicinity of the mouth of the Castille Pass Channel on the eastern margin of the delta. Sites 7, 8 and 9 were located on the western margin near the mouth of Log Island Pass.

The final run of the Experiment 2 sequence was much longer than had previously been attempted. It was designed to evaluate deposition over a real flood hydrograph with a constant 200 ppm sediment inflow. The observed flood during 1997 was used to perform the hydrodynamic and sediment transport simulations. The observed daily hydrograph was available for Simmesport for the entire 1997 year. At the inflow boundary, 70 percent of the Simmesport flow was assumed to flow through Morgan City. A continuous, 50-day simulation that included 25 days of the rising and 25 days of the falling limb of the 1997 hydrograph (Figure 4.33) was performed. The sediment transport

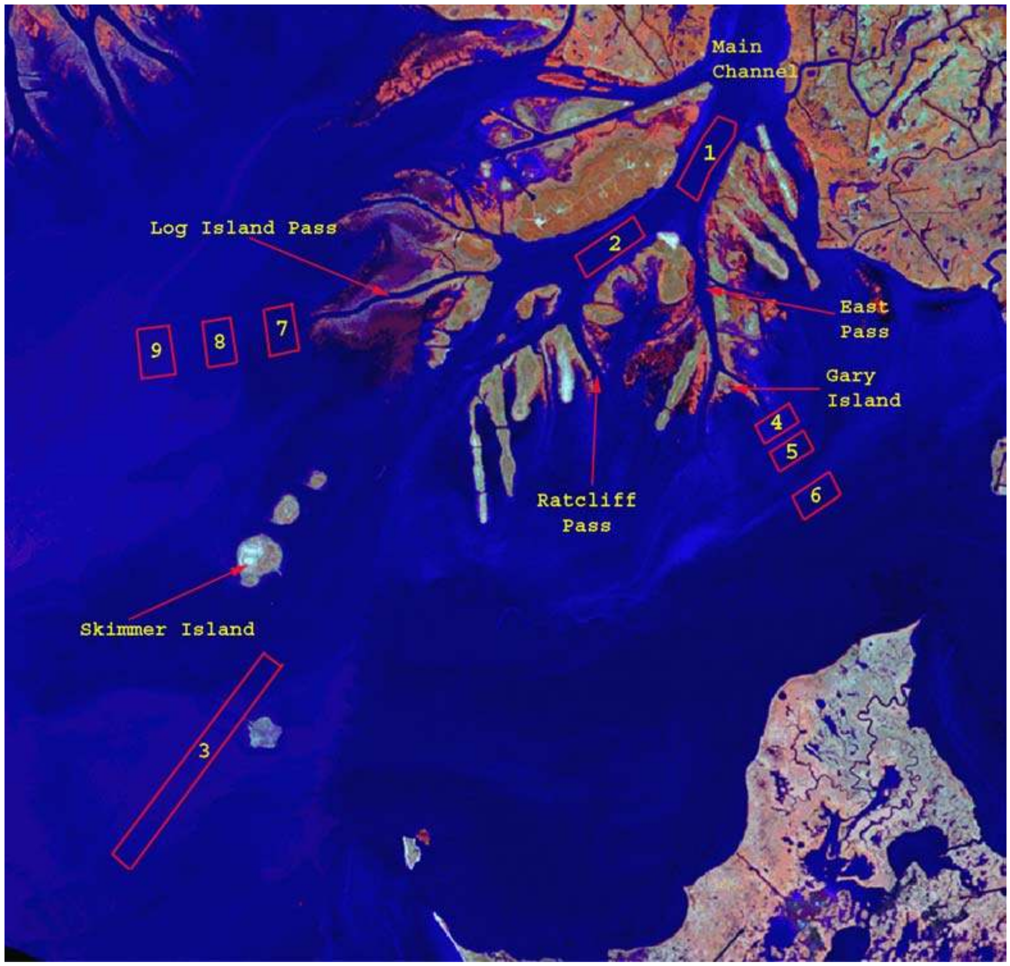


Figure 4.32. Delineation of the sedimentation or erosion sites.

Table 4.6. Schedule of flow at Morgan City.

Morgan City Discharge (CFS)	Morgan City Discharge (CMS)	Remark
300000	8493	Observed during 1984 and 1991 flood
400000	11324	Smaller peak than observed during 1983
500000	14155	Smaller peak than observed during 1975
600000	16986	Flow is between 1973 and 1975
700000	19817	Equivalent to the flow in 1973

