Two-dimensional numerical modeling of a proposed freshwater diversion from the Bonnet Carre Spillway to the Labranche wetlands

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TWO-DIMENSIONAL NUMERICAL MODELING OF A PROPOSED FRESHWATER DIVERSION FROM THE BONNET CARRE SPILLWAY TO THE LABRANCHE WETLANDS

A Thesis

Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering

in

The Department of Civil and Environmental Engineering

By
Josef Hoffmann
B.S., Louisiana State University, 2009
August 2011
DEDICATION

To my mother – for her care and support, and whose priority was always the formal education of her children.

To all of my previous instructors and fellow students - to whom I owe, in large part, my great love for engineering.

And to my father - to whom I owe, in large part, my love for the Great Engineer.
ACKNOWLEDGEMENTS

I must first of all express my immense gratitude to my advisor and mentor, Dr. Clint Willson. He taught me, above all, the paramount importance of creativity in the field of engineering and patiently directed my efforts down what was indeed a most rewarding path. I cannot thank him enough for his time and assistance. Yet, my actual completion of that path positively could not have been accomplished without the continual assistance of PhD student Erol Karadogan. His expertise with the ADH code and his generosity in transmitting that knowledge made the daunting task of finite element modeling a less overwhelming one. It has truly been a privilege to work with and learn from such fine engineers.

I would also like to thank the Office of Coastal Protection and Restoration (OCPR) and the LSU Coastal Sustainability Studio (CSS) for funding the work that went into this thesis. Particularly, I would like to thank Jeff Carney at the CSS, for whom I had the pleasure of working for an entire semester. I owe a large debt of gratitude to the members of my advisory committee, Dr. Clint Willson, Dr. Heather Smith, and Dr. John Day, Jr. for their time and expertise. I cannot thank them enough for their guidance and vision during this entire process.

I must acknowledge the immense help that was so freely given to me by the people at many different agencies and consulting groups. I would like to personally thank Brian Vosburg at OCPR, Scott Perrien at the U.S. Geological Survey, John Jurgensen and Jason Kroll at the U.S. Department of Agriculture, Ed Fike at Coastal Environments, Inc., and Mr. Allan Ensminger at the Wetlands and Wildlife Management Co. for their assistance in procuring the data necessary for this modeling effort. I would also like to
thank the LSU Center for Computation & Technology (CCT) and Information Technology Services (ITS) for providing the high performance computing resources that were so heavily used in this modeling work.

Finally, I would like to acknowledge some of the previous work that made this thesis possible. The particle-tracking code developed by Nathan Dill (and modified by Erol Karadogan for compatibility with ADH) was used extensively in the post-processing and visualization of the ADH model results. The research and field work performed by John Day and Rachael Hunter in the Labranche wetlands were the inspiration for this thesis, and their insights on the region were instrumental in moving this work forward.
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ABSTRACT

Engineered modifications to the lower Mississippi River within the past century have limited the magnitude and frequency of flood events in wetlands along the Louisiana coast. Without this natural delivery of freshwater, sediment, and nutrients, the ecological health of these wetlands are now degrading. A recent study within the Bonnet Carre Spillway has revealed that the 7,623 acres of floodway between the Mississippi River and Lake Pontchartrain is one of the few areas in Louisiana that is actively accreting land as a result of pulsed sediment-laden freshwater input during high discharge events in the Mississippi River. On the other hand, the productivity of the spillway region is geographically juxtaposed to the deterioration of the Labranche wetlands directly to the east, which have lost an extensive amount of marsh and swamp land to open water since becoming hydrologically disconnected from the river in the 1930’s. A two-dimensional finite-element numerical model of the Mississippi River, Bonnet Carre Spillway, Lake Pontchartrain, and Labranche wetlands is presented, which is used to examine the hydrodynamics of a freshwater input in the Labranche wetlands via a hypothetical diversion channel through the eastern guide levee of the Bonnet Carre Spillway. Flow velocities, water distribution patterns, and residence time distributions are used to highlight the potential for reintroducing river water and resources to these degrading wetlands.
CHAPTER 1. INTRODUCTION

1.1 Coastal Land Loss and River Diversions

Since the early twentieth century, extensive flood-control engineering has confined the lower ~ 300 miles of the Mississippi River between artificial levees and effectively disconnected the river from the coastal wetlands which had historically received the benefits of nutrient and sediment delivery that accompanied periodic flooding. Threatened with factors such as sea-level rise, subsidence, and salinity intrusion, these wetlands now lack the riverine input that would help offset these detriments to their productivity. Projections based on land loss trends since the 1950’s estimate that over 20,000 square miles of land in coastal Louisiana will have been inundated between the years 1956 and 2050 (Barras, et al., 2004).

Many researchers (Allison & Meselhe, 2010; Day, et al., 2007) agree that replication of natural pulsed flooding events in coastal wetlands can be accomplished in a controlled fashion through river diversions, that is, constructed channels that direct freshwater and its resources out of a river to a desired location. River diversions can be built to achieve a variety of objectives, such as flood control, freshwater delivery, nutrient delivery, or sediment delivery to an ecosystem. The primary objective of a river diversion governs its location, structure type, discharge capacity, and the frequency of its operation. Due to the high discharges required to transport large volumes of sand, sediment diversions must necessarily be capable of conveying large portions of river flow. In the lower Mississippi River, the two largest diversions are the Old River Control Structure and the Bonnet Carre Spillway – both of which are operated as flood-control diversions. However, their abilities to act as sediment diversions have been shown in studies of the
Atchafalaya River (Hupp, et al., 2008), Wax Lake Delta (Roberts, et al., 1997), Bonnet Carre Spillway and Lake Pontchartrain (Day, et al., 2010; White, et al., 2009), which have concluded that these receiving areas of large-scale Mississippi River diversions are some of the only locations in coastal Louisiana that are actively accreting land and that their land-building productivity is due in large part to Mississippi River input.

Studies such as Martin (2002), through emergy analysis, have shown the combined ecological and economic benefits of river diversions. However, the manpower, money, and legal authorization required to construct large sediment diversions are a few of the many obstacles that oppose land-building projects of this nature. Furthermore, the suspended sediment flux in the lower reach of the Mississippi River may have declined by as much as 70% since the 1850’s (Kesel, 1988), raising questions as to whether the current estimated sediment budget of the lower Mississippi River justifies the cost of large sediment diversion projects (Nittrouer, et al., 2008). With the many uncertainties that exist in the planning and design of river diversions, the use of numerical models to simulate the proposed infrastructure and their complex interactions with the river has a distinct place in the engineering profession.

1.2 Bonnet Carre Spillway and Labranche Wetland

The Bonnet Carre Spillway, located on the east bank of the Mississippi River at river-mile 127.1 (RM 127.1, referenced from Head of Passes) is one of the most significant assets within the Mississippi River basin for the management of the lower River. Congressionally authorized by the Flood Control Act of 1928, the spillway was designed with the capacity to divert 250,000 cubic feet per second (cfs) from the Mississippi River to Lake Pontchartrain to lower flood stages in the river and protect the
people of New Orleans (USACE, New Orleans District, 2009). Following the devastating Flood of 1927, the United States Army Corps of Engineers (USACE) undertook a slew of public works projects which collectively comprise the Mississippi River and Tributaries Project. The goal of this large scale effort was to protect inhabitants along the Mississippi River from a project flood of nearly three million cfs at the headwaters of Old River (McPhee, 1987). Designed to work in conjunction with other flood control measures (Old River Control and the Morganza Spillway) in the event of high-magnitude flow events, Bonnet Carre’s primary design objective was to act as the lowermost phase of a coordinated effort to reduce the project flood to a discharge that could safely pass New Orleans. That is, at its design capacity, Bonnet Carre was built to divert over 15% of the portion of the project flood that would pass the spillway. To this end, operation of the spillway has been regulated since its completion in 1931 by flood stages at the USACE river gage at Carrollton (RM 102.8). When the water level at Carrollton is predicted to reach 20 feet above the National Geodetic Vertical Datum of 1929 (NGVD29), the spillway is opened until the threat of high water at Carrollton has passed. The Bonnet Carre spillway has been operated in this fashion a total of ten times since its construction in response to the floodwaters which pulse down the Mississippi once a decade on average. Dates of the spillway openings were as follows: 1937, 1945, 1950, 1973, 1975, 1979, 1983, 1997, 2008, and 2011. For each of these openings, the spillway was in use for an average of approximately 41 days (Table 1.1). An additional experimental opening of the spillway was staged in 1994 for 42 days, during which time data was collected within the spillway and Lake Pontchartrain to track the movement of nutrients and sediments diverted from the Mississippi River (Lane et al., 2001).
### Table 1.1: Bonnet Carre Spillway Openings

<table>
<thead>
<tr>
<th>Dates Open</th>
<th>Bays Opened</th>
<th>Peak Discharge (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January 30 to March 7, 1937</td>
<td>285</td>
<td>211,000</td>
</tr>
<tr>
<td>March 23 to May 18, 1945</td>
<td>350</td>
<td>318,000</td>
</tr>
<tr>
<td>February 10 to March 19, 1950</td>
<td>350</td>
<td>223,000</td>
</tr>
<tr>
<td>April 8 to June 21, 1973</td>
<td>350</td>
<td>195,000</td>
</tr>
<tr>
<td>April 14 to April 26, 1975</td>
<td>225</td>
<td>110,000</td>
</tr>
<tr>
<td>April 18 to May 21, 1979</td>
<td>350</td>
<td>191,000</td>
</tr>
<tr>
<td>May 20 to June 3, 1983</td>
<td>350</td>
<td>268,000</td>
</tr>
<tr>
<td>March 17 to April 16, 1997</td>
<td>298</td>
<td>243,000</td>
</tr>
<tr>
<td>April 12 to May 7, 2008</td>
<td>160</td>
<td>169,300</td>
</tr>
<tr>
<td>May 9 to June 20, 2011</td>
<td>330</td>
<td>316,000</td>
</tr>
</tbody>
</table>

Approximately six miles upstream of the Bonnet Carre Spillway, on the outside of a nearly ninety-degree bend in the Mississippi River, is the site of what was once the Bonnet Carre Crevasse – to which the spillway owes its name. The resemblance of the river at this tight bend to the shape of a woman’s headpiece is undoubtedly the reason for the name Bonnet Carre, literally translated “square bonnet.” At this location, numerous crevasse splays were recorded as early as the mid-nineteenth century (Kesel, 1988). Crevasse splays are concentrated discharges through breaks in natural or artificial levees which occur during high flood stages when overbank flow has the stream power to create such an opening. Between the years 1849 and 1874, the Bonnet Carre Crevasse remained undammed through a series of five large flood events, which collectively deposited approximately 229-306 million cubic meters of sediment in the subaerial crevasse region and Lake Pontchartrain (Kesel, 1988). In terms of the estimated sediment budget of the Mississippi River between 1851 and 1893 (Kesel, 1988; Meade & Moody, 2009), the average volume of sediment that passed through the Bonnet Carre Crevasse during each flood event between 1849 and 1874 comprised approximately thirty to forty percent of the total suspended load that travelled down the lower part of the river annually. It is estimated that the discharges through the Bonnet Carre Crevasse during each of these
Events reached approximately 150,000 cfs (Davis, 2000). Due to the high discharges associated with crevasse splays and the sediment-carrying capacity of these flows, the resulting landforms created from crevasse deposits comprise what is known as “Crevasse Topography” (Howard & Penfound, 1942) and may remain as defined geologic features for periods as long as several hundred years (Kesel, 1988).

Like the topography of the Crevasse region upstream, the area between the confining levees of the Bonnet Carre Spillway reflects a history of relatively frequent interaction with Mississippi River freshwater and its nutrient/sediment constituents. During times of spillway operation, a 7,623-acre floodway conveys diverted freshwater from the Mississippi River to Lake Pontchartrain via five-and-a-half miles of artificial guide levees (USACE, 2009). During these times, river-borne sands and silts entrained in the diverted floodwaters are deposited in the floodway and are partly responsible for what Day, et al. (2010) determined to be an average vertical accretion rate of 2.6 to 2.7 centimeters per year since 1953. Additionally, freshwater often enters the floodway region when high river stages force leaks through the timber needles that dam the spillway, and the region receives the ecological benefit of this supplemental hydrologic input. Figure 1.1 shows a rating curve of the estimated “leakage” flow through the spillway structure as a function of the river stage recorded at the USACE river gage at Bonnet Carre (Gage ID 01280, RM 127.1).

Ground elevations between the guide levees oscillate irregularly but generally slope from 3.25 meters above the North American Vertical Datum of 1988 (NAVD 88) on the river-side to 0.25 meters on the lake-side of the floodway. Vegetation within the spillway is characterized primarily by forested land cover (bottomland hardwood forest
and baldcypress-tupelo swamp) and various aquatic and non-aquatic plant types (USACE, New Orleans District, 2009).

Figure 1.1: Spillway Leakage Curve Obtained from the U.S. Army Corps of Engineers, New Orleans District

Bordering the spillway to the east are the Labranche wetlands, approximately 20,000 acres of fragmented baldcypress swamp and freshwater marsh. The confining boundaries of the Labranche wetlands consist of artificial levees to the west (bordering the Bonnet Carre Spillway), east (bordering the city of Kenner), and south, and Lake Pontchartrain to the north. Comparison of aerial photos of the Labranche wetlands from the mid-twentieth century and the present shows that much previously visible land has been converted to open water over the past 60 years. Historical research has revealed that specifically between the years 1952 and 1983, the amount of Labranche wetland area submerged in open water increased nearly sixteen-fold (Day, et al., 2010). Local subsidence, eustatic sea level rise, salinity intrusion, transportation development, consumption and destruction of vegetation by nutria, and hurricane damage may all have significantly contributed to the degradation of the Labranche wetlands.
A study comparing ground elevations, long-term vertical accretion rates, and tree ring growth in the Bonnet Carre Spillway and Labranche wetlands concluded that the spillway region is a healthy sustainable ecosystem, while the adjacent Labranche wetland is deteriorating (Day, et al., 2010). The primary difference between these regions is that the Bonnet Carre Spillway receives periodic flow from the Mississippi River, while the Labranche wetlands are completely disconnected from any riverine input. For this reason, many restoration alternatives have been proposed by different groups and agencies, which are based on pulsed inputs of Mississippi River freshwater and resources to the Labranche wetland.

Figure 1.2 shows a 2008 aerial image of the Bonnet Carre Spillway, Labranche wetland, and the notable features of the surrounding area. To illustrate the topography that was previously described in the spillway and the wetland, color-contoured Lidar elevations of the region obtained from the LSU Atlas website (www.atlas.lsu.edu) are shown in Figure 1.3.

![Figure 1.2: 2008 Aerial Imagery of the Bonnet Carre / Labranche Region](image-url)
1.3 Objective and Scope

For this thesis, the two-dimensional finite element code ADH is used to simulate the introduction of Mississippi River freshwater into the Labranche wetland. This freshwater delivery is accomplished through a hypothetical, but proposed, diversion channel running from the Bonnet Carre Spillway to the western side of the wetland. Recognizing that the wetland hydraulics in this diversion scenario are directly related to the hydrodynamic conditions in the Mississippi River and the operation of the Bonnet Carre Spillway, a holistic modeling approach is taken that encompasses the combined Mississippi River / Bonnet Carre Spillway / Labranche wetland / Lake Pontchartrain system. To this end, the first objective of this thesis work is to calibrate and validate a numerical model that captures the hydrodynamics in the Bonnet Carre Spillway and the adjacent reach of the Mississippi River. The second objective is to use this model to
simulate the introduction of freshwater from the Bonnet Carre Spillway into the Labranche wetland during flow events of various magnitudes in the spillway.

