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3-D Hydrodynamic and Non-Cohesive Sediment Transport Modeling in the Lower Mississippi River

A Thesis

Submitted to the Graduate Faculty of the University of New Orleans in partial fulfillment of the requirements for the degree of

Master of Science In Engineering Civil & Environmental Engineering

by

Grecia Alejandra Terán González

B.S. University of Zulia, Venezuela, 2008

May 2014

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DEDICATION

I want to dedicate this achievement to my family,

Migue, Vero and Gabo you opened the doors of your home and took me there for all this time helping me to accomplish this important goal in my life.

Mami, here and in the distance you have always been there to guide me and comfort me.

Migla, you have always believed in me and encouraged me to trust in my potential.

Elio, my love, you were there unconditionally through these years supporting me.

I also wish to dedicate it to my friends,

From the distance, Mate, you always supported me and helped me.

Here, Luis, Tatiana and Sina, you guys became an important part of this journey; you gave me a hand when most needed, among many other invaluable experiences.

Thank all of you for all the love you have always given me, without it I would not be who I am today, I love you with all my heart.

ACKNOWLEDGEMENTS

In first place, I would like to thank Dr. John Alex McCorquodale for giving me the honor of working under his advisory. This opportunity was a blessing for me. I really appreciate all the effort and patience you addressed to teaching me giving me part of your amazing knowledge and wisdom.

This research was funded in part by National Science Foundation (NSF) as a part of the Northern Gulf Coastal Hazards Collaboratory (NG-CHC; http://ngchc.org).

I also want to acknowledge Coastal Protection and Restoration Authority (CPRA) for funding part of this research study.

It is also important for me to thank the Lake Pontchartrain Basin Foundation and Dr. John Lopez for providing resources needed in this research study.

I want to thank Deltares for providing me with the software license to carry out this research.

I want to give a special thanks to Dr. Joao Pereira, who was at my first steps in the Master's program the person who gave me the best advice and taught me the main tools needed to become a modeler. He also helped me overcome, if not solved, many of the problems I faced within my research project. Besides from all of that, he is a good friend, who despite the distance I will never forget.

I also wish to thank Dr. Meselhe for his cooperation and important participation in the development of this study.

I would like to express my gratitude to Dr. Mead Allison for providing me with observed data that was necessary for the accomplishment of this project.

I also want to thank the US Army Corps of Engineers, Gary Brown and Steve Ayers for the observed data.

I want to thank Dr. Chen and Dr. Hu for having the kindness of providing me data for the hurricane runs.

I wish to thank my team workers, Jed and Tshering, mostly during the last stage of this research, for the help.

Finally but not least, I want to show my appreciation to my committee members, Dr. La Motta and Dr. Cothren for the knowledge you have imparted to me during this academic goal.

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LIST OF SYMBOLS

Symbol	Description	Units
q _s	Total bed-material load	L^3/TL
qsb	Bed load	M/L^2
q _{ss}	Suspended load	L^2/T
σ	Sigma vertical coordinate	Dimensionless
Ζ.	vertical co-ordinate in physical space	L
ζ	free surface elevation above the reference plane	L
d	depth below the reference plane	L
Н	total water depth	L
U	depth-averaged velocity in ξ -direction	L/T
V	depth-averaged velocity in η-direction	L/T
C	coefficients used to transform curvilinear to rectangular	
$\sqrt{u_{\eta\eta}}$	coordinates	L
Gre	coefficients used to transform curvilinear to rectangular	
$\sqrt{2\xi\xi}$	coordinates	L
Q	global source or sink per unit area	L/T
<i>qin</i>	local source per unit volume	T_{1}^{-1}
<i>qout</i>	local sink per unit volume	T^{-1}
P _	precipitation	L/T
E	evaporation	L/T
M_{ξ}	source or sink of momentum in ξ -direction	L/T^2
M_{η}	source or sink of momentum in η -direction	L/T ²
и	flow velocity in the ξ -direction	L/T
V	fluid velocity in the η-direction	L/T
W	fluid velocity in $σ$ -direction	L/I L^2/T
$V_{\rm V}$ back	hashanan duartiash mining as officiant	L/I L^2/T
$v_{\rm V}$	background vertical mixing coefficient	L/I L^2/T
V _{mol}	kinematic viscosity (molecular) coefficient	L / I
	direction	\mathbf{L}^2/\mathbf{T}
V _{3D}	andient hydrostatic pressure in ^k direction	L/I $M/I^2 T^2$
Γζ D	gradient hydrostatic pressure in p-direction	$M/L^2 T^2$
Γ_{η}	turbulant momentum flux in ^E direction	I / T^2
Γ_{ξ}	turbulent momentum flux in p direction	L/T L/T^2
Γ_{η}	velocity in the σ -direction in the σ -coordinate system	L/T
701	fluid velocity in $\sigma_{-direction}$	L/T I/T
и Р	hydrostatic water pressure	M/LT^2
a	acceleration due to gravity	L/T^2
9 0	density of water	M/T^3
$\frac{\rho}{\tau}$	shear stress	M/T^2
	reference density of water	M/T^3
T_{EE} , T_{nn} , T_{En}	contributions secondary flow to shear stress tensor	M/LT^2
$\widehat{\Pi}$	velocity of water discharged in ξ -direction	L/T
Ŵ	velocity of water discharged in n-direction	L/T
, Dн	horizontal diffusion coefficient	L^2/T
D_V	vertical diffusion coefficient	L^2/T
λ_d	1 st order decay process	T^{-1}
λ_d	1 st order decay process	T^{-1}