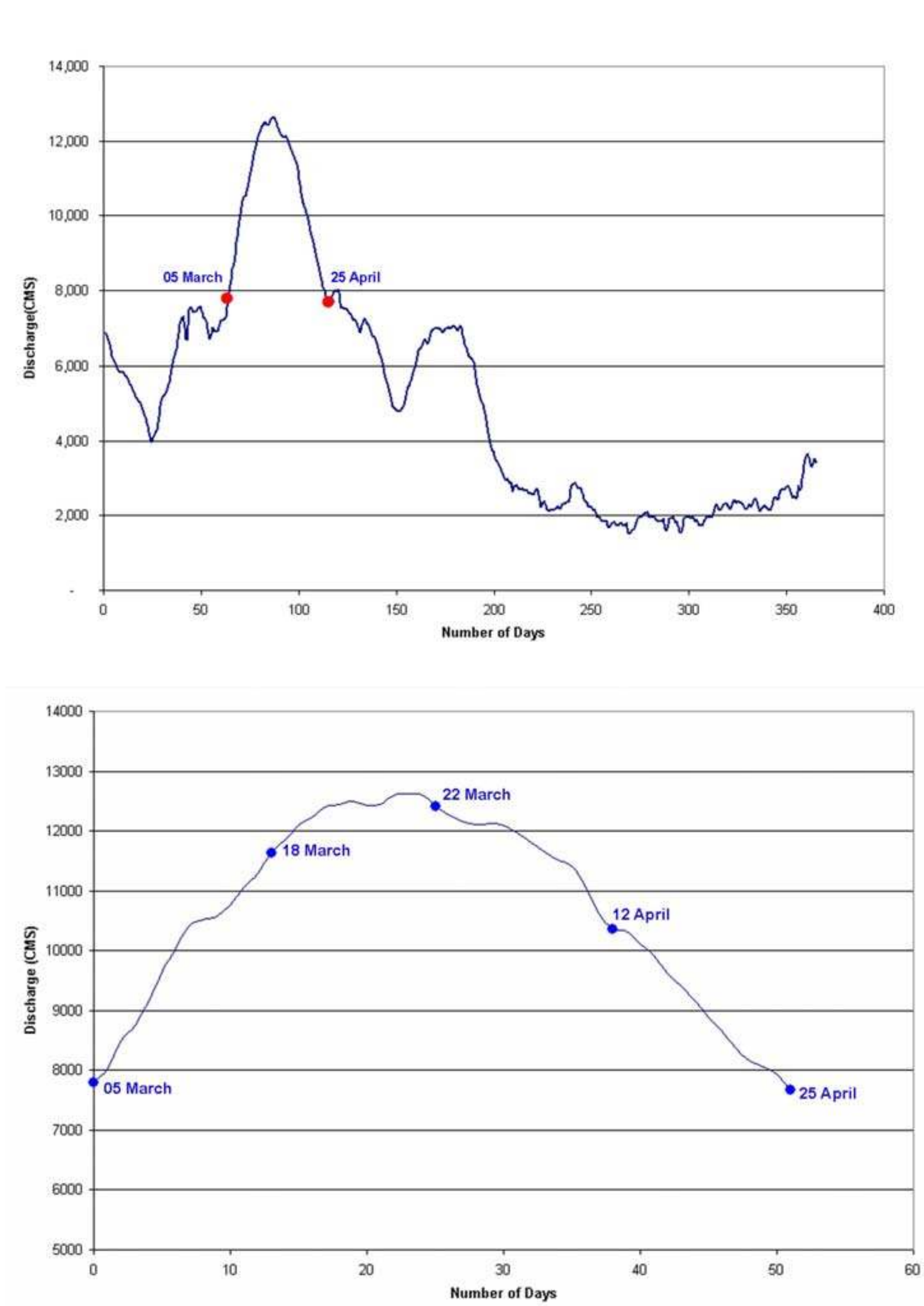


Figure 4.33. Range of the 1997 flood hydrograph selected for the simulation.

model was run separately for both the sand (0.093 mm) and silt (diameter 0.012 mm) sediment classes, using the prototype mesh. Rates of sedimentation or erosion obtained from the SED2D model were plotted for all nine sites (Figure 4.32).

4.4 Results of the Sediment Simulation

Sedimentation or erosion curves were generated for all nine sites (Figure 4.32) for sand (Figures 4.34, 4.35, 4.36 and Table 4.7) and for silt (Figures 4.38, 4.39, 4.40 and Table 4.8). The concentrations of suspended sand and silt were plotted separately along the Atchafalaya main channel (Figures 4.37, 4.41).

Finally, the hypothesis that sedimentation or erosion during a flood could be determined using the sedimentation erosion curves developed earlier was tested. For that purpose, the 1997 flood was divided into four two-week segments and deposition or erosion was calculated from the sand (Figures 4.34, 4.35, 4.36) and silt curves (Figures 4.38, 4.39, 4.40) previously developed.

Results from the continuous simulation of the full hydrograph were compared with those obtained from the shorter runs (Table 4.9). Differences for all sites ranged between 5 and 15% for silt. Larger variations were observed for the sand class at the most distal stations where less than a mm of sand was deposited. It was difficult to derive a number from the curves, when essentially no change occurred.

Simulated concentrations of suspended sand from the first test simulations were plotted longitudinally along the main channel (Figure 4.37). The model predicts a decrease in suspended sand concentration in the downstream direction, suggesting a depositing of sand in the main channel. The model further predicts that up to 80% of the suspended sand is deposited before River mile 145, a segment called the bay reach by the USACE (Figure 3.3). When simulated silt concentration is plotted longitudinally in the same manner (Figure 4.41), a very different pattern emerges. Most of the silt stays in suspension. while only 20% to 25% is deposited in the bay reach of the main channel.

The bay reach requires dredging nearly every year. Available USACE records for these sites show the composition of the material removed during each dredging cycle (Mashriqui et al., 1997). Bottom sediment composition in the downstream direction changes significantly as the bay reach transitions to what the USACE refers to as the bar reach (Table 4.10). Material dredged in the bay area (around River mile 145) is primarily sand, while that removed from the bar area is mainly silt and clay. These observations support the depositional pattern simulated by the model along the main channel. The simulated depositional pattern is in good agreement with the composition of bottom material indicated by the dredging records.