The model presented herein includes the Mississippi River between Reserve, LA (RM 138.8) and Carrollton, LA (RM 102.8), the Bonnet Carre Spillway, the Labranche wetland, and the southwestern portion of Lake Pontchartrain. Results of the model calibration under existing conditions are shown, by which appropriate values for bottom friction and coefficient of eddy viscosity within the model domain are selected. The use of an optional model correction for 3D bendway vorticity is also tested during the calibration simulations, and the results are shown. Results of validation simulations are presented to show model’s ability to reproduce measured hydrologic events using the physical parameters obtained from the calibration simulations. Finally, results of flow simulations from the Bonnet Carre Spillway into the Labranche wetland via a proposed diversion channel are presented, and a particle tracking model is used to better visualize the ADH output and examine the water flow paths, flow distribution, and residence times. Flow velocities, water distribution patterns, and residence time distributions are used to highlight the potential for reintroducing river water and resources to these degrading wetlands.
CHAPTER 2. BACKGROUND

2.1 Lake Pontchartrain: Effects of Spillway Discharge

Lake Pontchartrain was formed approximately 2000-4000 years ago, when the St. Bernard delta lobe of the Mississippi River deposited the alluvium that currently constitutes its southern and eastern boundaries. This is the same delta lobe responsible for building the land beneath the city of New Orleans, the Chandeleur Islands, and Breton Sound. The land barrier formed by the St. Bernard lobe, which partially encloses Lake Pontchartrain to the east, served to separate it from the higher salinity regimes of the Gulf of Mexico and formed the misnomered “lake” into a brackish estuary. Tidal exchange in Lake Pontchartrain occurs through the Rigolets, Chef Menteur Pass, and the New Orleans Inner Harbor Navigation Canal. The salinity levels associated with these tidal fluxes are normally tempered by the freshwater contributed by the Pontchartrain Basin, of which the Amite and Bogue Chitto Rivers are the primary tributaries. Consequently, the mean salinity in Lake Pontchartrain is 3.9 parts per thousand (ppt). However, due to vertical stratification, bottom salinity can be greater than surface salinity by as much as 10 ppt (Shaffer, et al., 2009).

With data collected during the 2008 Bonnet Carre Spillway opening, White, et al. (2009) performed a study assessing the physical, chemical, and biological effects of spillway discharge on the Lake Pontchartrain estuary. River discharges upstream of the spillway during the 2008 flood crested at 1,456,000 cfs on 04/28/08 at Tarbert Landing (USACE gage 01100 – RM 306.3), and the peak flow rate measured in the Bonnet Carre Spillway during the flood was 169,300 cfs. During its thirty-one days of operation, the Bonnet Carre Spillway delivered approximately 6.5 million acre-feet of Mississippi River...
freshwater to Lake Pontchartrain. This volume exceeds the total storage capacity of Lake Pontchartrain by roughly a factor of 1.3, yet the freshwater discharged through the spillway during this time only mixed with less than 40% of the brackish water in the estuary. The remainder of the freshwater exited Lake Pontchartrain by flowing along the south shore through the Rigolets. The flow patterns and mixing levels given by White, et al. were determined through the analysis of satellite imagery (Figure 2.1) and water quality samples from both inside and outside of the freshwater plume in Lake Pontchartrain during the time when the spillway was open.

![Figure 2.1: 04/28/08 Aerial Image (Left) and 04/29/08 MODIS Image (Right) of Freshwater Plume in Lake Pontchartrain (White, et al., 2009)](image)

At the time of the images shown in Figure 2.1, discharge in the spillway was measured at 160,000 cfs, or 64% of its 250,000 cfs design capacity. The right-hand image in the figure was captured by the Moderate Resolution Imaging Spectrometer, a NASA instrument orbiting the earth at 705 km on board the Terra and Aqua satellites, designed to image large-scale global processes on the earth’s surface. In both images the horizontal stratification of freshwater and brackish in Lake Pontchartrain is evident, showing that the highest concentrations of freshwater are confined very near to the south shore during this spillway opening. The MODIS image also shows the location of ten sampling
stations (red) where water quality data was collected to quantify the impact of the freshwater discharge on Lake Pontchartrain at different locations. The data from these measurements are summarized in Table 2.1 (average measurement ± one standard deviation). Data prefaced by “Top” were collected at ten centimeters below the water surface, while “Mid” measurements were taken at mid-depth.

**Table 2.1: Water Quality Measurements at Plume and Lake Stations on April 29 and May 5, 2008 (Reproduced from White, et al., 2009)**

<table>
<thead>
<tr>
<th>Water Quality Measure</th>
<th>Within Plume (n=12)</th>
<th>Within Lake (n=6)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Temperature (°C)</strong></td>
<td>Top 19.3 ± 1.3</td>
<td>Top 22.9 ± 1.2</td>
</tr>
<tr>
<td></td>
<td>Mid 19.7 ± 0.4</td>
<td>Mid 22.9 ± 0.5</td>
</tr>
<tr>
<td><strong>Salinity (g/kg)</strong></td>
<td>Top 0.17 ± 0.02</td>
<td>Top 2.9 ± 0.20</td>
</tr>
<tr>
<td></td>
<td>Mid 0.18 ± 0.01</td>
<td>Mid 2.6 ± 0.77</td>
</tr>
<tr>
<td><strong>Dissolved O₂ (mg/L)</strong></td>
<td>Top 8.93 ± 0.34</td>
<td>Top 9.93 ± 0.28</td>
</tr>
<tr>
<td></td>
<td>Mid 8.26 ± 0.33</td>
<td>Mid 8.38 ± 0.88</td>
</tr>
<tr>
<td><strong>pH (pH units)</strong></td>
<td>Top 7.7 ± 0.8</td>
<td>Top 7.3 ± 1.4</td>
</tr>
<tr>
<td></td>
<td>Mid 7.2 ± 1.5</td>
<td>Mid 7.1 ± 1.5</td>
</tr>
<tr>
<td><strong>Nitrate (mg-N/L)</strong></td>
<td>Top 1.3 ± 0.074</td>
<td>Top 0.272 ± 0.086</td>
</tr>
<tr>
<td></td>
<td>Mid 1.4 ± 0.071</td>
<td>Mid 0.239 ± 0.016</td>
</tr>
<tr>
<td><strong>Ammonium (mg-N/L)</strong></td>
<td>0.03 ± 0.013</td>
<td>0.040 ± 0.010</td>
</tr>
<tr>
<td><strong>Dissolved Reactive Phosphorus (mg-P/L)</strong></td>
<td>0.051 ± 0.013</td>
<td>0.010 ± 0.005</td>
</tr>
<tr>
<td><strong>Silica (mg-Si/L)</strong></td>
<td>2.58 ± 0.28</td>
<td>0.80 ± 0.35</td>
</tr>
<tr>
<td><strong>Total Suspended Solids (mg/L)</strong></td>
<td>45.7 ± 12.8</td>
<td>17.8 ± 3.76</td>
</tr>
<tr>
<td><strong>Chlorophyll (µg/L)</strong></td>
<td>6.52 ± 4.01</td>
<td>29.2 ± 1.80</td>
</tr>
</tbody>
</table>

The results of this study show that the freshwater plume that exited the Bonnet Carre Spillway in 2008 contained much higher nutrient concentrations and much lower salinity concentrations than the unmixed portion of Lake Pontchartrain. While a lack of similar data for other spillway openings inhibits the assurance that the results of this study are representative of spillway operation in general, this data does provide some knowledge of what the nutrient and salinity loads might be for a freshwater input from the Bonnet Carre Spillway into the Labranche wetland.
2.3 Numerical Modeling in the Lower Mississippi River

Previous work has been done in the field of numerical modeling of both hydrodynamic and sediment-transport processes in the lower Mississippi River, and it is ongoing yet. Pereira, et al. (2009) calibrated and validated a quasi-unsteady one-dimensional mobile bed HEC-RAS model to simulate the hydrodynamics and erosion/deposition patterns of suspended sand in the reach of the Mississippi River between Tarbert Landing (RM 306.2) and Venice (RM 11). Davis (2010) extended the Pereira, et al. model downstream to include Head of Passes and analyzed the effect of several proposed freshwater diversions on the hydrodynamics of the Mississippi River with HEC-RAS 4.0. Dill (2007) compared two popular two-dimensional finite element models, ADCIRC and RMA2, to assess the hydrodynamic effects of a hypothetical freshwater diversion from the Mississippi River to a receiving area near Empire, LA. Rego, et al. (2010) used the three-dimensional numerical model H3D to simulate the hydrodynamics, salinity circulation, and transport of fine sands in the combined Mississippi-Atchafalaya river systems.

More specifically, precedents exist in previous modeling efforts for the use of ADH in the Lower Mississippi River. Brown, et al. (2009) used ADH in conjunction with HEC-RAS and the three-dimensional model CH3D-SED to assess the impact of the West Bay sediment diversion on the erosion and deposition patterns in the Lower Mississippi River. Karadogan and Willson (2011) calibrated and validated a two-dimensional hydrodynamic model of the Mississippi River between Carrollton, LA (RM 102.8) and the Gulf of Mexico using ADH. This model was then used to evaluate the relative
impacts of upstream river discharge and sea level rise on freshwater fluxes through the main channel and passes in the lower river.

Numerical hydrodynamic modeling of the Bonnet Carre Spillway control structure was performed by URS Corporation in 2009 as part of a feasibility study for a container facility along the Mississippi River directly in front of the spillway. The two-dimensional depth-averaged finite volume model CMS was used to assess the potential alterations in spillway intake due to the construction of the container facility. The model domain consisted of the Mississippi River between Reserve, LA (RM 138.8) and Carrollton, LA (RM 102.8), the Bonnet Carre control structure, and a portion of the Bonnet Carre floodway. Notable among their findings were the results of their 04/22/08 calibration simulation, during which time 160 of the Bonnet Carre Spillway bays were open and 169,221 cfs entered the spillway (Figure 2.2).

Figure 2.2: CMS Model Results of Flow Distribution at the Bonnet Carre Spillway During the 2008 Flood (URS Corporation, 2009)
The bottom of Figure 2.2 shows the cross-sectional configuration of the spillway control structure, which is represented in the CMS grid as a 40-cell series of weirs. Each cell represents 8.75 of the 350 total spillway bays, whose invert elevations vary between 15.35 and 17.22 feet NAVD as shown in Figure 2.2. The blue-colored cells in the cross-sectional view signify the bay groups that were open on 04/22/08.

Figure 2.2 is interesting because it shows the flow distribution between the Mississippi River and Bonnet Carre Spillway during a recorded flood event when the spillway was flowing at 68% of its design capacity and diverting roughly 12% of Mississippi River flow. These results, in terms of engineering and management practices in the Lower Mississippi River, are useful in conveying insight into how the operation of a large-scale river diversion impacts the main-channel river hydraulics. The accuracy of the results shown in Figure 2.2 is supported by a -2.1% error in the simulated stage at Reserve, LA, where the model underpredicted the observed stage by 0.5 feet.

Figure 2.3: Flow Patterns at a Rectangular River Diversion (Neary & Sotiropoulos, 1996)
Figure 2.3 illustrates conceptually how a river diversion alters the riverside flow-field by producing a dividing streamline at the river/diversion junction and a secondary wake immediately downstream. For navigability, these types of currents are important to predict in a river as heavily traveled by commercial and industrial ship traffic as the Mississippi River. However, as shown in Figure 2.2, under the river and spillway conditions during the 2008 flood, the flow division at the Bonnet Carre Spillway is confined mostly within the north-bank floodplain of the Mississippi River. Beyond that, the streamlines within the river are mostly unaltered.

2.3 Restoration Activities and Proposals in the Labranche Wetlands

The primary contributors to the current degradation of the Labranche wetland are salinity intrusion and subsidence, unmitigated in recent decades due to the hydrologic alteration of levee construction and disconnection from the Mississippi River (Day, et al., 2010). High salinity regimes in the wetland are checked to some degree by occasional freshwater input from precipitation and several pumping stations.

Figure 2.4: Drainage Pump Stations Discharging into the Labranche Wetland
Stations 1, 2, 3, and 4 in Figure 2.4 are storm water pumps that discharge into the wetland’s channelized waterways and contribute to lower salinity levels when operational. However, Station 1 is a drainage pump from the Norco oil refinery that discharges to Bayou Trepagnier and is responsible for contamination that has placed the bayou on the EPA 303d list of impaired water bodies since 1996 (http://www.epa.gov).

In recent decades, several restoration strategies have been proposed and performed in the Labranche wetland. These projects have varied in size, cost, and scope, and have targeted different mechanisms of stimulating and sustaining ecological health within the wetland. Federal funding resulting from the Coastal Wetlands Planning, Protection, and Restoration Act (CWPPRA) has been responsible for one completed marsh nourishment project in Labranche and several pending proposals. CWPPRA project PO-17, completed in 1994, placed 2.7 million cubic yards of dredged material in a northwest section of the wetland and successfully converted what was formerly 300 acres of agricultural impoundment to emergent marshland. CWPPRA project PO-75, approved in January 2010, proposes 729 acres of marsh creation in Labranche with dredged material from Lake Pontchartrain, adjacent to the PO-17 project site. These types of one-time dredge-and-fill projects are useful in offsetting subsidence and in physically building a firm substrate that promotes tree and plant growth. However, to maximize the benefit of this sediment delivery, pulsed freshwater inputs to the system would be beneficial in sustaining vegetation and driving out salinity intrusion.

Estimates for the amount of freshwater discharge that would maximize the ecological benefits in Labranche range from 1,000 cfs (CWPPRA, 2007) to 4,000 cfs.
The locations of proposed freshwater inputs are shown in Figure 2.5 and elaborated upon in Table 2.2.

**Figure 2.5: Locations of Proposed Freshwater Inputs for the Labranche Wetland**

**Table 2.2: Description of Freshwater Input Proposals**

<table>
<thead>
<tr>
<th>Location #</th>
<th>Proposal Name/Agency (Year)</th>
<th>Proposed Flow Rate</th>
<th>Estimated Construction Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>CWPPRA PPL-17 (2007)</td>
<td>1,000 to 3,000 cfs</td>
<td>$15 to $20 million</td>
</tr>
<tr>
<td>4</td>
<td>Lake Pontchartrain Basin Foundation (2009)</td>
<td>4,000 cfs</td>
<td>$25 million</td>
</tr>
<tr>
<td>5</td>
<td>USACE (1996)</td>
<td>4,000 cfs</td>
<td>$30 million</td>
</tr>
</tbody>
</table>

Additional features of CWPPRA PPL-18 include a partial damming (via a submerged weir) of Duncan Canal, which serves as the eastern boundary of the Labranche wetland.
This would promote greater dispersion and circulation of discharged storm water from the city of Kenner that travels by way of this canal to Lake Pontchartrain. Pumping of treated sewage from the city of Kenner is also included in the proposal, which would deliver nutrients to the wetland and supplement the benefits of the proposed freshwater and sediment inputs (CWPPRA, 2008).