S	source and sink terms per unit area due to discharge	T^{-1}
L	length	L
κ	turbulent kinetic energy	M^2/T^2
$F_L(Ri)$	damping function	
\mathbf{D}_k	energy dissipation term	M^2/T^3
\mathbf{P}_{k}^{κ}	production term in transport equation for turbulent kinetic	M^2/T^3
ĸ	energy	
P _{Inu}	production term due to wave action	M^2/T^3
\mathbf{B}_{k}	Buovancy flux term in transport equation for turbulent	M^2/T^3
Σ_{k}	kinetic energy	
e	dissipation in transport equation for turbulent kinetic	M^2/T^3
0	energy	101 / 1
c'	constant in Kolmogoroy-Prandtl's eddy viscosity	
$c \mu$	formulation	Dimensionless
7'	vertical co-ordinate	I
	the total depth-averaged due to wave breaking	I
D_w	density of the water	\mathbf{M}/\mathbf{T}^3
p_w H /2	root mean square wave height	IVI/ I I
$\Pi_{rms}/2$	Drondtl Schmidt number	L
$O_{ ho}$	constant relating mixing length turbulant kinetic energy	Diffensioness
0	and dissingtion in the k a model	Dimonsionloss
c_D	allu dissipation in the K- E model	Dimensionless
C_{μ}	friction valority at the had	
u_{*b}	unction velocity at the bed	L/ I I /T
u_*	vertically averaged inclion velocity	L/I I
Δz_b	distance to the computational grid point closest to the bed	
z_0	bed roughness length	
u_{*s}	friction velocity at the free surface	L/I
D_*	non-dimensional particle diameter	Dimensionless
D_{50}	diameter of sediment	L
T	dimensionless bead shear	Dimensionless
$ au_{bc}$	critical bed shear stress	M/LT ²
$C_{g,90}$	Chézy coefficient	Dimensionless
θ_{cr}	Shields parameter	Dimensionless
C_a	reference concentration	M/L^3
q	depth averaged velocity	L/T
h	water depth	L
f_{cs}	shape factor	Dimensionless
ξς	the reference level or roughness height	L
Z_{c}	the suspension number	Dimensionless
	the vertical sediment mixing coefficient for sediment	
۶ ^(ℓ)	fraction	Dimensionless
$\mathcal{E}_{s}^{(\ell)}$	vertical fluid mixing coefficient	Dimensionless
ρ	Van Diin's "hate" faster for the adiment fraction	Dimensionless
р D	van Kijn's beta lactor for the sediment fraction	Dimensionless
r	flow rote	L J^{3}/T
$\mathcal{U}_{\mathcal{L}}$	now rate	L/I Dimonstrul
J_s	sin factor used to incorporate sediment effect	Dimensionless
D_{50}	median grain size	
V	velocity	L/ I
К	nyaraulic radius which represents the depth	L

Α	cross-sectional area	L^2
O_i	observed value	-
P_i	modeled value	-
\overline{O}	average of the observed value	-
\overline{P}	average of the modeled value	-
Ν	number of observations	Dimensionless

ABSTRACT

The purpose of this research is to develop a 3-D numerical model on the Lower Mississippi River to simulate hydrodynamics and non-cohesive sediment transport. The study reach extends from Bonnet Carré Spillway (RM 127) to Head of Passes (RM 0). Delft3D with sigma coordinates was selected as the river modeling tool. This model River domain is characterized by a complex distributary system that connects the Mississippi River to the Gulf of Mexico. The boundary conditions were: water levels in the Gulf and Head of Passes; and discharges upstream. For the calibration, there are observed data for both types of boundary conditions. Several periods of high discharge were simulated to compare water level, discharge, velocity profiles and sediment transport with measurements and accomplish calibration and validation of the model. A calibrated 3-D model has been developed with the following %RMSE: 5% for stage; 6% for discharge; and 5% for sand load.