Predicted deposition or erosion of sand along the main channel at sites 1, 2 and 3 for a range of inflows (Figure 4.34) indicate that deposition increases from site 1 to site 2, and diminishes at site 3. This is consistent with dredging records that show most sand is captured in the vicinity of site 2. The pattern at site 1 shows an interesting response to discharge. When discharge exceeds 14,156 cms (500,000 cfs), deposition at site 1 increases significantly to a level comparable to that predicted at site 2 downstream. This suggests that this portion of the river is more confined at lower stages and discharges, but becomes less confined as the delta becomes submerged during higher flows.

Sand deposition along the relatively unconfined extension of the Castille Pass beyond the mouth bar (Figure 4.35) is minor, compared to the main channel. At the 16,988 cms (600,000 cfs) inflow, the model predicts 5.5 mm of deposition in the main channel at site 2 for a two-week simulation, compared to 1.5 mm at site 4. Deposition under this high flow regime at site 4 is 27 percent of that at site 2. Minimal sand deposition, even during a high flood discharge at site 4, suggests that most of the deposition occurs before sand leaves the mouths of the distributary channels. A similar depositional pattern was observed on the western side of the delta near Log Island Pass (Figure 4.36).

Deposition of silt along the main channel increases almost linearly with discharge (Figure 4.38). Deposition of silt in the bay beyond the mouths of Castille Pass and Log Island Pass indicates that far more silt than sand reaches the Bay (Figures 4.39, 4.40).

The development of a set of curves importantly assists engineers and scientists in estimating what flow would result on a given deposition or erosion. Estimation of deposition based on these curves would provide a scientific tool to quantify the volume of deposition. The result would determine a valued depositional benefit on wetland creation or new land formation. The simplicity of procedure in using such curves engages well-documented determinations.

Sedimentation and erosion curves developed during this research predicted deposition for two-week periods under a variety of discharges. A constant input using these curves could be augmented by an additional series to show the effect of varying sediment input. These curves could then calculate deposition for a given flow and sediment concentration for a period of any length.

As seen from the curves, accumulation at the most distant bay stations (sites 6 and 9) is small for both sand and silt in discharges. For medium flood conditions (14,156 cms, 500,000 cfs), the combined deposition of sand and silt is approximately 2 mm/month. This number, about 2 cm/year, is in good agreement with the bay deposition as reported by DeLaune et al. (1987).

Development of sedimentation or erosion curves was suggested as a technique to determine the amount of accumulation for a long period of time. In the Atchafalaya River flow and sediment concentrations are highly variable. The total number of curves necessary would depend on the range of flow and sediment concentration. For example, to evaluate floods from 8,494 cms (300,000 cfs) to 19,819 cms (700,000 cfs) at the 2,831 cms (100,000 cfs) interval, five curves must be developed for one sediment concentration class. If it is assumed that five concentration classes (from 100 ppm to 500 ppm) would be sufficient to cover the sediment concentration, then a total of 25 curves must be developed.

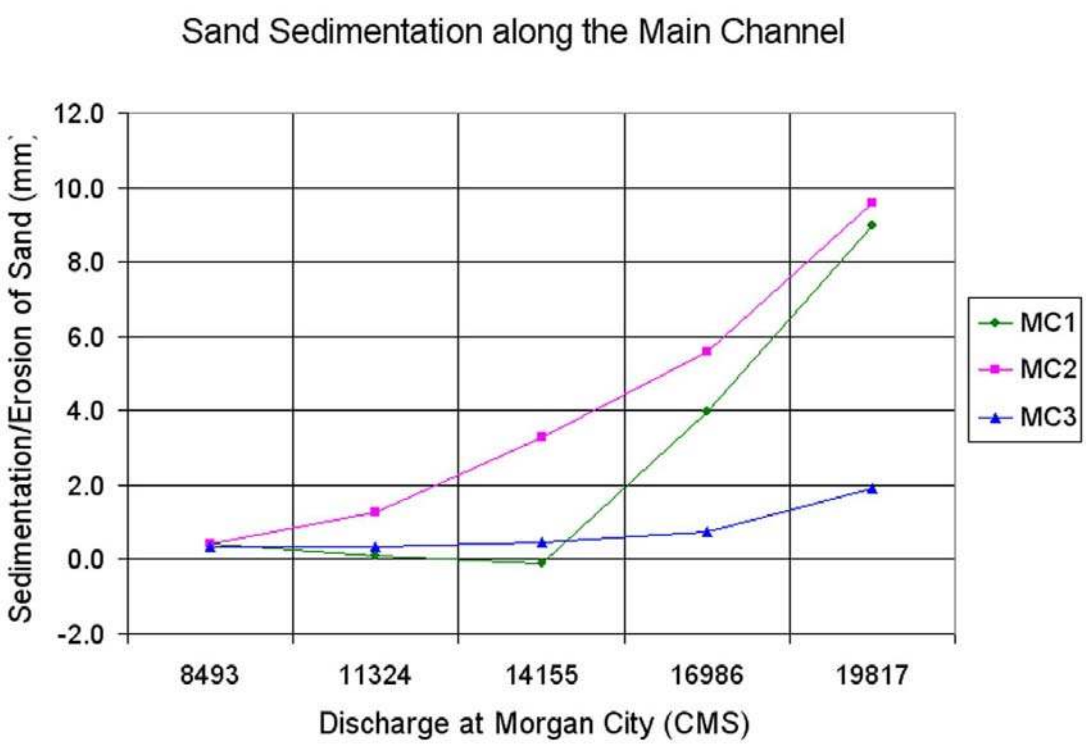


Figure 4.34. Sedimentation and erosion of sand along the main channel.