One final CWPPRA strategy (CWPPRA, 2011) proposes an opportunistic use of the Bonnet Carre Spillway for the benefit of the Labranche wetland without the construction of any hard infrastructure. This proposal suggests a more frequent operation of the Bonnet Carre Spillway than is necessary for flood protection. Partially opened annually in the early spring, the spillway would allow up to 4,000 cfs to flow into Lake Pontchartrain, effectively freshening the south shore and lowering salinity levels in the Labranche wetland. The operational length of this proposed project is twenty years, with a total estimated cost of $1.1 million.

2.4 Simulated Freshwater Diversion Strategy

The model simulations presented in this thesis reflect the strategy of freshwater introduction into Labranche from the Bonnet Carre Spillway that is presented in USACE (1996), CWPPRA (2007), and the Lake Pontchartrain Basin Foundation (2009). The aforementioned proposals vary in the exact location and size of the diversion channel, but the diversion scenario investigated in this thesis most closely resembles that of USACE (1996). This scenario poses several drawbacks and benefits when compared to the other proposed freshwater input locations (Figure 2.5), and its selection for the current modeling task is not an endorsement of this particular channel’s size or location. It is
merely a more in-depth analysis of one alternative, which is appealing for several reasons:

1. By tying into the Bonnet Carre Spillway, there is no need to acquire the property rights or Congressional authorization to run a diversion channel directly from the Mississippi River. Furthermore, the ability of an existing large-scale flood-control diversion to act also as an agent of ecological restoration can be explored through the simulation of this alternative.

2. By connecting to the spillway’s Lower Borrow Canal (Figure 2.6) the proposed diversion channel could receive flow from the spillway whenever it is available in the canal. This could be at times of leakage through the spillway pins (Figure 1.1) or at times of spillway operation. Either way, the channel could receive freshwater as often as it flows through the spillway, which has proven to be frequent enough to produce beneficial results in the spillway.

3. Mississippi River sediment has been shown to deposit in significant amounts within the Bonnet Carre Spillway (Day, et al., 2010). A portion of this sediment has the potential of being available to the proposed Labranche diversion channel.

A diversion channel from the Bonnet Carre Spillway also presents several drawbacks. Flow originating from the western side of the wetland has the potential to short-circuit through Bayou Labranche directly to Lake Pontchartrain without nourishing the eastern portion of the wetland. Such a diversion would also not have the ability to operate if flow in Bonnet Carre were not sufficient to provide the freshwater source. The channel would necessarily cross the lower guide levee of the Bonnet Carre Spillway, which is heavily used as an access route for both official and recreational activities in the
spillway. Construction of a bridge at the levee/diversion junction would be required. On the wetland side, the diversion channel would have to cross both the Engineers Canal and Bayou Trepagnier, whose contaminant loads are such that mixing with the diversion water would be undesirable. Culvert crossings for both of these waterways would need to be implemented, such as those detailed by the USACE (1996).

Figure 2.6: Diversion Channel Used in ADH Simulations

The size and alignment of the channel outline shown in Figure 2.6 is replicated exactly from USACE design drawings (USACE, 1996). The design capacity of this channel is 4,000 cfs.
CHAPTER 3. METHODS

3.1 Hydrodynamic Code

The ADaptive Hydraulics modeling system (ADH) is a comprehensive software package with the ability to simulate many types of flow and constituent transport problems. The ADH module used in this thesis work is a two-dimensional depth-averaged finite element code that solves the shallow water equations (SWE) to obtain total water depth and depth-averaged x- and y-velocity. Using the assumption of hydrostatic pressure distribution, the SWE are a vertical integration of the three-dimensional Reynolds-averaged Navier-Stokes equations, which describe incompressible fluid motion in a control volume through the principles of mass and momentum conservation. The form of the SWE solved by ADH is given below (Berger & Stockstill, 1995).

\[
\frac{\partial U}{\partial t} + \frac{\partial F}{\partial x} + \frac{\partial G}{\partial y} + H = 0
\]  \hspace{1cm} (3.1)

where:

\[
U = \begin{bmatrix} h \\ uh \\ vh \end{bmatrix}
\]  \hspace{1cm} (3.2)

\[
F = \begin{bmatrix} uh \\ u^2h + \frac{1}{2}gh^2 - h \frac{\sigma_{xx}}{\rho} \\ uvh - h \frac{\sigma_{xy}}{\rho} \end{bmatrix}
\]  \hspace{1cm} (3.3)

\[
G = \begin{bmatrix} vh \\ uvh - h \frac{\sigma_{xy}}{\rho} \\ vh^2 + \frac{1}{2}gh^2 - h \frac{\sigma_{yy}}{\rho} \end{bmatrix}
\]  \hspace{1cm} (3.4)
Table 3.1: Variables Used in the Shallow Water Equations (Eqs. 3.1 – 3.5)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>h</td>
<td>Total water depth</td>
</tr>
<tr>
<td>u</td>
<td>x-component of the depth-averaged velocity</td>
</tr>
<tr>
<td>v</td>
<td>y-component of the depth-averaged velocity</td>
</tr>
<tr>
<td>g</td>
<td>Acceleration due to gravity</td>
</tr>
<tr>
<td>σ</td>
<td>Reynolds stresses due to turbulence</td>
</tr>
<tr>
<td>ρ</td>
<td>Fluid density</td>
</tr>
<tr>
<td>z_b</td>
<td>Bed elevation</td>
</tr>
<tr>
<td>n</td>
<td>Manning’s roughness coefficient</td>
</tr>
<tr>
<td>P</td>
<td>Pressure head</td>
</tr>
</tbody>
</table>

The Reynolds stresses, according to the Boussinesq approach, are related to the gradients of the mean velocities as shown below:

\[
\sigma_{xx} = 2 \rho v \frac{\partial u}{\partial x} \quad \text{(3.6)}
\]

\[
\sigma_{xy} = \sigma_{yx} = \rho v \left( \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \right) \quad \text{(3.7)}
\]

\[
\sigma_{xx} = 2 \rho v \frac{\partial v}{\partial y} \quad \text{(3.8)}
\]

where \( v \) represents the turbulent and molecular kinematic viscosity.

Numerical solution of the SWE in ADH is accomplished through a finite element method which utilizes an upwind weighted test function. An extension of the streamline upwind Petrov-Galerkin (SUPG) scheme, this method has been shown to be adequate for shock-capturing and numerical dissipation at locations of rapidly-varied flow, while remaining precise in the hydraulically smoother regions. Therefore, this method is well-suited to advection-dominated flow and high-velocity open channel problems (Berger & Stockstill, 1995).
The ADH modeling software is an open-source research code developed at the USACE Coastal and Hydraulics Laboratory (https://adh.usace.army.mil/). It may be run on a serial processor or compiled for parallel computation, on both Windows and Unix-based machines. Execution of the model is performed from a project directory that contains the model executable files and three user-defined input files: (1) the mesh file, (2) the boundary condition file, and (3) the hotstart file. The mesh file contains the computational grid over which model equations are solved. The boundary condition file defines the flow or transport problem to be modeled, with information detailing all of the input variables (both physical and computational) and time-series boundary conditions. The hotstart file contains the initial conditions for each node in the mesh file at the start of the simulation. The simulations presented in this thesis were performed with a parallel version of ADH version 3.3, compiled to run on the LSU high-performance computing clusters.

The model unknowns in the hydrodynamic module, depth and velocity, are computed in ADH using an implicit solver. Thus, the Courant criterion is not imposed for the model solution, and a shorter timestep will be automatically applied if instability arises from the user-defined maximum timestep. Otherwise, ADH will apply this maximum timestep until a solution cannot be reached within the iteration tolerances that are defined in the boundary condition file. The three iteration parameters required in the boundary condition file specify (1) the maximum number of non-linear iterations, (2) the convergence tolerance for the non-linear iterations, and (3) the maximum number of linear iterations for each non-linear iteration executed by ADH. Additionally, an optional iteration parameter may be included which specifies the tolerance for maximum change
in depth and velocity. At each timestep, ADH will perform up to the maximum number of non-linear and linear iterations until the maximum residual norm for all mesh nodes falls below the user-defined convergence tolerance. If the optional incremental tolerance is included with the boundary conditions, then the incremental maximum norm must fall below this tolerance. If a solution cannot be reached within the convergence tolerances, ADH will cut the initial timestep to ¼ of its value and begin the iterative calculations again. Conversely, if a solution is reached with a lower timestep than the user-defined maximum value, ADH will double the successive timesteps until the maximum timestep is achieved.

The basic underlying assumption of the two-dimensional SWE is that of vertical hydrostatic pressure distribution, which states that all vertical accelerations are due solely to gravity and the resulting vertical pressures increase linearly with depth. This assumption is reasonable in many open-channel applications, especially in straight channels. However, at meander bends, the change in flow direction causes a strong secondary current in the vertical plane, otherwise known as bendway vorticity. This concept is illustrated in Figure 3.1.

![Figure 3.1: Schematic Diagrams of (1) Hydrostatic Pressure Distribution (Akan, 2006) and (2) Bendway Vorticity in an Open Channel (Roca, 2009)](image)

The left-hand diagram in Figure 3.1 depicts the idealized scenario of linear vertical stresses in truly two-dimensional flow. The right-hand diagram depicts a cross-sectional...
view of how this linear stress model breaks down in the case of bendway vorticity and the resulting three-dimensional flow field. In many sinuous rivers, bendway vortices are responsible for significant amounts of turbulence, which entrain bed sediment into higher portions of the water column and cause significant scour. Bendway vortices are also responsible for energy losses in a two-dimensional sense, as the colliding streamlines around the bend force vertical accelerations that propel the vortex and cause turbulence that reduce the magnitude of the longitudinal flow.

Figure 3.2: Bendway Sediment Entrainment in a Small-Scale Physical Model

Figure 3.2 shows a vorticity observation at the Small-Scale Physical Model in the Vincent A. Forte Coastal and River Engineering Research Laboratory at Louisiana State University. The blue-colored model sediment that is deposited along the bed just upstream of this bendway is entrained at the bend and suspended in a vortex before being transported downstream. The focus here is not the sediment-transporting characteristics of bendway vorticity, but the identification of the vortex by the sediment which it entrains.
An optional card may be included in the ADH boundary condition file, which performs a correction for bendway vorticity. Hydrodynamically, a bendway vortex produces a change in the primary flow field through the migration of high velocities toward the outer bank of a bend (Figure 3.1 and Bernard & Schneider, 1992). Therefore, in a 2D model, the depth-averaged velocity magnitudes on the inside of a bend are over-predicted if no correction is made for the actual cross-stream momentum flux. Conversely, the uncorrected velocities on the outer bank are under-predicted. The method of vorticity correction used by ADH is given by Bernard & Schneider (1992), who developed a governing equation for the secondary flow at bendways:

\[
\frac{\delta \Omega}{\delta t} + u \cdot \nabla \Omega = \frac{A_s}{rh(1+9h^2/r^2)} - D_s \sqrt{C_f} \frac{|u|}{h} + \frac{1}{h} \nabla \cdot (vh \nabla \Omega) \tag{3.9}
\]

**Table 3.2: Variables Used in the Governing Vorticity Equation (Eq. 3.9)**

<table>
<thead>
<tr>
<th>Variable</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta \Omega/\delta t$</td>
<td>Secondary flow velocity</td>
</tr>
<tr>
<td>$u$</td>
<td>Depth-averaged streamwise velocity</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Coefficient</td>
</tr>
<tr>
<td>$C_f$</td>
<td>Friction factor given by Manning’s equation (Eq. 3.10)</td>
</tr>
<tr>
<td>$r$</td>
<td>Lateral radius of curvature</td>
</tr>
<tr>
<td>$h$</td>
<td>Water depth</td>
</tr>
<tr>
<td>$D_s$</td>
<td>Coefficient</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Eddy viscosity (given as $\varepsilon_{EVI}$ and $\varepsilon_{SDA}$ in Eqs. 3.12 &amp; 3.13)</td>
</tr>
</tbody>
</table>

\[
C_f = 9.81n^2h^{1/3} \tag{3.10}
\]

Whenever the vorticity card is turned on, ADH solves a discretized form of Equation 3.9 to obtain $\delta \Omega/\delta t$, the horizontal cross-stream velocity. With the activation of the vorticity card, the ADH user must also define values for $A_s$ and $D_s$, two empirical coefficients. In their development of the flow model, Bernard & Schneider validated it for the case of a single 270-degree bend flume and two multiple-bend flumes. They showed that for these different channel configurations, values of $A_s = 5$ and $D_s = 0.5$.
worked well in the flow model for each. These are the default values in ADH, and they
are the ones used for the simulations presented in this thesis. Once the vorticity equation
is solved, the cross-channel velocity is used to compute the component of the eddy
viscosity due to anisotropic dispersion and the subsequent Reynolds stresses.

The eddy, or turbulent, viscosity term, \( \nu \) (Table 3.2), is a tensor with dimensions
[Length\(^2\) / Time]. It accounts for momentum transferred by turbulent eddies that are
contained within a finite element and therefore not explicitly represented by the mesh
resolution (Berger, et al., 2010). There are several methods by which the eddy viscosity
may be defined or computed in ADH, but the one used for the simulations presented in
this thesis is the anisotropic Estimated Eddy Viscosity (EEV) formulation. In this
method, ADH calculates the eddy viscosity at each timestep as a term with three separate
components.

\[
\varepsilon_{EVI} = 0.92K\sqrt{C_d}hu \\
\varepsilon_{SDA} = 1.3\sqrt{C_d}hu \\
\varepsilon_{TDA} = 0.5\sqrt{C_d}hu_{T,MAX}
\]

(3.11)  
(3.12)  
(3.13)

The first component (Eq. 3.11, Webel & Schatzmann, 1984) is an isotropic term
that accounts for turbulent mixing. The second component (Eq. 3.12) is an anisotropic
term in the direction of flow that accounts for streamwise dispersion. The third
component (Eq. 3.13) is an anisotropic dispersion term normal to the direction of flow
that accounts for transverse dispersion. The \( u_{T,MAX} \) term in Eq. 3.13 is the cross-stream
velocity from the vorticity calculation (\( \delta \Omega / \delta t \) from Eq. 3.9), previously explained. If the
vorticity correction is not active, Eq. 3.12 is not included in the calculation. The
remaining variables in Eqs. 3.11-3.13 are defined in Table 3.3.
Table 3.3: Variables Used in EEV Calculations (Eqs. 3.11 – 3.13)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>K</td>
<td>User-defined scaling coefficient (0.1 \leq K \leq 1)</td>
</tr>
<tr>
<td>(C_d)</td>
<td>Drag coefficient, as determined by the bed friction</td>
</tr>
<tr>
<td>h</td>
<td>Water depth</td>
</tr>
<tr>
<td>u</td>
<td>Depth-averaged velocity</td>
</tr>
</tbody>
</table>

A minimum value for \(u\) (Eqs. 3.11 – 3.13) is stipulated for the EEV calculation, given by:

\[
u_{\text{MIN}} = 0.1\sqrt{gh}
\]  

(3.14)

Once it is calculated, the estimated eddy viscosity is added to the kinematic viscosity defined in the boundary condition file to obtain \(\nu\) (Eqs. 3.6 – 3.8), from which the Reynolds stresses are computed.