Keywords: 3-D numerical modeling, Hydrodynamic simulation, Sediment transport, Lower Mississippi River, Distributary Flows

1. INTRODUCTION

1.1 General

The Mississippi River is one of the major rivers of the United States. For centuries it has been a natural resource that has been used for industrial and economic purposes. As it approaches the Gulf of Mexico, it creates a large delta, covering approximately 13,000 square miles. The installation of flood protection systems such as levees, along with dams and navigation works have negatively affected the replenishment of sediment in the delta. A large amount of sediment (up to 120 million tons per year) is transported into the Gulf of Mexico instead of going to the wetlands depriving them of sediment (Allison & Meselhe, 2010; Parker & Sequeiros, 2006).

The Mississippi River is a complex system and finding solutions to the restoration of the delta and adjacent wetlands is a very complicated task (CPRA, 2012). However, numerical modeling can be used as a tool in studying the behavior of the lower Mississippi River, through the analysis of hydrodynamics and sediment transport along the modeling domain (Meselhe, et al., 2005).



Figure 1. Modeling Domain from Bonnet Carré to Head of Passes (Visible Earth, 2001)

This research project presents a three-dimensional model of the Lower Mississippi River reach extending from Bonnet Carré (RM 127) to the Head of Passes (RM 0) as shown in Figure 1. Along the reach, there are no mayor inflows but there are numerous outflows such as West Bay and Main Pass, and the reach downstream of Bohemia (RM 47) on the east bank of the River where there is a natural levee that overtops in periods of high flow.

This study focuses on the development of a three dimensional numerical model that predicts the hydrodynamics and the non-cohesive sediment transport on the Lower Mississippi River. Delft3D (Deltares, 2011), a 3-D finite volume, orthogonal curvilinear grid, hydrodynamic and sediment transport computer software will be used for the 3-D modeling of the river domain.

1.2 Problem Statement

The use of computational models to replace physical models to study the hydrodynamics and sediment transport in environments such as rivers, lakes and coastal areas is a relatively recent approach but it is a very attractive tool. The computation of solutions for this kind of model involves solving continuity, momentum and energy equations along with differential equations for sediment continuity bringing the advantage of adaptability into the different physical domains compared to what a physical model can provide. Moreover, numerical models are not subjected to distortion effects as many physical models while being able to solve the equations for the same flow conditions as the ones observed in the field (Papanicolaou, Mohamed , Krallis, Prakash, & Edinger, 2008).

The modeling of sediment transport in particular is a very challenging task. It is a very complex process that requires experimental, field and numerical studies in order to accurately predict bed load and suspended load, interaction between turbulence, sediment transport in unsteady flows, among other important parameters (Barkdoll & Duan, 2008).

Furthermore, the Lower Mississippi River is a very unique domain. For high flow periods, the river bed follows a non-cohesive sediment bed behavior, which must be modeled under particular formulations in order to calculate erosion and deposition patterns (Pereira, 2011). Under low flow conditions the cohesive sediment regime is more important. Some issues facing managers of the Lower Mississippi River are: a) river stage and potential flooding, b) erosion and shoaling that may impact navigation, and c) saltwater intrusion.

2

The effect of new outlets/diversions on these issues requires the predictive capability of various types of models ranging from 1-D to 3-D models.

The Delft3D-FLOW module (Deltares, 2011) will be used to simulate the hydrodynamics and non-cohesive sediment transport in the Lower Mississippi River. The non-cohesive sediment transport simulations will be performed by the implementation of the Van Rijn (1984) formulation.

1.3 Objective

The main objective of this research project is to develop a Delft3D three dimensional hydrodynamics and non-cohesive sediment transport model for the Lower Mississippi River that is capable of simulating the river response to large diversions.