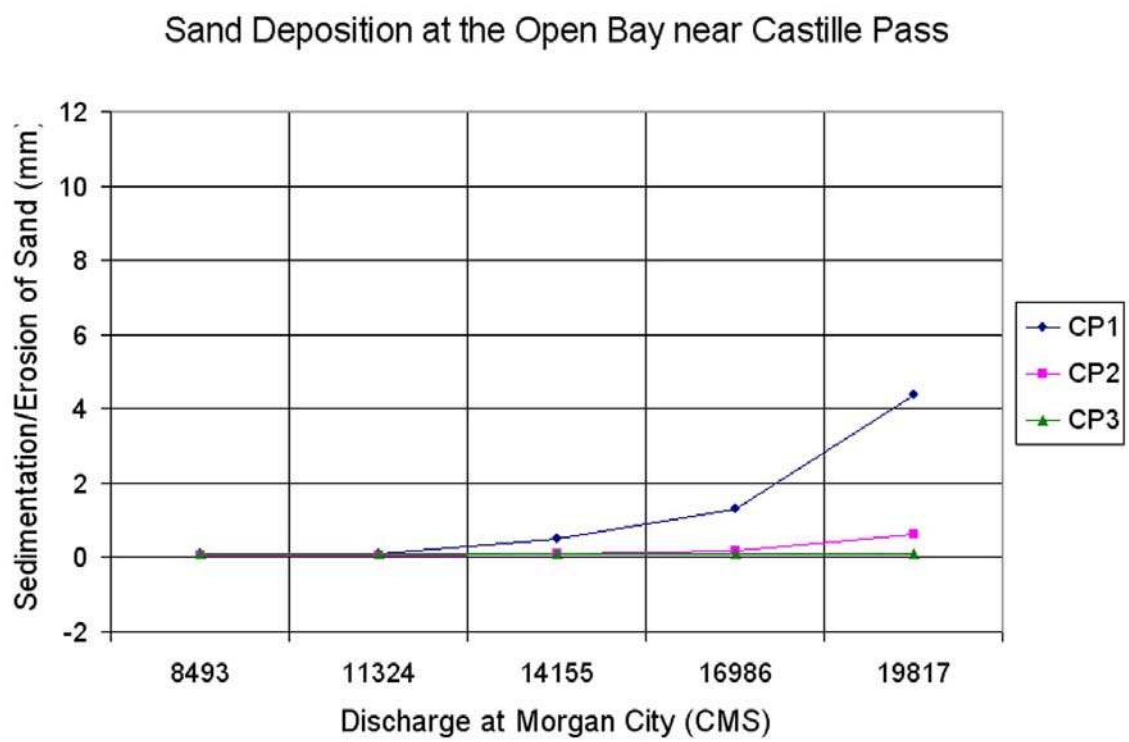


Figure 4.35. Sedimentation and erosion of sand near Castille Pass.

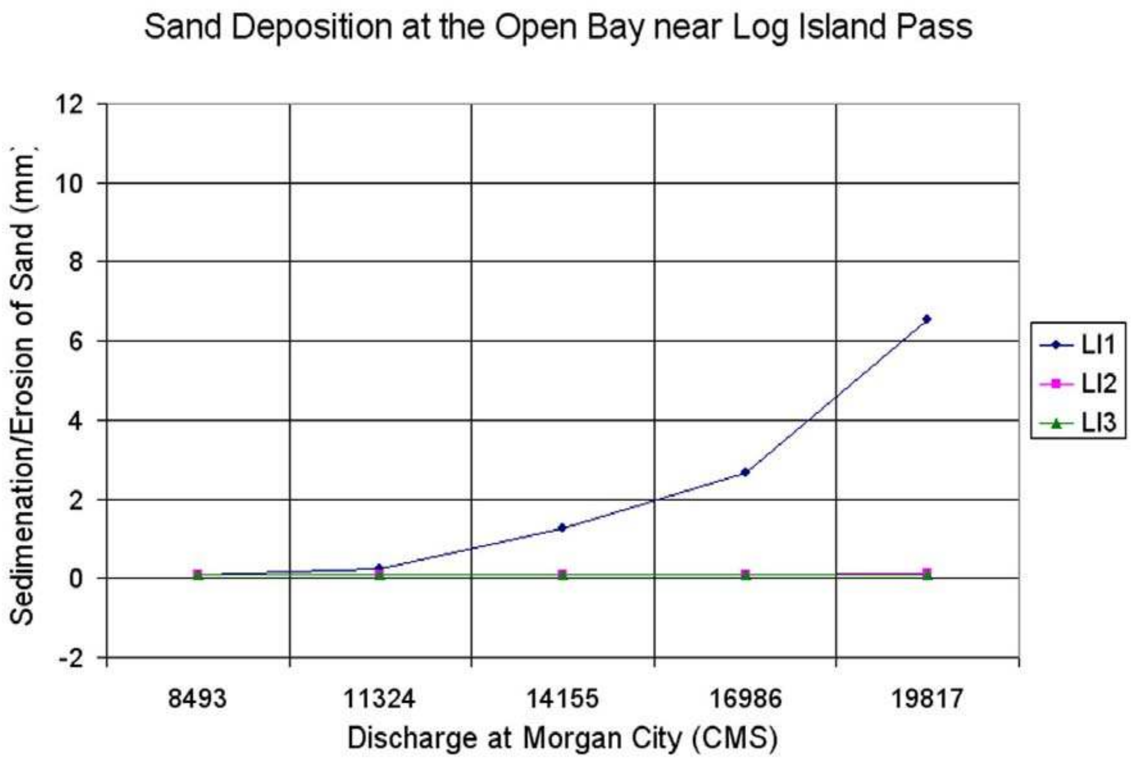


Figure 4.36. Sedimentation and erosion of sand near Log Island Pass.