The most notable feature of ADH, to which the software owes its name, is its ability to dynamically adapt the input mesh resolution based on a computed mass-balance error, given below (Tate, Berger, & Stockstill, 2006).

\[
E_e = \left\{ \int_{a_e} \frac{\delta h}{\delta t} + \text{grad} \cdot (v_h h_h) \right\}^{2} \delta a_e \right\}^{1/2} A_e
\]  

(3.15)

Table 3.4: Variables Used in the Refinement Indicator Equation (Eq. 3.15)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>(E_e)</td>
<td>Elemental mass-balance error</td>
</tr>
<tr>
<td>(a_e)</td>
<td>Local integration (=1)</td>
</tr>
<tr>
<td>(h_h)</td>
<td>Average elemental water depth</td>
</tr>
<tr>
<td>(v_h)</td>
<td>Average elemental xy-velocity</td>
</tr>
<tr>
<td>(A_e)</td>
<td>Area of the element</td>
</tr>
</tbody>
</table>

Since a finite element mesh with inadequate resolution can introduce significant discretization errors during solution of the governing equations, an automated refinement tool within the model can be very advantageous. Using the total mass balance error (Eq. 3.15) within an element as an indicator of needed refinement, ADH will split the flagged elements based on user-specified tolerance for acceptable error. In this way an initial
mesh can be generated which captures the prototype geometry, and ADH will add the resolution necessary to capture the hydrodynamics. In general, higher resolution is needed in areas where the flow changes direction or speed. As illustrated in Figure 3.3, the overall benefit of the automated adaption is a lower computational burden. The adapted mesh is free of the errors induced by an overly coarse mesh, without the burden of excessive nodes, in regions that do not require them, that cause unnecessary computation and longer simulation times.

On the other hand, ADH adaption may also be used to coarsen an overly refined mesh. If the computed mass balance error (Eq. 3.15) in an element is less than the user-specified tolerance by at least a factor of four, that element will be merged with an adjacent element that meets the same criteria.

Figure 3.3: Comparison of Manually Generated Coarse (Left) and Fine (Right) Meshes with the Automated Adapted Mesh (Center) in ADH (Tate, Berger, & Stockstill, 2006)

ADH has the ability to allow the wetting and drying of elements as needed throughout a simulation. That is, the entire model domain does not have to start wet and remain so for the course of the model run. However, problems may arise from portions of the inflow boundary becoming wet during a model run, resulting in computational instability and massive time-step cuts. Therefore, it is sometimes necessary to modify the
inflow boundary string so as to ensure that it is continuously fully submerged (Tate & Berger, 2006).

Additionally, the user is allowed the option of defining a wet/dry limit to mesh nodes, below which ADH applies a shock-capturing algorithm for computational stability. It is good practice to use this option, but the wet/dry limit must be set at a value much lower than the final wet node depth to ensure that the smoothing of the wetting shock does not affect the final result.

The Manning’s roughness coefficient is an empirical parameter that characterizes bed roughness and is used to calculate friction losses in mathematical models. It is called empirical because, while it references a physical attribute of the bed material, the Manning’s n lacks a “definite and complete physical concept” that would otherwise make it a unique and intrinsic physical property, such as mass density (Ding, Yafei, & Wang, 2004). Therefore, the values assigned to the Manning’s roughness coefficients in a numerical model must be properly justified through a trial-and-error procedure until the model results adequately match field observations. In the SWE, the Manning’s roughness coefficient is incorporated into the two momentum equations as an added force term and represents the most important of the unknown user-defined variables in ADH. Yet, the Manning’s roughness term is not entirely without a physical frame of reference, and ample literature exists for approximate estimations of this variable. For example, Chow (1959) provides tables and photographs of various channel types and lists typical ranges for their corresponding roughness values. Brownlie (1981) presents formulations for estimating Manning’s roughness based on grain size of the channel bed. Studies such as
this are useful for obtaining starting roughness values for calibration simulations with numerical models.

3.2 Particle Tracking Code

ADH resolves shallow water hydrodynamics by computing total water depth and the depth-averaged horizontal velocity, i.e. the temporal derivative of the fluid continuum’s position vector, at each mesh node. However, it is often desirable to track the locations of discrete Lagrangian particles over time as they are transported by the computed flow fields across the finite element mesh. Such a process requires the integration of the modeled velocity vectors to obtain particle positions at discrete timesteps, and such a process is implemented by a particle tracking model (PTM).

PTM’s may be run both on- and off-line of a hydrodynamic code and may vary in the types of transport processes they simulate. Advection, diffusion, and dispersion are typical examples of physical transport mechanisms accounted for in PTM’s to predict the fate of water-borne constituents. Additionally, PTM’s may include decay terms to simulate the deposition of sediment.

Dill (2007) surveyed the currently available PTM’s and improved upon the existing Maureparticle code, an offline PTM that advects and tracks massless particles in a two-dimensional mesh. Among other changes, Dill added the processing of dynamic velocity fields to Maureparticle’s capabilities and implemented an optional Runge-Kutta second-order integration scheme as an alternative to the forward Euler scheme. In this thesis, a version of Dill’s PTM, modified to accept ADH output by Erol Karadogan, is used to post-process the velocity results of the hydrodynamic simulations. Particle
trajectories and residence-time distributions are mapped to aid in the visualization of the hypothetical introduction of freshwater into the Labranche wetland.

3.3 Model Domain and Mesh Generation

In any modeling study, due consideration must be paid to the appropriate selection of the boundaries that enclose the model domain. Selection of these boundaries depends as much on the scope and objectives of the modeling effort as on the availability of gage records or field measurements. That is, the model boundaries must represent locations where the hydrodynamic conditions are known, yet be far enough away from the primary study area so as to not affect the results there. The model domain, outlined in green, for the simulations performed for this thesis is shown in Figure 3.4 below. Locations of river and tide gages are shown in blue, and locations of field measurements are shown in red.

The ADH model domain (Figure 3.4) consists of four distinct regions, namely the Mississippi River, the Bonnet Carre Spillway, the Labranche wetland, and Lake Pontchartrain. The total area within each region is given in Table 3.5. Model boundaries for the Mississippi River were selected based on the locations of USACE river gages (Table 3.6). The nearest gage upstream of Bonnet Carre is located at Reserve, LA, approximately twelve river-miles upstream of the spillway. This was deemed to be far enough upstream of the spillway so as not to introduce “boundary errors” due to model forcing. Similarly, the downstream river boundary was set at the location of the nearest gage south of the spillway – Carrollton, LA, approximately twenty-four river miles downstream. The same methodology was applied in selecting the downstream boundary
out in Lake Pontchartrain, choosing a location far enough away from the spillway and wetland that would not affect either region by a tailwater forcing.

![Image](image_url)

**Figure 3.4: 2008 Aerial Imagery, Model Domain, Field Measurements, and Permanent Gages**

**Table 3.5: ADH Model Domain**

<table>
<thead>
<tr>
<th>Region</th>
<th>Area (Square Miles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mississippi River</td>
<td>24.68</td>
</tr>
<tr>
<td>Bonnet Carre Spillway</td>
<td>10.83</td>
</tr>
<tr>
<td>Labranche Wetland</td>
<td>30.46</td>
</tr>
<tr>
<td>Lake Pontchartrain</td>
<td>274.09</td>
</tr>
</tbody>
</table>
After selecting the proper model domain, the finite element mesh was constructed. The finite element mesh is a grid of (x,y,z) points that represent the geometry within the model domain and serve as the vertices for the triangular elements used in the model computation. The mesh is the virtual surface on which the model equations are solved, and its importance in the solution of the hydrodynamic problem cannot be overstated. However, it is the job of the hydrodynamic modeler to construct a mesh that not only retains the integrity of the real-world geometry, but also is as
conservative as possible with the number of nodes that increase the computational burden of the model.

The mesh used for the simulations in this thesis was generated in Aquaveo’s Surface Water Modeling System (SMS) version 10.1, a graphical user interface software designed for the visualization and analysis of numerical data. It is also equipped with many additional tools for numerical modeling. The following steps describe the mesh generation process that was used in SMS.

1. Importing aerial imagery of the study area
2. Tracing feature arcs at the desired vertex spacing to define the mesh boundaries and resolution
3. Selecting and executing the triangulation method for building polygons
4. Converting the “map” file of data in the arcs and polygons into the 2D mesh
5. Manually improving the mesh quality in areas of abrupt transitions in element sizes
6. Importing bathymetry and topography data to interpolate to the mesh nodes
7. Defining material types for each element for the assignment of model parameters

The aerial imagery that served as the backdrop for mesh construction were 2008 infrared Digital Ortho Quarter-Quad (DOQQ) satellite images produced by the U.S. Geological Survey. Generating the mesh in SMS was done in a piecewise fashion, beginning with the Mississippi River.
The feature arc tool in SMS was used to trace the main channel and overbank areas between the artificial levees that enclose this reach of the river. The maximum vertex spacing was 330 feet. Additional feature arcs were drawn at the centerline of the main channel at bendways, and the vertex spacing for these arcs was set at 130 feet in anticipation of necessary mesh refinement in those areas. Cross-sectional arcs (from levee to levee) were drawn at certain locations to delineate areas of different polygon generation methods. For instance, polygons in the relatively straight sections of the river were generated with a patching algorithm while the curvature at bendways required a paving algorithm. Polygons were then created, and the feature map was converted to a 2D mesh.

Node elevations in the generated mesh were interpolated from the latest USACE hydrographic survey data. Approximately every ten years, the USACE publishes hydrographic survey maps of the Mississippi River from north of Old River Control (RM 321) down to the Gulf of Mexico, given in cross-sections that are surveyed every 0.10 to 0.25 river miles within the main channel. Figure 3.6 is an excerpt from the most recent USACE hydrographic survey maps, published in 2007, near the Bonnet Carre Spillway.
Figure 3.6: Hydrographic Survey Map Near the Bonnet Carre Spillway (USACE, 2007)

The data used to produce the USACE maps were obtained as text files from the Louisiana Office of Coastal Protection and Restoration (OCPR) in the native USACE *.830 survey format. These data files were processed, and the survey points were extracted for interpolation to the ADH mesh. Figure 3.7 shows the portion of the ADH mesh corresponding to the survey map in Figure 3.6, overlain with the survey data in SMS.

It can be seen in Figure 3.7 that the USACE cross-sectional survey omits large portions of the floodplain in some areas. In such cases, the river bathymetry data was supplemented with Lidar topography data downloaded from the LSU Atlas website. This Lidar data was gathered in 2003 and is the most recent topography data available for the area. No attempt was made to verify the accuracy of the Lidar in all locations, but it was found to be fairly consistent with the USACE bathymetric survey data in areas of overlap.
After interpolating the bathymetry and topography data to the mesh nodes in SMS with a linear interpolation algorithm, several ADH adaption simulations were run with the mesh at flow rates of up to 1,200,000 cfs. The original mesh contained 16,236 nodes and 30,734 elements; the adapted mesh, shown in Figure 3.8, contains 24,413 nodes and 47,074 elements. In relative terms, the adaptive runs were responsible for over a 50% increase in the number of both nodes and elements.

After generating the Mississippi River portion of the ADH mesh, a feature map of the Bonnet Carre Spillway region was traced in SMS. Feature arcs were drawn over 2008 DOQQ images at a maximum vertex spacing of 330 feet. A minimum vertex spacing of 16.5 feet was required to capture relevant hydrologic features within the floodway. These features include the upper and lower borrow canals, which run parallel to the spillway.
guide levees from Highway 61 to Lake Pontchartrain. The feature map of the Bonnet Carre Spillway is shown in Figure 3.9.

**Figure 3.8**: Color-Contoured ADH Mesh of the Mississippi River Between Reserve, LA (RM 138.8) and Carrollton, LA (RM 102.8)

**Figure 3.9**: Feature Map of Bonnet Carre Spillway (Feature Arcs, Nodes, and Vertices) Used for Mesh Generation
Polygons were generated from the feature map of the spillway, and the 2D mesh was created. The total number of mesh nodes in the spillway is 30,592 and the total number of elements is 60,640. Lidar topography within the spillway region was downloaded from the LSU Atlas website and linearly interpolated to the mesh nodes in SMS.

![Lidar Points and Sampled Cross-Sections in the Bonnet Carre Spillway](image)

**Figure 3.10: Lidar Points and Sampled Cross-Sections in the Bonnet Carre Spillway**

However, because of its highly irregular nature, some degree of processing was required to “smooth” the raw Lidar elevations and make them suitable for modeling purposes. To this end, a series of cross-sections were defined within the spillway, and a Matlab code was written to sample the Lidar points across these transects at 5-meter intervals. The cross-sections were plotted for visual analysis, and approximations of
average elevations within each cross-section were manually determined and plotted with the raw Lidar elevations for comparison. These “averaged” cross-sections include linear elevation changes where necessary, but they neglect the topographic irregularities that are not necessary to capture for the scope of this modeling study. Figure 3.10 shows a plan view of raw spillway Lidar and the cross-sections that were sampled.

Figure 3.11 shows the cross-section taken along Highway 61, which corresponds to transect F/F* in Figure 3.10.

**Figure 3.11: Raw and Averaged Lidar Topography at the Highway 61 Cross-Section in the Bonnet Carre Spillway**

After all eleven cross-sections were manually averaged (as shown in Figure 3.11), the processed cross-sections were imported into SMS and overlain on the spillway mesh.
The processed cross-sections were linearly interpolated to the spillway mesh, and the resulting mesh elevations were compared with the raw Lidar elevations.
It can be seen qualitatively in Figure 3.13 that the topographic character of the raw Lidar data is well reflected in the processed ADH mesh elevations. Bathymetry for the upper and lower borrow canals were gathered from recreational brochures of the Bonnet Carre Spillway and checked against spot elevations found in USACE (1996). Channel depths of 20 feet were assigned to both canals at the far end near the Highway 61 bridge abutments, and depths of 50 feet were assigned to the ends near Lake Pontchartrain. Linear slopes were applied between the ends of each canal for bed slopes of approximately -0.002, as shown in Figure 3.14. A mid-channel cross-section representative of both canals is shown in Figure 3.15, illustrating the channel width and side slope applied to the canals.

Figure 3.14: Bathymetric Profiles for Upper and Lower Borrow Canals in the Bonnet Carre Spillway
After the Bonnet Carre Spillway portion of the ADH mesh was completed, a series of “test” simulations were performed at flow rates of up to 250,000 cfs in the spillway to gauge the need for adaption. The reported mass-balance errors for these simulations were very low, signifying adequate mesh resolution and the need for no additional refinement.

The Lake Pontchartrain portion of the ADH mesh was not generated manually, but the desired portion clipped from the existing ADCIRC SL-15 mesh, developed by Dr. Joannes Westerlink at the University of Notre Dame (URS, 2007). The SL-15 mesh (shown in colored contours) is shown in Figure 3.16, with the extents of extracted Lake Pontchartrain portion outlined in red. Digitized bathymetry from navigation maps (http://www.noaa.gov) were obtained in ASCII format and interpolated to the clipped portion of the SL-15 mesh. Prior to the mesh interpolation, the necessary horizontal and vertical datum conversions were performed on the data, for consistency with the portions
of the ADH mesh already generated. The Lake Pontchartrain portion of the ADH mesh is shown in Figure 3.17.
Finally, mesh generation of the Labranche wetland was performed. This task presented large challenges in terms of scale, as the hydrology of the wetland is primarily defined by channels on the order of ten to twenty meters wide. Figure 3.18 shows an aerial image of Labranche, labeled with the named waterways.