Another important objective is to achieve more independence from other models for future boundary conditions. This model utilizes stage values as boundary conditions for the major outlets to the Gulf of Mexico. Stage is preferable to discharge boundary conditions since is available from monitoring stations and/or sea level rise models.

Finally, the model will be tested for applicability under storm surge hurricane conditions in the area evaluated.

2. LITERATURE REVIEW

2.1 Background Research

A three-dimensional morphological sediment transport model was developed by Lesser et al. using the Delft3D-FLOW module, a multidimensional hydrodynamic and transport model that calculates non-steady flow and transport phenomena (Deltares, 2011). Different cases were evaluated, simulating a straight flume, a curved flume and cases applied to wave and current flume and the Ijmuiden harbor area among other experiments. It was found on the validation studies a response on different important processes as entrainment, transport, settling of sediment, varying levels of uniform bed shear stress, bed slope effects, among others. They also established that further evaluation on the model had to be done due to some special cases evaluated showing high sensitivity to the bed roughness changes (Lesser, Roelvink, van Kester, & Stelling, 2004).

A CH3D-SED three-dimensional hydrodynamic model was developed to compute sediment transport, erosion, and deposition in sand-bed rivers. This model was found to be well suited for predicting erosion and deposition patterns in bends, distributaries, and thalweg crossings between bends. The model was validated for the hydrodynamics and sediment transport simulations for several reaches of the Mississippi River. They found, for instance, at Red Eye Crossing (RM 223) a 13% difference between their predicted values and observations for sediment deposition. Moreover, for one of the models, at Head of Passes, they reproduced successfully the flow distribution, and found good agreement for observed and predicted velocities and suspended sediment concentrations (Gessler, et al., 1999).

Pereira developed a three dimensional ECOMSED and a one dimensional CHARIMA unsteady flow mobile-bed model of the Lower Mississippi River from Belle Chasse (RM 76) to Main Pass (RM 3) to simulate river currents, diversion sand capture efficiency, erosional and depositional patterns with and without diversions. Also, the introduction of new diversions at different locations with different geometries and outflows was studied. He found that the smaller diversions had little impact on the downstream sand transport but larger diversions had important effects such as the reduction in the slope of hydraulic grade line, available energy for transport along channels, sand transport capacity in the main channel, and an increment in shoaling.

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Upstream of the diversion he found a tendency for erosion and possible head-cutting while immediately downstream of the diversion there was a zone of deposition (Pereira, 2011).

A 1-D numerical model from Tarbert Landing to the Gulf of Mexico was calibrated, validated, and applied to predict the response of the Lower Mississippi River to different stimuli, such as proposed diversions, channel closures, channel modifications, and relative sea level rise. The model was developed by using HEC-RAS 4.0, a 1-D mobile-bed numerical model, which was calibrated based on a discharge hydrograph at Tarbert Landing and a stage hydrograph at the Gulf of Mexico to calculate the hydrodynamics of the river. The model showed that RSLR will decrease the capacity of the river to carry bed material (Davis, 2010).

Two one dimensional mobile bed numerical models were set for the Lower Mississippi River by Gurung (2012). A 1-D HEC-RAS model from Tarbert Landing to the Gulf of Mexico, based on the 1-D HEC-RAS model developed by Davis (2010); and a 1-D CHARIMA model from Belle Chasse to the Gulf of Mexico were developed in order to aid in the restoration and flood control effort. The models were calibrated and validated to predict the response of the river to channel modifications, varied flow and hurricane conditions. He observed flow distributions in the un-leveed channels, obtained prediction of shoaling or erosion in the main stem, and propagation of storm surges and reverse flows under hurricane conditions (Gurung T. , 2012)

Terán et al. (2013) developed two models to simulate the surge due to Hurricane Isaac for the Lower Mississippi River, a 1-D HECRAS model from Tarbert Landing to the Gulf of Mexico and a 2-D Delft3D model from Bonnet Carré to Head of Passes. The period evaluated represented a very unusual event since the river discharge was near a record low flow and the storm was moving extremely slow. The 1-D and 2-D models were evaluated for their ability to accurately predict a hurricane surge in the Mississippi River validating against observed data obtained from the U.S. Army Corps of Engineers (USACE:rivergages, 2012). Both models gave good representations of the surge movement by capturing the height and speed of travel of the storm surge (Teran, et al., 2013).