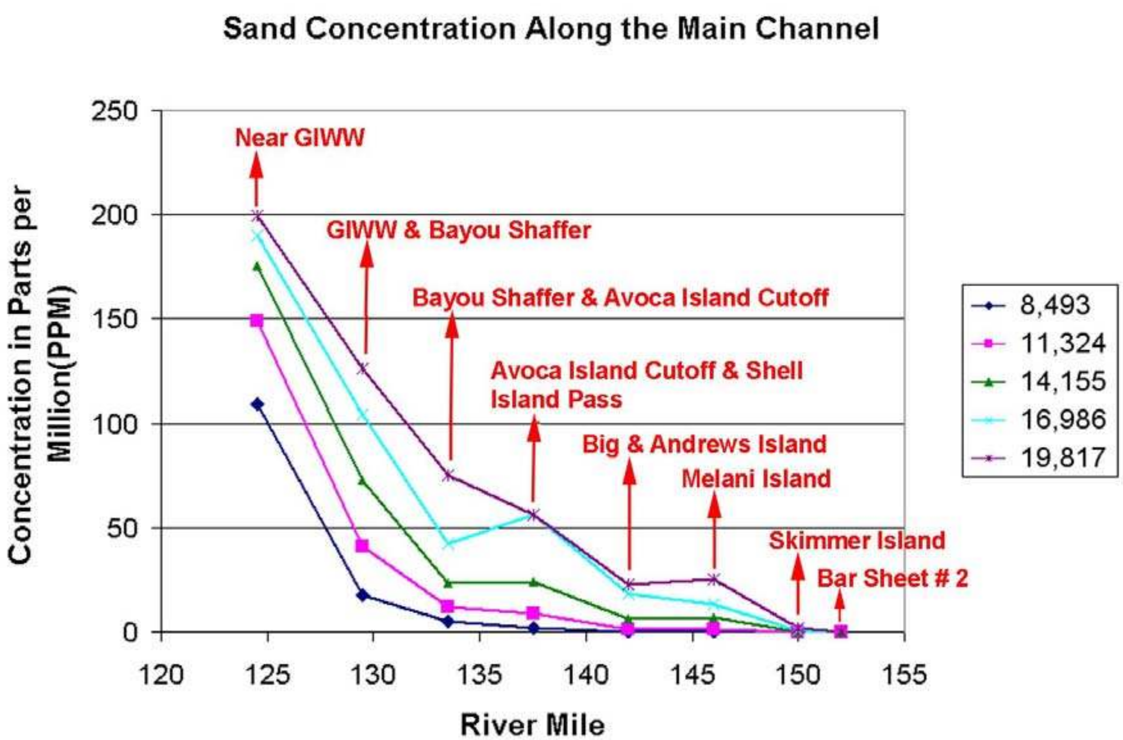


Figure 4.37. Sand concentration along the main channel.

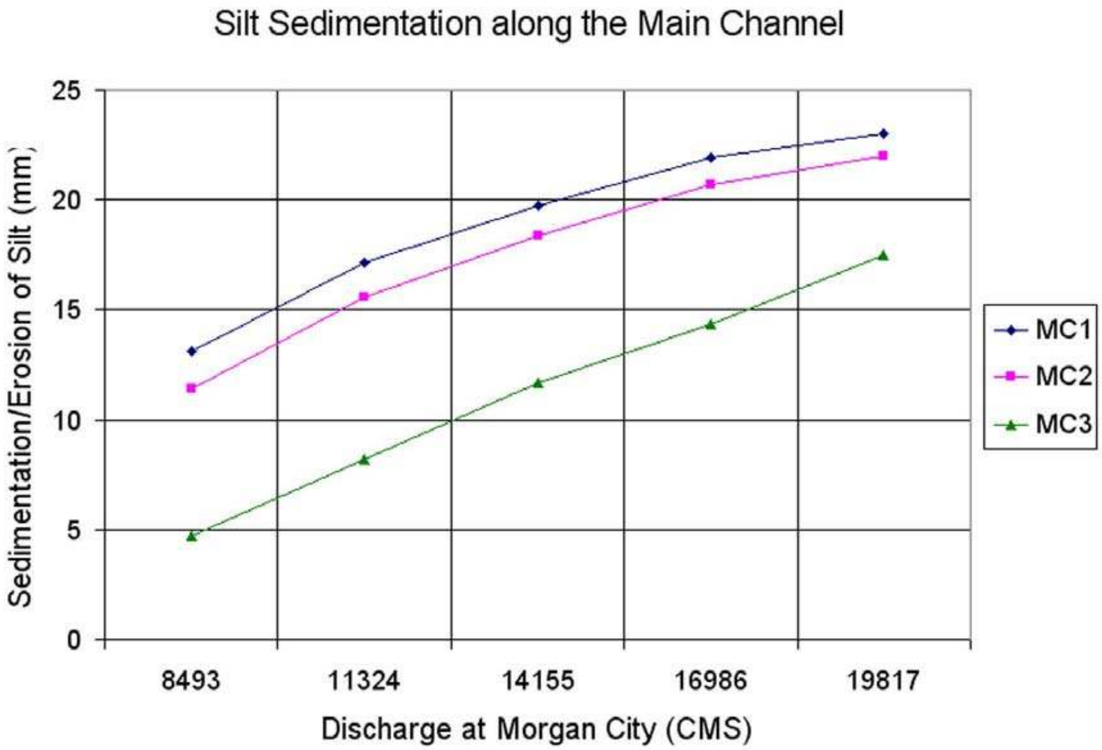


Figure 4.38. Sedimentation and erosion of silt along the main channel.