![Figure 3.18: Named Waterways in the Labranche Wetland](image_url)

Flow circulation (due to tides, precipitation, and storm water conveyance) is highly affected by the presence of the Illinois Central Gulf (ICG) Railroad, which forms a hydrologic barrier within the wetland. There exist six openings beneath the railroad through which water may pass, and these are shown in Figure 3.19. Though the channels that pass beneath the railroad are relatively small-scale features when compared to the size of the wetland, it is impossible to realistically represent the hydrologic connectivity of the wetland without explicitly including these channels in the ADH mesh. Therefore, a feature map of the Labranche wetland in SMS was drawn to reflect the presence of these channels, as well as some of the other small-scale flow features. Feature arcs were drawn
with a minimum vertex spacing of 6.50 feet along channel banks, to ensure multiple mesh nodes across the channels and accurately define the geometry. Feature arc vertices were spaced as widely as possible when not near channels, at a maximum length of 1150 feet. The resulting feature map is shown in Figure 3.20.

![Feature Map of Labranche Wetland](image)

**Figure 3.19: Channel Crossings Beneath the ICG Railroad**

![Feature Map of Labranche Wetland](image)

**Figure 3.20: Feature Map of Labranche Wetland (Feature Arcs, Nodes, and Vertices) Used for Mesh Generation**
Lidar topography data of the wetland was downloaded from the LSU Atlas website ([www.atlas.lsu.edu](http://www.atlas.lsu.edu)) and imported into SMS. As with the topography of the Bonnet Carre Spillway, large irregularities in the raw Labranche Lidar necessitated some processing to make the data suitable for modeling. To that end, a series of cross-sections were sampled from the Labranche Lidar in the same manner as they were in the Bonnet Carre Spillway. Figure 3.21 shows the raw Lidar data and the sampled cross-sections.

**Figure 3.21: Lidar Points and Sampled Cross-Sections in the Labranche Wetland**

Unlike the manual averaging process used for the spillway cross-sections, an automated moving-average low-pass data filter was implemented on each of the wetland cross-sections using the built-in Matlab function “smooth.” In this manner, the high-frequency oscillations in the Lidar data were removed in a very efficient way. The end result was a series of processed cross-sections that retained the general character of the
Lidar topography without the extreme point-to-point elevation changes common to some of the raw data. Figure 3.22 shows one of the filtered cross-sections, corresponding to section N/N* in Figure 3.21.

![LaBranche Cross-Section #3](image)

**Figure 3.22: Raw and Filtered Lidar Cross-Section the Labranche Wetland**

After all twelve cross-sections were filtered, the processed cross-sections were imported into SMS and overlain on the wetland mesh (Figure 3.23). The processed cross-sections were interpolated to the wetland mesh, and the overall mesh elevations were compared to the raw Lidar elevations. As shown in Figure 3.24, the overall character of the raw Lidar data is well reflected in the processed ADH mesh elevations.

Bathymetric elevations in the wetland were assigned to the ADH mesh based on both survey data and unofficial field measurements obtained from the U.S. Department of Agriculture (USDA), the Louisiana Department of Natural Resources (LDNR), Coastal Environments, Inc. (CEI), and the Wetlands and Wildlife Management Company.
The bathymetric connectivity within the wetland, as defined in the ADH mesh, is shown in Figure 3.25. The dark regions of the mesh in Figure 3.25 are due to very dense nodal spacing, not elevation contours.
Figure 3.25: Bathymetric Portion of ADH Mesh in the Labranche Wetland

For comparison, Figures 3.26 and 3.27 show a survey map and the contoured ADH mesh, respectively, for an open water area near Lake Pontchartrain in the northwest portion of the mesh. This open water area is the location for the proposed marsh nourishment project CWRPPA PO-75. After assigning bathymetry elevations, the Labranche portion of the mesh was complete. It contained a total of 256,129 nodes and 511,061 elements. Preliminary test simulations at maximum flow rates of 5,000 cfs with the wetland mesh indicated that no adaption was necessary. Completion of the Labranche portion of the mesh signified a completion of the total mesh generation process for this model study.
Figure 3.26: T. Baker Smith Bathymetric Survey for CWRPPA PO-75 (See Figure 3.25 for Location of this Area)

Figure 3.27: ADH Mesh Populated With Survey Data (See Figure 3.25 for Location of this Area)
A color-contoured image of the complete ADH mesh is shown in Figure 3.28, which represents the existing conditions of the region.

Figure 3.28: Complete ADH Mesh With Color-Contoured Elevations

A three-dimensional rendering of this mesh, produced with the Visualization Science Group’s proprietary Avizo software suite, is shown in Figure 3.29.

Figure 3.29: 3D Rendering of the Complete ADH Mesh
After completely generating the ADH mesh, modifications to include the proposed Labranche diversion channel were necessary in preparation of the diversion simulations in ADH. Figure 3.30 (USACE, 1996) shows a plan and profile view of the diversion channel proposed by the USACE, with a design discharge of 4,000 cfs.

![Plan-and-Profile Drawing of Proposed Labranche Diversion Channel (USACE, 1996)](image)

**Figure 3.30: Plan-and-Profile Drawing of Proposed Labranche Diversion Channel (USACE, 1996)**

The proposed 98-foot-wide channel (Figure 3.30) extends perpendicularly from the lower borrow canal in the Bonnet Carre Spillway into the Labranche wetland for a total length of approximately 2,650 feet. An invert elevation of -10 feet is maintained from the channel’s inflow end at the lower borrow canal until it crosses the spillway’s lower guide levee, Engineers Canal, and Bayou Trepagnier. After crossing Bayou Trepagnier, the diversion channel’s bottom elevation increases at a slope of 0.0125 for 800 feet to meet the ground elevation in the wetland (~0 ft). The USACE design proposes culvert crossings where (1) the diversion channel crosses the Bonnet Carre Spillway lower guide levee and (2) both Engineers Canal and Bayou Trepagnier cross beneath the
diversion channel. None of these culvert crossings are represented in the ADH model.

The Labranche diversion is represented as an open channel from beginning to end, as it is thought that a mechanical gate structure would be installed at the spillway levee to regulate flow into the wetland. However, a control structure of this kind is not represented in the model either, as the simulations are meant to analyze the potential of this diversion channel operated at full capacity. The two culvert crossings beneath the diversion channel at Engineers Canal and Bayou Trepagnier are excluded from the model because their effect on the overall model results are thought to be insignificant. Figure 3.31 shows the Labranche diversion channel as it was incorporated into the ADH mesh.

![Figure 3.31: Labranche Diversion Channel in the ADH Mesh](image)

The mesh shown in Figure 3.31 represents the proposed conditions for the Labranche diversion scenario; it was saved as a separate mesh file to be used for the diversion simulations after calibration/validation of “existing conditions” mesh.
3.4 Calibration and Validation Simulations of Existing Conditions

Calibration, validation, and sensitivity analyses of the ADH model under existing conditions were performed prior to any simulations of the proposed Labranche diversion. These simulations were performed separately on individual sections of the complete ADH mesh (Figures 3.28 & 3.29) in order to determine proper values for model variables and test the model’s performance in isolated regions of the mesh in a systematic way. For the calibration, validation, and sensitivity simulations, the ADH mesh was divided into four separate meshes.

![Mesh #1 (Mississippi River)](image)

**Figure 3.32: Mesh #1 (Mississippi River)**

Figure 3.32 shows what will be hereafter referred to as Mesh #1, the Mississippi River portion of the complete ADH mesh. The purpose of the calibration simulations with Mesh #1 was to establish the correct Manning’s roughness values, coefficients of eddy viscosity, and whether or not vorticity correction was appropriate for this portion of the mesh. The calibration simulations were performed by forcing an inflow boundary condition along the nodestring that defines the upstream edge of the mesh and a tailwater forcing along the nodestring that defines the downstream edge.
Two different material types in the ADH model were delineated for Mesh #1, corresponding to the main channel and overbank regions. Initial simulations with Mesh #1 showed no sensitivity to Manning’s roughness values in the overbanks, since the floodplain is a relatively small percentage of the total flow for this reach of the Mississippi River. A Manning’s roughness value of 0.035 was then specified for the overbanks, knowing that the true roughness value, however insignificant, should be defined as higher in the overbanks than in the main channel. Therefore, the overbank roughness was treated as a constant for the calibration and validation simulations, and the coefficient of eddy viscosity in the overbanks were varied with those in the main channel. Figure 3.33 shows the delineation of material types, to which roughness and eddy viscosity coefficients were assigned.

![Figure 3.33: Material Delineation in Mesh #1 (Main Channel and Overbank)](image)

Boundary conditions for the Mesh #1 calibration simulations were applied based on the rating curve analyses of USACE gage data performed by PhD candidate Erol Karadogan. Locations of the USACE river gages at Reserve, Bonnet Carre, and Carrollton are detailed in Figure 3.4 and Table 3.6. Daily stage data for these gages were downloaded ([http://www.mvn.usace.army.mil/eng/edhd/wcontrol/miss.asp](http://www.mvn.usace.army.mil/eng/edhd/wcontrol/miss.asp)) and plotted.
against the corresponding records of discharge at Tarbert Landing, lagged by two days. This was done for each of the three gages over the historic time period from October 1987 to September 2009. Plotting stage vs. the assumed discharge for each gage, a polyfitted curve of the data was generated using Matlab and standard deviations were computed at each flow rate to quantify the observed stage variability. The result was a rating curve for each of the three gages, as exemplified by one at Reserve shown in Figure 3.34.

![Figure 3.34: Rating Curve at Reserve, LA (Errorbar = ± Std. Dev.)](image)

Thus, for flow rates from 300,000 cfs to 1,200,000 cfs, the average stage for each of the three gages in Mesh #1 was obtained, for the time period that was analyzed. The ADH model for Mesh #1 was then calibrated to these average stage values, by varying the relevant model variables until the simulated stages matched the average observed rating-curve stages in a least-square sense. Initial estimates for Manning’s roughness values ranged from 0.014 to 0.022, and the limits of the coefficient of eddy viscosity are 0.1 and 0.9. Therefore, a total of 30 calibration simulations for Mesh #1 were performed,
where the Manning’s roughness coefficient was varied from 0.014 to 0.022 at 0.02 intervals, coefficient of eddy viscosity was varied among values of 0.1, 0.5, and 0.9, and the vorticity correction card was varied between “on” and “off.” Boundary conditions were unchanged among all the simulations. A ramped discharge was applied at the inflow boundary, using a hyperbolic tangent function to increase flow from zero to 1,200,000 cfs. A dynamic tailwater boundary condition was used at the downstream end of the model, which corresponded to the Carrollton rating-curve stage for each value in inflow time series. At each 100,000 cfs interval, the model was brought to a steady state to ensure that the model results at each of these flow rates could be compared to the average rating curve stages without bias to prior hydrodynamic conditions in the model.

After calibration, two validation simulations were performed with Mesh #1 to test the model’s ability to reproduce recorded flood events. Time periods for the validation events were selected based on (1) recent occurrence, (2) mid- to high-flow rates, (3) approximate two-week duration. The selected validation events are shown in Figures 3.35 and 3.36, which specify the time period and boundary conditions for the simulations.
Figure 3.36: Boundary Conditions for Mesh #1, Validation Simulation #2

Figure 3.37 shows what will hereafter be referred to as Mesh #2. It includes the Bonnet Carre Spillway and Lake Pontchartrain portions of the ADH mesh, and a series of calibration simulations was performed to determine the correct model variables for these regions under existing conditions. Material delineations for Mesh #2 are shown in Figure 3.38.

The first set of calibration simulations performed with Mesh #2 targeted the model parameters in Lake Pontchartrain. Different combinations of Manning’s roughness and eddy viscosity coefficients within the lake were specified under tidal forcing with zero inflow through the spillway. Using the ADCIRC tidal database (http://www.unc.edu/ims/ccats/tides/tides.htm) to obtain the tidal constituents for the mesh nodes along the outflow boundary, a time series of tidal tailwater forcing along the open edge string was computed using the T-Tide package in Matlab. The dates selected for this analysis were a 12-day period when the maximum tidal amplitude was 0.30 feet.
For these calibration runs, the Manning's roughness in the lake was varied from 0.1 to 0.3, and the coefficient of eddy viscosity was varied between 0.1 and 0.9. The
optional vorticity correction card was turned off, since it was expected to insignificantly affect the results. The boundary condition for these simulations is shown in Figure 3.39.

![Tidal Forcing At Model Boundary (From ADCIRC Tidal Database)](image)

**Figure 3.39: Tidal Forcing for Model Calibration of Lake Pontchartrain, Using Mesh #2**

The second set of calibration simulations performed with Mesh #2 targeted the model parameters in the Bonnet Carre Spillway. Daily water depths and depth-averaged longitudinal velocities were measured in the Bonnet Carre Spillway by the U.S. Geological Survey (USGS) during the 1997 and 2008 openings. These measurements, taken along the Highway 61 bridge, are the basis of the ADH model calibration in the Bonnet Carre Spillway. The location of these measurements is shown in Figure 3.40. The spillway data collected in 1997 and 2008 were obtained from Scott Perrien at the USGS office in Baton Rouge. The daily water depth and velocity measurements were integrated across the total cross-section in an Excel spreadsheet to obtain total discharge beneath the Highway 61 bridge.
Figure 3.40: Location of USGS Measurements Along the Highway 61 Bridge During the 1997 and 2008 Bonnet Carre Spillway Openings

Data analysis of the USGS spillway data was performed primarily by Cynthia Boshell and a team of undergraduate civil engineering students employed by Dr. Clint Willson at LSU during the summer of 2010. Of these integrated discharge values, the five highest ones were selected and used as the inflow boundary conditions for the spillway calibration runs with Mesh #2. Tide gage records in Lake Pontchartrain (Figure 3.4) were obtained for the dates selected and used to set the tailwater boundary condition in Lake Pontchartrain. Since it is unknown at what time the USGS measurements were taken on each day, the stage records in Lake Pontchartrain were averaged over the entire day for all the gages that were available. Flow through the spillway was ramped up to the desired discharge for each simulation and held constant until the model reached a steady state with the corresponding tailwater condition. Table 3.7 shows the five discharge events that were chosen for spillway calibration.
Table 3.7: Model Forcings for Five Calibration Simulations in the Bonnet Carre Spillway, Using Mesh #2

<table>
<thead>
<tr>
<th>Date</th>
<th>Inflow (cfs)</th>
<th>Tailwater (ft-NAVD88)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4/19/2008</td>
<td>139,100</td>
<td>1.48</td>
</tr>
<tr>
<td>3/22/1997</td>
<td>167,000</td>
<td>0.79</td>
</tr>
<tr>
<td>4/22/2008</td>
<td>169,300</td>
<td>1.42</td>
</tr>
<tr>
<td>3/31/1997</td>
<td>209,000</td>
<td>1.36</td>
</tr>
<tr>
<td>3/27/1997</td>
<td>243,000</td>
<td>1.51</td>
</tr>
</tbody>
</table>

For each of the calibration simulations, different combinations of Manning’s roughness (varied between 0.02 and 0.07) and coefficient of eddy viscosity (varied between 0.1 and 0.9) were tested. As shown in Figure 3.38, different material types were delineated between the borrow canals and overland regions of the Bonnet Carre Spillway. A constant value of 0.02 was assigned to the borrow canal materials, and only the overland Manning’s roughness was tested during the calibration runs. This value was chosen because it was the lower end of the feasible range that was tested for the overland material, and documentation exists for earthen channels with this roughness value (Chow, 1959). The optional vorticity card was left off in an effort to reduce the number of calibration simulations, and it was not expected to significantly impact the results in the spillway. No validation runs were simulated for the spillway, since no data existed for flow events besides those sampled for calibration.