A 2D/3D hydrodynamic and sediment transport model was set up in Yangtze Estuary region in China. The simulations were run using Delft3D-FLOW. The model was found to be capable of reproducing the hydrodynamics and sediment transport processes. It was applied to the storm surge problem and the morphological evolution of Jiuduansha Shoals. It was found that better results were obtained if wind and wave effects are taken into account for storm surge simulations; and that the fractions of cohesive and non-cohesive sediment should be also included to reproduce morphological changes (Hu, Ding, Wang, & Yang, 2008).

A three dimensional model was developed for tidal estuary in the Pontchartrain Estuary to simulate long term salinity (Retana, 2008). For the development of the model, a multi-step approach was used involving a physical model of salinity exchange through a pass, a 3-D FVCOM model of the physical experiment, an FVCOM model of idealized Pontchartrain Basin and an FVCOM model for the entire estuary, including inputs from the Mississippi. The model reproduced seasonal salinity. It was also found that a variable friction coefficient distribution was needed to reproduce tides and salinity and that the model presented a high sensitivity to this parameter. It was also found that the salinity transport was improved by implementing a bi-directional open boundary condition in the vertical (Retana, 2008).

2.2 General Concepts

2.2.1 Computational Fluids Dynamics

The analysis of systems involving fluid flow, heat transfer and associated phenomena based on computer simulations is defined as Computational fluid dynamics (CFD). The introduction of more advanced high-performance computing hardware and user-friendly interfaces has promoted to the use of CFD in the solution of many problems including open channel flows (Versteeg & Malalasekera, 2007).

CFD codes are structured around the numerical features: pre-processor, solver and post-processor (Versteeg & Malalasekera, 2007). The pre-processing step treats the input of a flow problem, which involves different activities. One of these is the geometry definition of the region to be studied, known as the computational domain. Another important task is the grid generation, which is the subdivision of the domain into smaller, non-overlapping sub-domains called cells which constitute the mesh or grid.

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Moreover, the phenomena to be analyzed must be selected, the fluid properties must be defined and appropriate boundary conditions must be specified (Versteeg & Malalasekera, 2007).

The solution process can be done through three numerical solution techniques, which include: finite difference, finite elements and spectral methods. The finite volume method represents a special finite difference formulation which involves the integration of a control volume (employing the divergence theorem to convert some of the volume integrals to surface integrals) that distinguishes the finite volume method from all other CFD techniques, and its statements for each finite size cell makes all definitions easier to understand than the finite element and spectral methods (Versteeg & Malalasekera, 2007).

Finally the post-processing is the final last stage where results are visualized. There are versatile data visualization tools that allow domain geometry and grid display, plots of vector, lines and shaded contour, 2D and 3D surface, and also allow particle tracking, view manipulation (translation, rotation, scaling, etc.) and color PostScript or other graphics output (Versteeg & Malalasekera, 2007).

2.2.2 Sediment Transport of Non-cohesive Sediment

The transport of sediments by flow of water is the complete transport of solids that pass through a channel cross section. The sediment transport mechanisms can be explained by different kinds of motion (Graff, 1998).

There are three main ways in which non - cohesive sediment particles are transported, which are rolling, suspension and saltation. The rolling motion is given when the bed shear velocity is slightly greater than the critical bed shear velocity for movement initiation; suspension takes place when the bed shear velocity is higher than the critical value allowing the movement of the particle without being in contact with the bed; and saltation occurs when the bed shear velocity is high enough to allow the particle to travel for a distance without hitting the bed but not high enough to be suspended (Pereira, 2011). Figure 2 shows the motion modes affecting non-cohesive sediment.



Figure 2. Particle motion mechanisms (UDM, 2013)

According to the mechanism of transport the particles that constitute the total bed material load, q_s , can be divided into bed load, q_{sb} , the volumetric discharge per unit width of rolling particles; and suspended load, q_{ss} , the volumetric discharge per unit width of saltating particles. The total bed material load is defined as the summation of the bed load and suspended load as follows $q_s = q_{sb} + q_{ss}$ (Graff, 1998). Some researchers use volumetric loading units and others use mass loading units.

2.3 Delft3D General Overview

Delft3D is an integrated modeling framework with a multi-disciplinary approach that can carry out 2-D and 3-D computations for coastal, river, lake and estuarine areas. It can perform simulations of flows, sediment transports, waves, water quality, morphological developments and ecology. Delft3D is composed of several modules which are grouped on a mutual interface being capable to interact with one another. The hydrodynamic simulations are run with Delft3D -FLOW, a multi-dimensional program that performs unsteady flow and transport phenomena resulting from tidal and meteorological forcing on a rectilinear or curvilinear grid. The sigma co-ordinate is used to define the vertical distribution for the three dimensional simulations (Deltares, 2011).