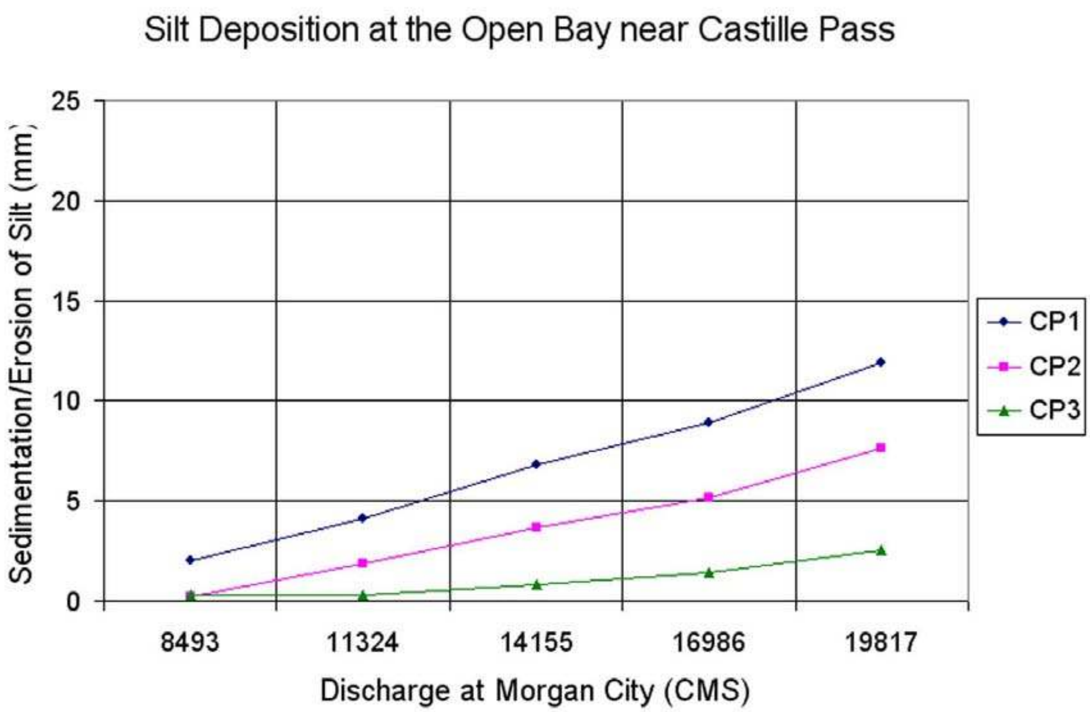


Figure 4.39. Sedimentation and erosion of silt near Castille Pass.

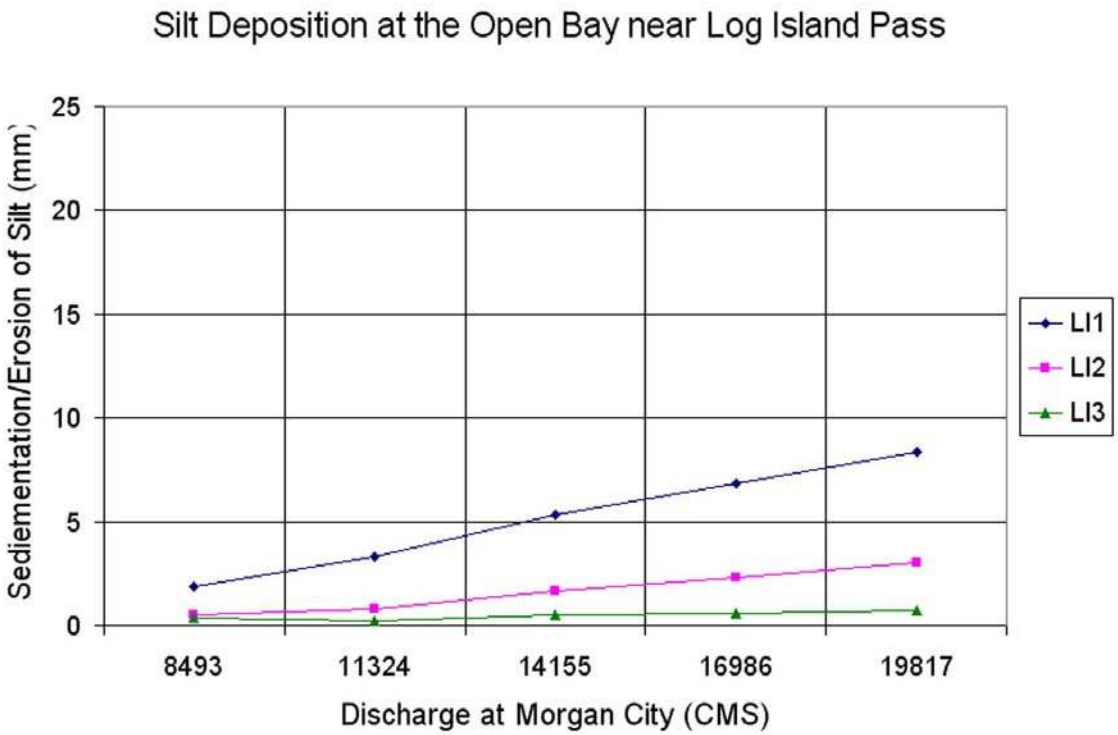


Figure 4.40. Sedimentation and erosion of silt near Log Island Pass.