After calibration of the river, spillway, and lake, calibration attempts were performed for the Labranche wetland. These were done with Mesh #3, shown in Figure 3.41. Mesh #3 represents the Labranche wetland under existing conditions and its connection to Lake Pontchartrain. The diversion channel is nonexistent in this mesh. The goal of the calibration simulations performed with Mesh #3 was to obtain the correct Manning’s roughness and eddy viscosity coefficients for the bathymetric portions of the...
wetland under normal tidal forcing. The materials assigned to Mesh #3 are shown in Figure 3.42.

Figure 3.41: Mesh #3 (Labranche Wetland and Lake Pontchartrain)

Figure 3.42: Mesh #3 Materials (Labranche Overland Material, Labranche Bathymetric Material, and Lake Pontchartrain Material)
Figure 3.43: Measured Channel Depths at ICG Railroad Crossings in the Labranche Wetland

Figure 3.44: Measured Channel Velocities at ICG Railroad Crossings in the Labranche Wetland
Field measurements of water depth and flow velocity were taken on several days between December 2004 and May 2005 by John Day and Rachael Hunter with Comite Resources. These measurements were recorded at each of the six ICG Railroad crossings in the wetland (Figures 3.4 & 3.19) and are shown in Figures 3.43 and 3.44.

Flow in and out of the wetland is affected by tides, precipitation, wind, and discharge from several pump stations (Figure 2.4). To simplify the calibration simulations for the wetland, the dates from Figures 3.43 and 3.44 were desired on which flows were generated primarily from tides. This would allow for relatively simple model forcing – a time series of tailwater elevations that would drive water fluxes within the model. February 15, 2005 stands out as a notable date for that reason. Figure 3.45 shows how closely the predicted tide levels match the observed water surface elevations at the National Oceanic and Atmospheric Administration (NOAA) gage in Bayou Labranche on 2/15/05.

![Graph showing predicted and observed water levels](image)

**Figure 3.45: Predicted and Observed Water Levels at NOAA Gage in Bayou Labranche on 02/15/05**

The observed water levels are under predicted by the tidal forecast in the last eight hours of the day (Figure 3.45) but are otherwise a close match with the NOAA
predictions. Figure 3.46 shows the wind record on the same day from the same NOAA gage. Large gusts (Figure 3.46) were recorded at the same time that observed water levels (Figure 3.45) began to deviate from their predicted trend. While this increased water setup only equates to roughly 0.15 feet, Figures 3.45 and 3.46 indicate that the impact of wind on water levels in Bayou Labranche may be significant relative to tides.

Figure 3.46: Observed Wind Speeds and Gusts at NOAA Gage in Bayou Labranche on 02/15/05

February 15, 2005 was chosen as a calibration date, and the tidal forcing along the open water boundary in Lake Pontchartrain was computed from a T-Tide computation of the constituents obtained from the ADCIRC Tidal Database. The time series of tailwater forcing for this calibration simulation is shown in Figure 3.47. Given that mean sea level in Lake Pontchartrain, given by NOAA’s VDatum software, is roughly 0.70 feet NAVD88, the boundary condition given by T-Tide (Figure 3.47) matches well with the predicted tide in Bayou Labranche (Figure 3.45).
Figure 3.47: Tidal Forcing for Bayou Labranche Calibration with Mesh #3

An initial Manning’s roughness value of 0.02 was assigned to all of the bathymetric features in the Labranche wetland and a value of 0.05 for all of the topographic regions for the calibration simulation shown in Figure 3.46. The eddy viscosity coefficient was set to 0.5, and the vorticity correction card was turned on. These values were planned to be adjusted in light of the initial results.

3.5 Diversion Simulations

Simulations of the proposed Labranche diversion began with a series of sensitivity simulations. These were performed primarily to analyze the ADH model sensitivity to the input Manning’s roughness values in light of uncertainties surrounding the calibration simulations (explained in Chapter 4). These simulations were performed with a modified version of Mesh #3, which was changed to include the proposed Labranche diversion channel. This mesh will be referred to as Mesh #4 and is shown in Figure 3.48.
Figure 3.48: Mesh #4 (Labranche Wetland with Diversion Channel and Lake Pontchartrain)

Material types in Mesh #4 are identical to those shown in Figure 3.42, except that the diversion channel was assigned its own material. For the sensitivity runs, the Manning’s roughness in the diversion channel was kept constant at 0.020 while the overland and submerged regions were assigned various roughness values. In light of calibration results for Meshes #1-3, the coefficient of eddy viscosity was kept constant at 0.5 for all materials for the sensitivity runs in Mesh #4. The vorticity correction card was turned on. The design capacity for the diversion channel that is replicated in Mesh #4 is given as 4,000 cfs (USACE, 1996). The boundary conditions for the sensitivity runs consisted of a flow ramp from zero to 4,000 cfs through the diversion channel and a constant tailwater of 0.070 (mean sea level) at the open water boundary in Lake
Pontchartrain. Manning’s roughness values for the sensitivity runs are shown in Table 3.8.

### Table 3.8: Roughness Values Used for Wetland Sensitivity Simulations

<table>
<thead>
<tr>
<th>Sensitivity Simulation ID</th>
<th>Manning’s Roughness Values (n)</th>
</tr>
</thead>
</table>
| 1                        | Wetland Overland $n = 0.060$  
Wetland Bathymetric $n = 0.060$  
Diversion Channel $n = 0.020$  
Lake Pontchartrain $n = 0.020$ |
| 2                        | Wetland Overland $n = 0.060$  
Wetland Bathymetric $n = 0.010$  
Diversion Channel $n = 0.020$  
Lake Pontchartrain $n = 0.020$ |
| 3                        | Wetland Overland $n = 0.030$  
Wetland Bathymetric $n = 0.060$  
Diversion Channel $n = 0.020$  
Lake Pontchartrain $n = 0.020$ |
| 4                        | Wetland Overland $n = 0.030$  
Wetland Bathymetric $n = 0.010$  
Diversion Channel $n = 0.020$  
Lake Pontchartrain $n = 0.020$ |

Finally, four simulations were performed to assess the impact of the proposed freshwater diversion into Labranche. The objectives of these simulations were to (1) determine the amount of flow entering the wetland at different magnitudes of flow in the Bonnet Carre Spillway and (2) determine if any possible short-circuiting of freshwater through the wetland could be mitigated through a damming of Bayou Labranche at the ICG Railroad. Figure 3.49 shows Mesh #5, which was used for these simulations. It includes the Bonnet Carre Spillway, Labranche wetland, and Lake Pontchartrain. The ultimate goal was to perform these diversion simulations with the complete ADH mesh, which included the calibrated portion of the Mississippi River (Mesh #1, Figure 3.32). This would have allowed the simultaneous simulation of (1) the interaction between the Mississippi River and the Bonnet Carre Spillway and (2) the interaction between the Bonnet Carre Spillway and the Labranche wetland.
ADH has a built-in structures library capable of approximating flow through weirs, flap gates, and spillways. These structures cards were used in various attempts to model the Bonnet Carre control structure and connect the separately calibrated ADH models of the Mississippi River and Bonnet Carre Spillway. Since the explicit incorporation of the individual spillway bays in the Bonnet Carre mesh was entirely unfeasible for scope and scale of this model, the use of the structures library in ADH was the only possibility for modeling flow from the river into the spillway. However, all attempts at implementing the structures cards produced unusable results. A variety of initial and boundary conditions were used, but the model eventually became unstable when flow was regulated by one of the built-in ADH structures. Particularly, high
maximum residual norms and incremental maximum norms would occur at and near the levee nodes of the node string along which the structures were defined. These high norms caused massive time-step cuts that effectively killed the simulations. Therefore, the diversion simulations were performed without connecting the Bonnet Carre Spillway to the Mississippi River. Rather, this task is left as future work.

The inflow forcings for the four diversion simulations with Mesh #5 were based on estimates for high and low discharges in the Bonnet Carre Spillway. The low flow condition was based upon leakage estimates through the spillway bays during normal times when the spillway is closed. The rating curve in Figure 1.1 estimates a maximum leakage of 10,000 cfs through the spillway when the USACE gage at Bonnet Carre is at 22.20 feet NGVD29 with the structure is closed. Therefore, to evaluate the proposed Labranche diversion under the conditions of maximum estimated spillway leakage, the low flow condition for these diversion simulations was selected as 10,000 cfs. The high flow condition is 250,000 cfs through the spillway, the design capacity of the structure. Roughness values and eddy viscosity coefficients were assigned to Mesh #5 based on the results of the previous calibration and sensitivity simulations. The four diversion simulations, with constant tailwater at mean sea level in Lake Pontchartrain, can be summarized as follows:

1. 10,000 cfs forcing in the Bonnet Carre Spillway, Bayou Labranche undammed
2. 10,000 cfs forcing in the Bonnet Carre Spillway, Bayou Labranche dammed
3. 250,000 cfs forcing in the Bonnet Carre Spillway, Bayou Labranche undammed
4. 250,000 cfs forcing in the Bonnet Carre Spillway, Bayou Labranche dammed
The results of both the wetland sensitivity simulations and the diversion simulations described immediately above were followed-up with a particle-tracking analysis to map total travel times and flow paths in the wetland. Particles were placed at six-inch intervals across the diversion channel where it enters the wetland at the lower guide levee of the Bonnet Carre Spillway (194 particles total). Their initial placement is shown in Figure 3.50.

![Initial Particle Placement](image)

**Figure 3.50: Initial Particle Placement for Diversion Simulations**

These particles were released once into the steady-state flow fields and advected through the wetland for five days at a constant timestep of ten seconds.
CHAPTER 4. RESULTS AND DISCUSSION

4.1 Calibration and Validation Simulations

Figures 4.1 and 4.2 show the results of the calibration simulations for Mesh #1 at the Reserve and Bonnet Carre gages, respectively. There is no plot for the third USACE gage in the model domain, at Carrollton, because this is where the tailwater elevations were forced for use as a boundary condition. Therefore, the simulated water surface elevation (WSE) at Carrollton always equals the observed. Each curve shown in Figures 4.1 and 4.2 represents a different combination of model parameters, namely Manning’s roughness, coefficient of eddy viscosity, and vorticity correction. The red markers in each plot represent the rating curve values that were fit to the 22 years of sampled data, with errorbars bounding +/- one standard deviation of all the observed stages (minus any outliers) at each flow rate. One notable feature of these curves is the fact that the errorbars are larger at low-to-medium flow rates, indicating higher variability and suggesting that these river stages are influenced by processes other than discharge, such as winds or tides.

It can be seen in both Figures 4.1 and 4.2 that the goodness-of-fit of the simulated rating curves to the observed data changes near 800,000 cfs. For lower flow rates, the curves representing the higher Manning’s roughness values match best with the observed stages. At higher flow rates the observed data curve changes slope, and matches better with the simulated stages at lower Manning’s roughnesses. This phenomenon would suggest that the bed friction in the actual river lessens with increasing discharge, and most notably when flow rates approach 800,000 cfs. The concept of discharge-varying roughness is noted by Chow (1959) and is explained by the fact that larger bedforms are
present on the river bottom at lower flow rates when the suspended sand in the water column settles out. These bedforms become a source of higher friction and energy loss until they are washed out at higher flow rates.
A notable point about the calibration results shown in Figures 4.1 and 4.2 is the lack of model sensitivity to the coefficient of eddy viscosity (EEV). Values ranging from 0.1 to 0.9 all produced results that fell along the same curve for constant Manning’s roughness and vorticity correction.

Table 4.1: RMSE Analysis of Calibration Simulations – Reserve Gage
(Parameters with Lowest RMSE Value for 300-1200 kcf s and 800-1200 kcf s)

<table>
<thead>
<tr>
<th>Model Parameters</th>
<th>RMSE – ft (300-1200 kcf s)</th>
<th>RMSE – ft (800-1200 kcf s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>n = 0.014</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity Off</td>
</tr>
<tr>
<td>n = 0.014</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity On</td>
</tr>
<tr>
<td>n = 0.016</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity Off</td>
</tr>
<tr>
<td>n = 0.016</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity On</td>
</tr>
<tr>
<td>n = 0.018</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity Off</td>
</tr>
<tr>
<td>n = 0.018</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity On</td>
</tr>
<tr>
<td>n = 0.020</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity Off</td>
</tr>
<tr>
<td>n = 0.020</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity On</td>
</tr>
<tr>
<td>n = 0.022</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity Off</td>
</tr>
<tr>
<td>n = 0.022</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity On</td>
</tr>
</tbody>
</table>

Table 4.2: RMSE Analysis of Calibration Simulations – Bonnet Carre Gage
(Parameters with Lowest RMSE Value for 300-1200 kcf s and 800-1200 kcf s)

<table>
<thead>
<tr>
<th>Model Parameters</th>
<th>RMSE – ft (300-1200 kcf s)</th>
<th>RMSE – ft (800-1200 kcf s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>n = 0.014</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity Off</td>
</tr>
<tr>
<td>n = 0.014</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity On</td>
</tr>
<tr>
<td>n = 0.016</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity Off</td>
</tr>
<tr>
<td>n = 0.016</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity On</td>
</tr>
<tr>
<td>n = 0.018</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity Off</td>
</tr>
<tr>
<td>n = 0.018</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity On</td>
</tr>
<tr>
<td>n = 0.020</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity Off</td>
</tr>
<tr>
<td>n = 0.020</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity On</td>
</tr>
<tr>
<td>n = 0.022</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity Off</td>
</tr>
<tr>
<td>n = 0.022</td>
<td>EEV = 0.1, EEV = 0.5, EEV = 0.9</td>
<td>Vorticity On</td>
</tr>
</tbody>
</table>

Tables 4.1 and 4.2 show the results of root-mean-square-error (RMSE) analyses of the calibration results graphically represented in Figures 4.1 and 4.2. A RMSE was
calculated for each combination of Manning’s roughness, coefficient of eddy viscosity, and vorticity correction that was tested in the calibration runs. The discrete data points that were used for the RMSE calculations were the simulated and observed river stages at each gage location for selected flow rates. In light of the goodness-of-fit differences at high and low discharges, two different RMSE values were calculated at each gage in order to compare the results. One RMSE was calculated from the simulated and observed stages at 300, 400, 500, 600, 700, 800, 900, 1000, 1100, and 1200 kcfs and another at only the higher flow rates of 800, 900, 1000, 1100, and 1200 kcfs. At the Bonnet Carre gage, the same model parameters produced the lowest RMSE (indicating the best fit to the observed data) values for (1) all flow rates and (2) just the high flow rates. However, a different combination of model parameters produced the lowest RMSE at the Reserve gage when just the stages at higher flow rates were used in the calculation as opposed to the stages at all flow rates. Since the desired model calibration for Mesh #1 is during times of spillway operation, the model parameters that produce the best fit at higher flow rates are needed. Therefore, the lowest RMSE for the 800-1200 cfs range is taken as the indicator for best fit, which is lower than the lowest RMSE for the 300-1200 cfs range in both cases.