2.4 Delft3D Formulation

2.4.1 Hydrodynamic equations

Delft3D-FLOW solves the Navier Stokes equations for incompressible flow. In 3D models the vertical velocities are computed from the continuity equation. The set of partial differential equations in combination with an appropriate set of initial and boundary conditions is solved on a structured grid (Deltares, 2011).

In the horizontal direction orthogonal curvilinear coordinates are used in the Cartesian system, (ξ, η) .

For the vertical direction the system is defined based on the boundary fitting coordinate system known as the sigma (σ) co-ordinate system which is defined by the following equation,

$$\sigma = \frac{z - \zeta}{d + \zeta} = \frac{z - \zeta}{H} \tag{1}$$

where z is the vertical co-ordinate in physical space; ζ is the free surface elevation above the reference plane (at z = 0); d is the depth below the reference plane and H is the total water depth given by $H = d + \zeta$ (Deltares, 2011).



Figure 3. Example of σ-model (Deltares, 2011)

The vertical σ system presents layers that are bounded by two sigma planes, and that follow the bottom topography and the free surface. The number of layers remain constant along the entire domain, however; the distribution of the relative layer thickness can be variable allowing to give more resolution to the area of interest such as the bed for sediment transport, among others. For this system we have that the bottom corresponds to $\sigma = -1$ and the free surface to $\sigma = 0$ as shown in Figure 2 (Deltares, 2011).

The continuity equation is given by:

$$\frac{\partial \zeta}{\partial t} + \frac{1}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}} \frac{\partial \left[(d+\zeta) U\sqrt{G_{\eta\eta}} \right]}{\partial \xi} + \frac{1}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}} \frac{\partial \left[(d+\zeta) V\sqrt{G_{\xi\xi}} \right]}{\partial \eta} = Q \quad (2)$$

where U is the depth-averaged velocity in ξ -direction, V is depth-averaged velocity in η -direction, and $\sqrt{G_{\eta\eta}}$; $\sqrt{G_{\xi\xi}}$ are coefficients used to transform curvilinear to rectangular coordinates. With Q representing the contributions per unit area due to the discharge or withdrawal of water, precipitation and evaporation:

$$Q = H \int_{-1}^{0} (q_{in} - q_{out}) \, d\sigma + P - E \tag{3}$$

The momentum equations in the horizontal for the ξ -direction and the η -direction are given respectively by:

$$\frac{\partial u}{\partial t} + \frac{u}{\sqrt{G_{\xi\xi}}} \frac{\partial u}{\partial \xi} + \frac{v}{\sqrt{G_{\eta\eta}}} \frac{\partial u}{\partial \eta} + \frac{\omega}{d+\zeta} \frac{\partial u}{\partial \sigma} - \frac{v^2}{\sqrt{G_{\xi\xi}}} \frac{\partial \sqrt{G_{\eta\eta}}}{\partial \xi} + \frac{uv}{\sqrt{G_{\xi\xi}}} \frac{\partial \sqrt{G_{\xi\xi}}}{\partial \eta} - fv = -\frac{1}{\rho_0 \sqrt{G_{\xi\xi}}} P_{\xi} + F_{\xi} + \frac{1}{(d+\zeta)^2} \frac{\partial}{\partial \sigma} \left(\nu_V \frac{\partial u}{\partial \sigma} \right) + M_{\xi}$$
(4)

and

$$\frac{\partial v}{\partial t} + \frac{u}{\sqrt{G_{\xi\xi}}} \frac{\partial v}{\partial \xi} + \frac{v}{\sqrt{G_{\eta\eta}}} \frac{\partial v}{\partial \eta} + \frac{\omega}{d+\zeta} \frac{\partial v}{\partial \sigma} + \frac{uv}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}} \frac{\partial \sqrt{G_{\eta\eta}}}{\partial \xi} + -\frac{u^2}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}} \frac{\partial \sqrt{G_{\xi\xi}}}{\partial \eta} + fu = -\frac{1}{\rho_0 \sqrt{G_{\eta\eta}}} P_\eta + F_\eta + \frac{1}{(d+\zeta)^2} \frac{\partial}{\partial \sigma} \left(\nu_V \frac{\partial v}{\partial \sigma} \right) + M_\eta.$$
(5)

where *u*, *v*, *w* are the flow velocities in ξ -direction, η - direction and σ –direction respectively. The three dimensional turbulence is represented by v_V , which is the vertical eddy viscosity defined as:

$$\nu_V = \nu_{mol} + \max(\nu_{3D}, \nu_V^{back}) \tag{6}$$

 v_V^{back} is the background vertical mixing coefficient; v_{mol} is the kinematic viscosity of water and v_{3D} is computed by a 3-D turbulent closure model.