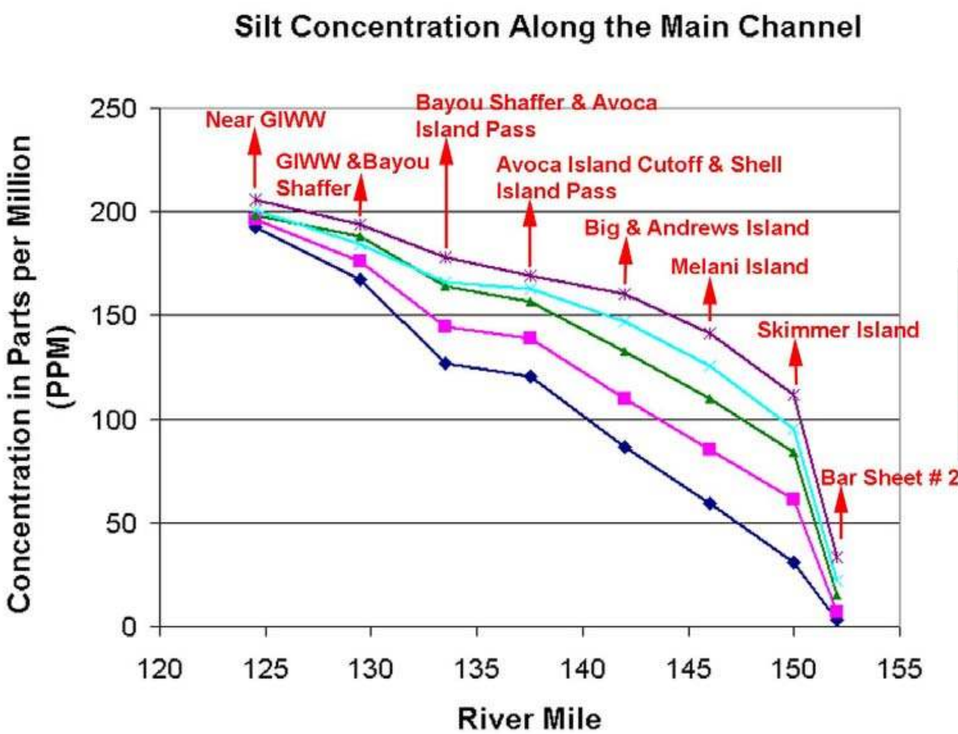


Figure 4.41. Silt concentration along the main channel.

Table 4.7. Results of the sand simulations.

			Inflow (CMS) & Simulation Time					
			w2	**w2**	**w2**	**w2**	**w2**	**w7**
Site No.	**Location**	**ID**	**8493**	**11324**	**14155**	**16986**	**19817**	**y1997**
1	Main Channel Near Big Island	MC1	0.4	0.1	-0.1	4.0	9.0	0.3
2	Main Channel near Roger Brown Island	MC2	0.4	1.3	3.3	5.6	9.6	2.8
3	Bar Area	MC3	0.3	0.4	0.5	0.7	1.9	0.6
		ID	**8493**	**11324**	**14155**	**16986**	**19817**	**y1997**
4	Open Bay area 1 Near Castille Pass	CP1	0.1	0.1	0.5	1.3	4.4	0.4
5	Open Bay area 2 Near Castille Pass	CP2	0.1	0.1	0.1	0.2	0.6	0.1
6	Open Bay area 3 Near Castille Pass	CP3	0.1	0.1	0.1	0.1	0.1	0.1
		ID	**8493**	**11324**	**14155**	**16986**	**19817**	**y1997**
7	Open Bay area 1 Near Log Island	LI1	0.1	0.2	1.3	2.7	6.5	0.6
8	Open Bay area 2 Near Log Island	LI2	0.1	0.1	0.1	0.1	0.1	0.1
9	Open Bay area 3 Near Log Island	LI3	0.1	0.1	0.1	0.1	0.1	0.1

Table 4.8. Results of the silt simulations.

			Inflow (CMS) & Simulation Time					
			w2	**w2**	**w2**	**w2**	**w2**	**w7**
Site No.	**Location**	**ID**	**8493**	**11324**	**14155**	**16986**	**19817**	**y1997**
1	Main Channel Near Big Island	MC1	13.1	17.2	19.7	21.9	23.0	66.7
2	Main Channel near Roger Brown Island	MC2	11.4	15.6	18.4	20.7	22.0	60.8
3	Bar Area	MC3	4.7	8.2	11.7	14.3	17.5	34.0
		ID	**8493**	**11324**	**14155**	**16986**	**19817**	**y1997**
4	Open Bay area 1 Near Castille Pass	CP1	2.0	4.1	6.8	8.9	11.9	15.7
5	Open Bay area 2 Near Castille Pass	CP2	0.2	1.9	3.7	5.2	7.6	7.1
6	Open Bay area 3 Near Castille Pass	CP3	0.3	0.3	0.8	1.4	2.5	1.0
		ID	**8493**	**11324**	**14155**	**16986**	**19817**	**y1997**
7	Open Bay area 1 Near Log Island	LI1	1.9	3.3	5.3	6.9	8.3	13.3
8	Open Bay area 2 Near Log Island	LI2	0.5	0.8	1.6	2.3	3.0	3.1
9	Open Bay area 3 Near Log Island	LI3	0.3	0.2	0.5	0.6	0.7	0.7

Table 4.9. Comparisons of sedimentation based on two-week average peak flood from the curve with the simulated sedimentation for the 1997 flood.