The selected model parameters based on the RMSE analysis are highlighted in green in Tables 4.1 and 4.2. The model parameters highlighted in red in Table 4.1 are those that would have been selected if the RMSE for stages at all flow rates had been used as the indicator for best fit. The use of the vorticity correction is shown to produce the best fit, when a Manning’s roughness of 0.016 is used. Since the model is insensitive
to the EEV term, the median value of 0.5 was selected. As shown in Tables 4.1 and 4.2, these calibrated model parameters are the same at both gage locations.

Figures 4.3 – 4.5 show the time-series stage results for Validation Simulation #1 (blue), plotted with the observed river stages (green) and the rating curve values corresponding to the simulated flow rates (red) at the three gage locations.

Figure 4.3: Results for Mesh #1, Validation Simulation #1 at Carrollton

Figure 4.4: Results for Mesh #1, Validation Simulation #1 at Bonnet Carre
Figure 4.5: Results for Mesh #1, Validation Simulation #1 at Reserve

The inflow hydrograph (Figure 3.35) at Reserve for the simulated two-week period in Validation Simulation #1 consists of a rising period (01/01/10 – 01/10/10) from 800,000 to 875,000 cfs, followed by a more rapid falling period (01/10/10 – 01/15/10) from 875,000 to 750,000 cfs. During the beginning of the hydro-period, the observed WSE at Carrollton (to which the simulated WSE is a forced one-to-one match with the directly observed WSE) slightly over predicts the rating curve values, up until January 6 (Figure 4.3). After the peak the observed WSE at Carrollton under predicts the rating curve values, nearly to the point being outside the errorbars. This trend (of over predicting stages on the rising end and under predicting on the falling end of the hydrograph) is expected, based on the assumption that the bottom friction in the channel changes with discharge (Chow, 1959).

The simulated WSE at Bonnet Carre (Figure 4.4) very closely matches the observed WSE for the entire two-week simulation, only slightly over predicting after the flow peak. The ability of the model to so closely predict the observed stages at Bonnet
Carre is supportive of a claim that, at least for these simulated flow rates, the calibrated combination of model parameters adequately account for the energy losses (and therefore adequately predict the hydraulic gradients) between Bonnet Carre and Carrollton. With regard to the rating curve values, the trends in both the simulated and observed WSE’s at Bonnet Carre are the same as those noted at Carrollton.

The ability of the model to predict stages at Reserve (Figure 4.5) is a marked contrast with the model’s performance at Bonnet Carre (Figure 4.4). For the entirety of this validation run, the simulated WSE’s under predict both the observed and the rating curve values. The highest errors (of ~1 ft) between the simulated and observed stages occur during the rising period of the hydrograph and lessen (to ~0.5 ft) during the falling period; this trend is at least intuitive. What is immediately surprising about the errors at Reserve, given the fact that the model is able to very closely predict the total head losses over the twenty-four mile reach between Bonnet Carre (RM 127.1) and Carrollton (RM 102.8), is that they are generated over a relatively simpler stretch of the river that is less than half as long as the stretch between Bonnet Carre and Carrollton. The significant meander bends in the Mesh #1 model domain are shown in Figure 4.6, only two of which are located between the Bonnet Carre and Reserve gages.

Figure 4.6 is a 2008 aerial image of the Mississippi River, overlain with the boundary of Mesh #1 (green), the locations of USACE gages (blue), and the locations of meander bends (yellow). One striking contrast between the Reserve/Bonnet Carre and Bonnet Carre/Carrollton reaches is the high degree of sinuosity that characterizes the latter. By comparison, the reach between Reserve and Bonnet Carre is much shorter with many fewer bends.
Due to the relative differences in length and sinuosity, it is suspected that the two reaches (Reserve/Bonnet Carre & Bonnet Carre/Carrollton) bear dissimilar sensitivities to the two model parameters affecting energy losses, namely the Manning’s roughness coefficient and the vorticity correction. It is hypothesized that overall energy losses in the reach between Bonnet Carre and Carrollton are primarily due to the high number of bendways, while the losses between Reserve and Bonnet Carre are dominated by bed friction. This would explain the fact that at the flow rates simulated in Validation Simulation #1 (which are the flow rates that mark the transition zone in bed roughness as shown in the calibration results – Figures 4.1 and 4.2), the slightly low Manning’s coefficient of 0.016 is compensated for in the reach between Bonnet Carre and Carrollton by vorticity-induced energy losses, thereby producing very satisfactory stage results at Bonnet Carre. On the other hand, if the Manning’s roughness coefficient is unphysically low at these flow rates, there is no other significant source of head loss between Reserve and Bonnet Carre, and the simulated stages at Reserve are going to be underpredicted.
Figures 4.7 – 4.9 show the time-series stage results for Validation Simulation #2 (blue), plotted with the observed river stages (green) and the rating curve values corresponding to the simulated flow rates (red) at the three gage locations.

Figure 4.7: Results for Mesh #1, Validation Simulation #2 at Carrollton

Figure 4.8: Results for Mesh #1, Validation Simulation #2 at Bonnet Carre
Figure 4.9: Results for Mesh #1, Validation Simulation #2 at Reserve

The inflow hydrograph (Figure 3.36) at Reserve for Validation Simulation #2 consists of higher discharges than were simulated in Validation Simulation #1. Occurring within a month of the time period simulated in the first validation event, the goal of this second event was to test the validity of the model at flow rates of over 1,000,000 cfs.

At Bonnet Carre (Figure 4.8) the model stages match very well with the observed, containing a maximum error of +0.5 feet (simulated – observed) that only lasts for approximately a two-day time period between 02/09 and 02/11 (Figure 4.8). Throughout the simulation, both the simulated and observed stages remain within or very near the average variability based on the simulated discharge.

At Reserve (Figure 4.9) the simulated stage under predicts the observed by ~1 foot at the start of the simulation, when the flow rate is 800,000 cfs. This error reduces to within -0.5 feet (simulated – observed) as the discharge increases to 1,000,000 cfs.

The results of the two validation simulations show that the model performed well in both cases in predicting stages at the Bonnet Carre gage. They also show that, because
the results at Reserve were more realistic during the second event, the model is better suited to simulating higher flow rates, as was intended. The hypothesis stated in the discussion of Validation Simulation #1, that the performance of the model in predicting stages at Reserve is dependent primarily on Manning’s roughness while its performance at Bonnet Carre is dependent primarily on the vorticity correction, is upheld by the results of the second validation run. In both simulations, regardless of whether the discharge was 750,000 or 1,000,000 cfs, the model performed well at Bonnet Carre because the vorticity card was active. On the other hand, the model performance at Reserve in Validation Simulation #2 was much better, when the Manning’s roughness was more physically representative of the bed conditions at those higher flow rates.

Figures 4.10 – 4.12 show the results of the model calibration for Lake Pontchartrain using Mesh #2. Daily observed data was available for comparison at the Frenier (Figure 4.10) and West End (Figure 4.11) USACE gages. For the NOAA gage at New Canal (Figure 4.12), only predicted tides were available.

Figure 4.10: Results of Model Calibration for Lake Pontchartrain, Mesh #2 at Frenier
The results of these calibration simulations for Lake Pontchartrain show that the model is not sensitive to either the Manning’s roughness coefficient (within the tested range that was applicable to the region) or the coefficient of eddy viscosity. Figures 4.10
and 4.11 show the ability of Mesh #2 to propagate the tide, but its inadequacy in predicting water levels influenced by meteorological events (i.e. wind, rain), which is probably the cause of increased lake stages between 01/12 and 01/16. Comparing the simulated tidal signal at New Canal to the NOAA tidal prediction (Figure 4.12) allows for an assessment of the computed ADCIRC tidal constituents that were used to generate the model forcing. The simulated tidal signal can be characterized as being slightly out of phase and having a smaller amplitude than the NOAA prediction. However, it is not the objective of this modeling effort to replicate the hydrodynamics in Lake Pontchartrain, which serves only as a receiving basin for outflow from other parts of the model. It is merely the goal in these “calibration” runs to obtain a reasonable roughness value for Pontchartrain that will not affect flow coming out of the Bonnet Carre Spillway and Labranche wetland. Seeing as how the model within Lake Pontchartrain is insensitive to Manning’s roughness under tidal conditions, the median value (of those tested) of 0.02 was selected. Likewise, an EEV value of 0.5 was selected for Lake Pontchartrain.

Figures 4.13 and 4.14 show the simulated and observed stages and velocities, respectively, for the highest discharge – 243,000 cfs – that was used in calibrating the Bonnet Carre Spillway portion of Mesh #2. The results shown in these two figures are intuitive and expected, with increasing roughness values producing higher stages and lower velocities. In Figure 4.14, the model performs well in simulating the trend of higher velocities around the levees (Sta. 0 and 6000), but does not adequately capture the lower velocities in general in the western portion of the cross-section. The red curve in Figure 4.13, which represents the USGS depth measurements along the Highway 61 bridge, is an averaged value that was computed in order to make quantitative WSE
comparisons with the simulation results. There was some degree of uncertainty in analyzing the USGS data, due to (1) the stationing values along the transect that referenced an unknown starting point (assumed to be the westernmost bridge abutment) and (2) the unknown floodway elevation corresponding to each depth at the time of the measurements. Therefore, the measured depths were added to the model floodway elevations at the given stationing distance from the westernmost bridge abutment. These values, which fluctuated mildly, were averaged across the bridge section to obtain a single value, plotted as the “Average Measured WSE,” similar to that shown in Figure 4.13. These single values for the “Average Measured WSE’s” are the discrete observed points shown in Figure 4.15.

**Figure 4.13: Results of Model Calibration for the Bonnet Carre Spillway, Mesh #2**

Simulated and Observed Stages at 243,000 cfs – Highway 61 Transect

Figure 4.15 shows the simulated and observed stage vs. discharge relationships that were used in selecting the model parameters. For each of the simulated rating curves, the RMSE was calculated, as shown in the legend in Figure 4.15.
Figure 4.14: Results of Model Calibration for the Bonnet Carre Spillway, Mesh #2
Simulated and Observed Velocities at 243,000 cfs – Highway 61 Transect

Figure 4.15: Results of Model Calibration for the Bonnet Carre Spillway, Mesh #2
Simulated and Observed Stage vs. Discharge (1997 & 2008 Events)

All of the USGS data from the 1997 and 2008 events were plotted in Figure 4.15, even though only five of the highest discharges from both events were simulated. The observed data from the 1997 and 2008 openings appear to fall along two distinct curves,
separated by approximately a two-foot difference. However, since the simulated discharges for the model calibration were sampled from both events, the simulated rating curve with the lowest RMSE ($n = 0.060$) falls in the middle, and therefore represents a good average, of both events. As with previous simulations, the model showed no sensitivity to the EEV term. To reduce the number of calibration simulations, they were all performed with the vorticity card off. But a subsequent test simulation at 243,000 cfs with model parameters [$n=0.06$, EEV=0.5, Vorticity On] produced a stage difference of ~0.15 feet at the Highway 61 bridge location in comparison with the model parameters [$n=0.06$, EEV=0.5, Vorticity Off], which was deemed to be a negligible difference. Therefore, the calibrated model parameters for the Bonnet Carre Spillway were selected as [$n=0.06$, EEV=0.5, Vorticity On].

4.2 Diversion Simulations

The results of the calibration attempt (by varying the submerged roughness values between $n=0.02$ and $n=0.06$) in the Labranche wetland under tidal forcing (Figure 3.47) with Mesh #3 were inconclusive in that the simulated velocities were found to be nearly zero at all simulation times at the six ICG railroad openings where Comite Resources gathered measurements. The recorded velocities through the railroad openings for the 02/15/05 calibration date ranged from 1.00 to 2.25 ft/sec, which are near the average for the days that were sampled (Figure 3.44). Yet, the recorded depths beneath the ICG railroad on 02/15/05 (Figure 3.43) range from 0.5 to 5.0 feet lower than on some of the other measuring dates. The fact that the measured velocities are not necessarily correlative to the measured depths suggests that wind may have played a large role in producing surface velocities in Labranche on this measuring date. Of the pump stations
that discharge into Labranche, stage data upstream and downstream of Pumps 2 and 3 (Figure 2.4) are available online (www.rivergages.com). However, neither of the flow records extend as far back as 2005. In weighing the options of obtaining, processing, and incorporating into the model the other possible field conditions that existed on measurement dates (including wind, precipitation, pumped discharge, and tides) it was questioned whether these additional model forcings would produce significant depth-averaged velocities that could be used to set the proper bottom friction values in the model. Therefore, it was decided that a sensitivity analysis would be more beneficial than additional calibration attempts, by which the importance of Manning’s roughness in obtaining a unique result could be gauged.

For the four sensitivity simulations (Table 3.8), a slug of particles released in the diversion channel (Figure 3.50) were tracked through the steady-state flow field. Figures 4.16 to 4.23 show the particles paths (with color-contoured travel times) and particle breakthrough curves for each of the six ICG railroad openings. The goal of these simulations was to test the effect of different combinations of Manning’s roughness in the wetland on the flow paths, particle dispersion, travel times, and exit locations from the south side of the railroad.

For the particle tracking simulations in the sensitivity analysis, not all particles are accounted for in the breakthrough curves (Figures 4.17, 4.19, 4.21, 4.23), indicating retention within the system. Some particles, because of their initial placement near the sidewalls of the diversion channel, became trapped within the boundary layer and were never advected out. Other particles were carried into portions of the wetland where velocities slowed to zero, and some were run aground.
Figure 4.16: Results of Sensitivity Simulation #1 ($n_{OV} = 0.060$, $n_S = 0.060$), Mesh #4
Particle Trajectories and Residence Times at 4,000 cfs

Figure 4.17: Results of Sensitivity Simulation #1 ($n_{OV} = 0.060$, $n_S = 0.060$), Mesh #4
Particle Breakthrough Curves at ICG Railroad
Figure 4.18: Results of Sensitivity Simulation #2 ($n_{OV} = 0.060$, $n_S = 0.010$), Mesh #4
Particle Trajectories and Residence Times at 4,000 cfs

Figure 4.19: Results of Sensitivity Simulation #2 ($n_{OV} = 0.060$, $n_S = 0.010$), Mesh #4
Particle Breakthrough Curves at ICG Railroad
Figure 4.20: Results of Sensitivity Simulation #3 ($n_{OV} = 0.030$, $n_S = 0.060$), Mesh #4 Particle Trajectories and Residence Times at 4,000 cfs

Figure 4.21: Results of Sensitivity Simulation #3 ($n_{OV} = 0.030$, $n_S = 0.060$), Mesh #4 Particle Breakthrough Curves at ICG Railroad
The captions of Figures 4.16 to 4.23 give values for $n_{OV}$ and $n_S$, denoting the Manning’s roughness values that were used for the overland and submerged areas in each
simulation, respectively. It can be seen by comparing Figures 4.17 and 4.21 to Figures 4.19 and 4.23 that lower Manning’s roughness in the channels promotes the short-circuiting of larger numbers of particles through Bayou Labranche and overall shorter retention times in the wetland. For high Manning’s roughness in the submerged/channel regions, the incoming flow from the diversion has no incentive (via lower resistance) to enter the bayous and therefore continues on its overland path. The most uniform dispersion of particles to the different railroad exits occurs when both the overland and channel roughnesses are high (Figure 4.17), and least uniform dispersion (highest short circuiting) occurs when both the overland and channel roughnesses are low. The greatest sensitivity to maximum breakthrough time is Tie Ditch, which shows 6% total breakthrough after 2.5 days for [high overland roughness / low submerged roughness] and the same total breakthrough after 1.5 days for [low overland roughness / low submerged roughness]. In summary, the overall particle trajectories and residence times were found to be sensitive to different combinations and magnitudes of overland and channel bottom roughness. Increases in channel depths at the six railroad crossings for each of the sensitivity runs are shown in Table 4.3.