Density variations are neglected, except for the pressure gradients, P_{ξ} and P_{η} and the horizontal Reynold's stresses are represented by the forces F_{ξ} and F_{η} .

The vertical velocity ω is computed from the continuity equation and it represents the vertical velocity relative to the moving σ -plane. It is define as follows:

$$\frac{\partial \zeta}{\partial t} + \frac{1}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}} \frac{\partial \left[(d+\zeta) \, u\sqrt{G_{\eta\eta}} \right]}{\partial \xi} + \frac{1}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}} \frac{\partial \left[(d+\zeta) \, v\sqrt{G_{\xi\xi}} \right]}{\partial \eta} + \frac{\partial \omega}{\partial \sigma} = H \left(q_{in} - q_{out} \right)$$

$$(7)$$

The physical vertical velocities w, which are required for the post-processing, can be expressed in the horizontal velocities, water depths and vertical ω -velocity according to:

$$w = \omega + \frac{1}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}} \left[u\sqrt{G_{\eta\eta}} \left(\sigma \frac{\partial H}{\partial \xi} + \frac{\partial \zeta}{\partial \xi} \right) + v\sqrt{G_{\xi\xi}} \left(\sigma \frac{\partial H}{\partial \eta} + \frac{\partial \zeta}{\partial \eta} \right) \right] + \left(\sigma \frac{\partial H}{\partial t} + \frac{\partial \zeta}{\partial t} \right)$$

$$\left(\mathbf{8} \right)$$

Under shallow water assumption, the vertical momentum equation is reduced to a hydrostatic pressure equation given by:

$$\frac{\partial P}{\partial \sigma} = -g\rho H. \tag{9}$$

where g is the acceleration due to gravity and ρ is density of water. After integration, the hydrostatic pressure is presented as:

$$P = P_{atm} + gH \int_{\sigma}^{0} \rho\left(\xi, \eta, \sigma', t\right) d\sigma'.$$
(10)

For water of constant density and taking into account the atmospheric pressure, the pressure gradient is defined as:

$$\frac{1}{\rho_0 \sqrt{G_{\xi\xi}}} P_{\xi} = \frac{g}{\sqrt{G_{\xi\xi}}} \frac{\partial \zeta}{\partial \xi} + \frac{1}{\rho_0 \sqrt{G_{\xi\xi}}} \frac{\partial P_{atm}}{\partial \xi},$$
(11)

$$\frac{1}{\rho_0 \sqrt{G_{\eta\eta}}} P_\eta = \frac{g}{\sqrt{G_{\eta\eta}}} \frac{\partial \zeta}{\partial \eta} + \frac{1}{\rho_0 \sqrt{G_{\eta\eta}}} \frac{\partial P_{atm}}{\partial \eta}$$
(12)

The gradients of the free surface level are the so-called barotropic pressure gradients. The atmospheric pressure is included in the system for storm surge simulations, since atmospheric pressure gradients are important in the external forcing at peak winds (Deltares, 2011).

If density is non-uniform, the pressure gradients related to temperature and salinity effect are defined as:

$$\frac{1}{\rho_0 \sqrt{G_{\xi\xi}}} P_{\xi} = \frac{g}{\sqrt{G_{\xi\xi}}} \frac{\partial \zeta}{\partial \xi} + g \frac{d+\zeta}{\rho_0 \sqrt{G_{\xi\xi}}} \int_{\sigma}^{0} \left(\frac{\partial \rho}{\partial \xi} + \frac{\partial \rho}{\partial \sigma} \frac{\partial \sigma}{\partial \xi}\right) d\sigma'$$
(13)

$$\frac{1}{\rho_0\sqrt{G_{\eta\eta}}}P_\eta = \frac{g}{\sqrt{G_{\eta\eta}}}\frac{\partial\zeta}{\partial\eta} + g\frac{d+\zeta}{\rho_0\sqrt{G_{\eta\eta}}}\int_{\sigma}^0 \left(\frac{\partial\rho}{\partial\eta} + \frac{\partial\rho}{\partial\sigma}\frac{\partial\sigma}{\partial\eta}\right)d\sigma'$$
(14)