SAND				Graph-Model (mm)
Location	Deposition from graph (mm)	Deposition from model (mm)	% Difference	Difference
MC1	0.4	0.451	-11	-0.05
MC2	3.4	2.894	17	0.51
MC3	0.8	0.568	41	0.23
CP1	0.1	0.356	-72	-0.26
LI1	0.5	0.569	-12	-0.07
SILT				Graph-Model (mm)
Location	Deposition from graph (mm)	Deposition from model (mm)	% Difference	Difference
MC1	64.5	56.58	14	7.92
MC2	58.4	51.51	13	6.89
MC3	30.1	28.55	5	1.55
CP1	14.8	13.15	13	1.65
LI1	12.8	11.15	15	1.65

Table 4.10. Composition of Atchafalaya Dredge Materials.

Location	Observed Data (Average)		
	% Sand	% Silt	% Clay
Bay area	79	20	0
Bar area	3	53	44

CHAPTER 5. CONCLUSIONS AND FUTURE WORK

This research showed that a standard engineering modeling tool could be used to quantitatively simulate sediment transport and deposition in a system as complex as the Atchafalaya River delta. This model satisfactorily reproduced the formation of a river mouth bar, natural levees, and observed rates of deposition and scour within confined channels and adjacent open bay areas.

Results demonstrated that numerical models could be used to simulate the inception phase of distributary channel formation, bifurcation and elongation. If flow and sediment are abundant, a sub-aqueous deltaic deposit starts to form at the mouth of the feeder channel. As the simulation continues, a more prominent distributary channel and sub-aqueous levee begins to appear. If the model is changed to accelerate sub-aerial feature development, new distributary channels begin to branch off.

Model simulation affirmed that an efficient sediment ramp or adverse slope could be designed for a distributary channel to divert sediments efficiently to the bay. Simulations that began with a test slope of 1V to 51H evolved to form a much milder slope, close to 1V to 412H and similar to the natural slopes observed in the Wax Lake Outlet delta. Adverse slopes at the Wax Lake Outlet delta ranged from 1V to 340H to 1V:850H, with 1V to 543H as the average. The model further showed that an equilibrium slope does not depend on the concentration of the sediment inflow.

Data from the sedimentation/erosion curves suggested that there is a threshold discharge for the transport of coarse sediments (sand) in the bay. If the discharge were less than 11,325 cms (400,000 cfs) at Morgan City, most of the sand would not be transported to the bay. The model also showed that there is very little transport of coarse sediments (sand) beyond 1000 meters from the mouth of the feeder channel, even with high discharge.

Graphic representation of river deposition or erosion may be accomplished by developing a set of curves, which could be used at a later time for engineering purposes to estimate long-term deposition or erosion. This technique will importantly reduce the need for model simulations, thereby providing a sound technological means to economically design new projects.

In addition, the developed technique could be applied to existing freshwater diversions such Davis Pond, or Caernarvon diversion projects, to determine sediment deposition or erosion. The concept could also determine the potential benefits of newly conceived projects, or those projects designated to build land for coastal Louisiana.

Results of this research revealed that a two-dimensional, vertically-averaged sediment model could be used successfully in the Atchafalaya River and the bay, where river processes dominate the mechanisms of sediment transport and delta formation. Fluvial characteristics of high flow, strong circulation and mixing, as well as the presence of a shallow bay between the river mouth and the continental shelf, combine to form a system that is predominantly a freshwater-dominated river mouth (van Heerden, 1980 &1983).

Discharge and sand concentrations in the Atchafalaya River are seasonal. High flow and high sand concentration are observed during winter and spring, while low flow and low sediment concentration are observed during summer and fall. Peak discharge and peak concentration, however, do not occur simultaneously. An average peak sediment concentration occurs roughly 27 days before peak discharge. Accuracy of the model simulations largely depends on the capture of the temporal variations of discharge and sediment concentrations in determining the rate of deposition or erosion.

Data reflecting direct deposition or erosion was unavailable to compare with model results in this study. In future studies, cross-section and profile surveys in the main channel, the bay, and distributary channels could be repeated periodically at key locations. The resulting data could indicate adjustments in frequency of the survey, and validate the model by observed deposition or erosion.

Continuous velocity and discharge measurements could be used for future hydrodynamic model calibration and validation. Calibration of the hydrodynamic model, applying continuous velocity and discharge, would indicate whether it improves the accuracy of sediment transport model results over models calibrated using primarily tidal data.

Developing sound and feasible engineering approaches to building new wetlands and nourishing deteriorating coastal ecosystems requires a new generation of capable and sensitive predictive tools, and a larger cadre of trained engineers. Engineers who seek to use these predictive tools will enable the resulting work to proceed in a manner that builds confidence, causing no societal consequences or costs. The wide-spread availability of low-cost, high-speed desktop computers now makes it practical for engineers to address ecosystem problems through the application of complex numerical models. These models, originally developed as research tools, provide new, exciting, and malleable solutions to real-world river management problems.

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