**Table 4.3: Stage Increase (ft) For Wetland Sensitivity Simualations at 4,000 cfs**

<table>
<thead>
<tr>
<th>Channel</th>
<th>Sim #1 n_OV=0.060 n_S=0.060</th>
<th>Sim #2 n_OV=0.060 n_S=0.010</th>
<th>Sim #3 n_OV=0.030 n_S=0.060</th>
<th>Sim #4 n_OV=0.030 n_S=0.010</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bayou Labranche</td>
<td>1.58</td>
<td>0.56</td>
<td>1.33</td>
<td>0.55</td>
</tr>
<tr>
<td>Fall Canal</td>
<td>1.58</td>
<td>0.73</td>
<td>1.34</td>
<td>0.74</td>
</tr>
<tr>
<td>Tie Ditch</td>
<td>1.53</td>
<td>0.70</td>
<td>1.30</td>
<td>0.70</td>
</tr>
<tr>
<td>Pipeline Canal</td>
<td>1.42</td>
<td>0.66</td>
<td>1.31</td>
<td>0.69</td>
</tr>
<tr>
<td>16-Mile Trestle</td>
<td>1.15</td>
<td>0.55</td>
<td>1.24</td>
<td>0.57</td>
</tr>
<tr>
<td>Walker Canal</td>
<td>1.03</td>
<td>0.47</td>
<td>1.25</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Overall, increases in water depths within the channels (Table 4.3) as a result of the freshwater introduction into Labranche are higher for high overland roughness values.
The greatest sensitivity to stage is observed in Bayou Labranche, where water depths are higher by over a foot for Simulation #1 (high overland roughness / high submerged roughness) as opposed to Simulation #4 (low overland roughness / low submerged roughness).

Broad comparisons can be made between these simulation results and those of a previously performed HEC-2 simulation (USACE, 1996) in the Labranche wetland. The goal of the USACE one-dimensional HEC-2 modeling effort was to determine the increase in stage levels at the six ICG Railroad openings under a steady-state inflow forcing of 4,000-cfs (the design capacity of the proposed diversion). The roughness values that were used in the HEC-2 simulations are unknown, but simulated increases in total water depth at the channel locations above (Table 4.3) were documented as exceeding 0.50 feet. For the ranges of feasible roughness values, represented by Sensitivity Simulations #1-4, a minimum stage increase of 0.50 feet in nearly all of the channels is seen (with the exception of Walker Canal, Simulation #2). Therefore, the overall conclusions of the two model studies are in agreement with regard to inundation levels, but no other detailed comparisons can be made.

After completion of the sensitivity analysis, a channel bottom roughness of 0.020 was selected and assigned to the bathymetric portions of the Labranche wetland. An overland roughness value of 0.050 was assigned to all visible land. Given that the calibrated roughness coefficient for the Bonnet Carre Spillway was 0.060, and the density of tree cover in the spillway appears much greater than in the wetland, this estimate of 0.050 seems reasonable. Figures 4.24 – 4.29 show the particle trajectories, residence times, and breakthrough curves for the diversion simulations with Mesh #5.
Figure 4.24: Results of Diversion Simulation #1, Mesh #5
Particle Trajectories and Residence Times at 10,000 cfs – Bayou Labranche Open

Figure 4.25: Results of Diversion Simulation #2, Mesh #5
Particle Trajectories and Residence Times at 10,000 cfs – Bayou Labranche Closed
Figure 4.26: Results of Diversion Simulations #1 & 2, Mesh #5
Particle Breakthrough Curves at ICG Railroad – 10,000 cfs

Figure 4.27: Results of Diversion Simulation #3, Mesh #5
Particle Trajectories and Residence Times at 250,000 cfs – Bayou Labranche Open
Figure 4.28: Results of Diversion Simulation #4, Mesh #5
Particle Trajectories and Residence Times at 250,000 cfs – Bayou Labranche Closed

Figure 4.29: Results of Diversion Simulations #3 & #4, Mesh #5
Particle Breakthrough Curves at ICG Railroad – 250,000 cfs
At 10,000 cfs in the Bonnet Carre Spillway (Diversion Simulations #1 & #2), the diversion channel diverts 850 cfs into Labranche. Because the flow is so low, it becomes channelized very easily at Bayou Labranche and mostly short-circuits to Lake Pontchartrain within approximately 1.5 days (Figures 4.24 and 4.26). After the simulated damming of Bayou Labranche, the low flow velocities that are forced overland take longer to get to Fall Canal (approximately 2.25 days) but short-circuit through that channel once they do (Figure 4.25 and 4.26). At 250,000 cfs in the Bonnet Carre Spillway, the diversion channel captures 17,500 cfs, which is enough to flood the entire wetland south of the railroad. Particles are dispersed as far east as Walker Canal and are retained within the system for approximately two days, but particles on the west side get flushed out within the first day. The damming of Bayou Labranche does not significantly affect the flow paths (Figures 4.27 and 4.28) or travel times (Figure 4.29) on the western side of the wetland at high flow, but it does increase total breakthrough time by ~0.25 days for particles exiting through Walker Canal (Figure 4.29).

Figure 4.30 shows the WSE results in the Labranche wetland for Diversion Simulations #1 and #3 (10,000 cfs and 250,000 cfs, respectively, in the Bonnet Carre Spillway with Bayou Labranche open).

Figure 4.30: Simulated Inundation in the Labranche Wetland for (Left) Diversion Simulation #3 (250,000 cfs – Bayou Labranche Open) and (Right) Diversion Simulation #1 (10,000 cfs – Bayou Labranche Open)
It can be seen in Figure 4.30 that for the high flow case (left), WSE’s are approximately 6 ft (NAVD88) near the diversion channel and approximately 5 ft everywhere else south of the railroad. North of the railroad, the average WSE is approximately 4 ft, indicating a buildup of water behind the railroad, which is gradually released through the 6 openings. For the low flow case (right), there is no buildup of water behind the railroad, and WSE’s are at a fairly constant 4 ft throughout the entire wetland.

Figure 4.31: PTM Results for Entire Wetland (Top), PTM Results at Area of Interest (Bottom Left) and Velocity Results for Area of Interest (Bottom Right)

Diversion Simulation #3 (250,000 cfs in the Spillway – Bayou Labranche Open)

Figure 4.31 shows the particle-tracking and velocity results for the region south of the railroad between Bayou Labranche and Fall Canal. This area is of notable interest because it is a region where many particles were trapped. For the PTM analysis of Diversion Simulation #3, 56% of the total particles released were retained behind the
railroad, and a significant portion of these appear to be trapped in the region shown in Figure 4.31 (bottom left). It can be seen in Figure 4.31 (bottom right) that the velocities at the railroad drop to zero, in between the stronger currents at Bayou Labranche and Fall Canal. Therefore, the particles that drift in between those two channels get pushed all the way up to railroad, where they are caught in the boundary layer (where the velocity is zero) and are not advected further. The lower velocities between Bayou Labranche and Fall Canal may indicate a possible retention zone with higher residence times for freshwater, nutrients, and sediment.

![Figure 4.32: Proposed Marsh Nourishment in Labranche (CWPPRA Project PO-75)](image)

![Figure 4.33: PTM (Left) and Velocity (Right) Results in CWPPRA Project Area for Diversion Simulation #3 (250,000 cfs in Bonnet Carre – Bayou Labranche Open)](image)
Another area of interest in the wetland is the site of the CWPPRA project PO-75, shown in the USGS map in Figure 4.32. Being aware of the other restoration proposals in Labranche, it is important to analyze the model results in light of those efforts. For the CWPPRA PO-75 project area, it can be seen that the existing conditions of the site are such that it receives inflow during Diversion Simulation #3, with travel times of approximately three hours for particles that are routed through the area. One possible conclusion of these results is that, if the nourished project site were graded so as to not completely obstruct flow, the created marshland would benefit from freshwater circulation during high flow events. Thus, a freshwater diversion into Labranche could add value to the marsh nourishment project, sustaining the area through the delivery of vital resources.

![Simulated Velocity Contours](image)

**Figure 4.34: Simulated Velocity Contours at High Flow in the Bonnet Carre Spillway Without (Left) and With (Right) the Labranche Diversion Channel**

Turning the attention from the wetland to the Bonnet Carre Spillway, the effects of the diversion on the spillway hydraulics can be analyzed. Figure 4.34 shows the velocity magnitude contours in the spillway with and without the diversion channel. The
image without the diversion channel (Figure 4.34, left) was taken from the calibration simulation for the spillway at 243,000 cfs. The image with the diversion channel (Figure 4.34, right) was taken from Diversion Simulation #3 (250,000 cfs, Bayou Labranche open). This 7,000 cfs difference is not significant in terms of the simulated spillway velocities, and little change is seen between the two images. This figure shows that, at high flow, the largest velocities occur at the entrance and exit of the spillway, and that velocities are lowest between the two borrow canals north (lakeside) of Highway 61 and before Lake Pontchartrain. It can be seen that the velocities in the spillway at high flow are not affected by the Labranche diversion, except in the immediate vicinity of the diversion channel.

From the hydrodynamic model results, insight may be gained into possible areas of erosion and deposition in the spillway and wetland.

![Figure 4.35: Rouse Number Contours for Medium Sand (250 µm) at 243,000 cfs Without Labranche Diversion](image)

(Rouse # > 1 Possible Deposition, Rouse # < 1 Possible Suspension)
Figures 4.35 and 4.36 show contours for the Rouse numbers calculated at each mesh node from the hydrodynamic model results. The Rouse number (Julien, 1998) is the ratio of a particle’s settling velocity to the bed shear velocity and may be taken as an index of particle suspension/deposition. The images in Figures 4.35 and 4.36 are contoured with a threshold value of 1.00, indicating possible depositional areas in blue and areas of likely particle suspension in red. It can be seen (Figure 4.35) that the areas of likely suspension in the spillway for medium sand, based on the Rouse number calculation, are in the borrow canals and where the spillway empties into Lake Pontchartrain. Comparing these areas with the velocity contours in Figure 4.34, the areas of predicted particle suspension are where velocities are the highest. Comparing Figures 4.35 and 4.36, the Labranche diversion channel does not affect the predicted sedimentation trends in the spillway, and basically all areas in the wetland are suitable for the deposition of medium sand. During
the 1997 spillway opening, the field data shows that only 12.4 ± 11.1% of the total suspended sand load (larger than 63 μm) that entered the Bonnet Carre Spillway reached the Highway 61 bridge (Allison & Meselhe, 2010). This indicates that most sand deposition takes place within the first third of total spillway length, which is in agreement with the potential depositional trends shown for 250 μm sand in Figure 4.35.

4.3 Conclusions and Future Work

The two-dimensional finite element code ADH was used to simulate the introduction of freshwater into the Labranche wetland via a proposed diversion channel from the Bonnet Carre Spillway. Model calibration, validation, and sensitivity analyses were performed with separate portions of the mesh before actually simulating the proposed diversion scenarios. The diversion scenarios that were selected for simulation represented extreme conditions for spillway and diversion operation. This was done so as to estimate the widest range of effects in implementing this proposed diversion. At a low flow condition in the spillway (10,000 cfs), approximately 850 cfs is captured by the diversion, which short-circuits directly through Bayou Labranche and exits the wetland in just over a day and a half. At high flow in the spillway (250,000 cfs), approximately 17,500 cfs is captured by the diversion channel, and a significant portion short-circuits through Bayou Labranche within 0.5 days. For low flow in the spillway, the simulated damming of Bayou Labranche increases retention times in the wetland, but does not promote dispersion of flow past Fall Canal. At high flow in the spillway, the simulated damming of Bayou Labranche does not significantly affect either the flow dispersion or retention times. The flow that would otherwise exit the wetland through Bayou
Labranche finds the next closest opening at Fall Canal and exits in relatively the same amount of time.

Rouse numbers in the spillway, wetland, and lake at high flow rates indicate that the deposition potential of medium sand is high everywhere in the system except within the spillway canals, the diversion channel, and the downstream-most end of the spillway.

One of the goals of this thesis work was to generate a fully connected ADH model of the Mississippi River, Bonnet Carre Spillway, Labranche wetland, and Lake Pontchartrain. While this was not accomplished, and the Mississippi River was unable to be used in the diversion simulations, insights were gained in the calibration and validation simulations of the river that provide a better understanding of energy losses between Reserve and Carrollton at different flow rates. This understanding could be valuable in future work, if the current model were to be used for a Labranche diversion directly from the Mississippi River downstream of the Bonnet Carre Spillway.

Information is presented in the early chapters of this thesis, which describes the ability of river diversions (and more specifically, the Bonnet Carre Spillway) to deliver freshwater, nutrients, and sediment to their respective receiving areas. A comprehensive modeling study should likewise address the hydrodynamics and constituent transport within the system. The model presented in this thesis does not do that; it is simply a hydrodynamic model that does not simulate transport of nutrients or sediment. However, ADH is equipped with built-in transport routines, and the respective cards within the code need only be activated to simulate nutrient and sediment transport in the model that was presented herein. Knowing that the opportunity exists for doing so, any transport simulations will be left for future work.
REFERENCES


CWPPRA. (2011). *Opportunistic Use of the Bonnet Carre Spillway PO-26*.


CWPPRA. (2007). *PPL17 Project Nominee Fact Sheet*.


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VITA

Josef Hoffmann was born and raised in Baton Rouge, Louisiana. One of his earliest memories is of watching his older sister fall head-first into a large outdoor fountain while vacationing with the family in Graceland, Tennessee. She emerged unharmed, but witnessing the accident instilled in Josef a cautious respect of water and had, seemingly, a profound influence on his later choice of career. At the age of eighteen, Josef began undergraduate studies in civil engineering at Louisiana State University, where he developed an interest in water resources and coastal engineering. He received a bachelor’s of science degree in civil engineering from LSU in 2009 and immediately began graduate studies at the same university, focusing his research efforts on hydrodynamic modeling. After receiving a master’s of science degree in the summer of 2011, Josef looks forward to beginning his career as a water resources engineer. He does not plan to seek professional licensure in the state of Tennessee in the immediate future.