The forces F_{ζ} and F_{η} in the horizontal momentum equations, which represent the unbalance of horizontal, are expressed as:

$$F_{\xi} = \frac{1}{\sqrt{G_{\xi\xi}}} \frac{\partial \tau_{\xi\xi}}{\partial \xi} + \frac{1}{\sqrt{G_{\eta\eta}}} \frac{\partial \tau_{\xi\eta}}{\partial \eta},$$
(15)

$$F_{\eta} = \frac{1}{\sqrt{G_{\xi\xi}}} \frac{\partial \tau_{\eta\xi}}{\partial \xi} + \frac{1}{\sqrt{G_{\eta\eta}}} \frac{\partial \tau_{\eta\eta}}{\partial \eta}.$$
 (16)

where τ is the shear stress. For the small-scale flow (partial slip along closed boundaries), for instance when shear stresses must be taken into account, expressing the shear stresses as:

$$\tau_{\xi\xi} = \frac{2\nu_H}{\sqrt{G_{\xi\xi}}} \left(\frac{\partial u}{\partial \xi} + \frac{\partial u}{\partial \sigma} \frac{\partial \sigma}{\partial \xi} \right)$$
(17)

$$\tau_{\xi\eta} = \tau_{\eta\xi} = \nu_H \left\{ \frac{1}{\sqrt{G_{\eta\eta}}} \left(\frac{\partial u}{\partial \eta} + \frac{\partial u}{\partial \sigma} \frac{\partial \sigma}{\partial \eta} \right) + \frac{1}{\sqrt{G_{\xi\xi}}} \left(\frac{\partial v}{\partial \xi} + \frac{\partial v}{\partial \sigma} \frac{\partial \sigma}{\partial \xi} \right) \right\}$$
(18)

$$\tau_{\eta\eta} = \frac{2\nu_H}{\sqrt{G_{\eta\eta}}} \left(\frac{\partial v}{\partial \eta} + \frac{\partial v}{\partial \sigma} \frac{\partial \sigma}{\partial \eta} \right)$$
(19)

For large-scale flow simulated with coarse horizontal grids, for example where shear stress along the closed boundaries may be neglected, the forces F_{ζ} and F_{η} are simplified as:

$$F_{\xi} = \nu_H \left(\frac{1}{\sqrt{G_{\xi\xi}} \sqrt{G_{\xi\xi}}} \frac{\partial^2 u}{\partial \xi^2} + \frac{1}{\sqrt{G_{\eta\eta}} \sqrt{G_{\eta\eta}}} \frac{\partial^2 u}{\partial \eta^2} \right),$$
(20)

$$F_{\eta} = \nu_H \left(\frac{1}{\sqrt{G_{\xi\xi}} \sqrt{G_{\xi\xi}}} \frac{\partial^2 v}{\partial \xi^2} + \frac{1}{\sqrt{G_{\eta\eta}} \sqrt{G_{\eta\eta}}} \frac{\partial^2 v}{\partial \eta^2} \right)$$
(21)

The discharge of water taking into account momentum adds a term in the *U* and *V* momentum equation:

$$M_{\xi} = q_{in}(\hat{U} - u) \tag{22}$$

$$M_{\eta} = q_{in}(V - v) \tag{23}$$

where M_{ξ} is the source or sink of momentum in ξ -direction; M_{η} is the source or sink of momentum in η -direction; \hat{U} is the velocity of water discharged in ξ -direction and \hat{V} is the velocity of water discharged in η -direction.

2.4.2 Transport Equations

The transport of suspended solids, dissolved substances, salinity and heat is often required in modeling water bodies. The transport is simulated under an advection-diffusion formulation in three dimensions. In order to represent discharges and withdrawals, the source and sink terms are included. The transport equation is defined as:

$$\frac{\partial (d+\zeta) c}{\partial t} + \frac{1}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}} \left\{ \frac{\partial \left[\sqrt{G_{\eta\eta}} (d+\zeta) uc\right]}{\partial \xi} + \frac{\partial \left[\sqrt{G_{\xi\xi}} (d+\zeta) vc\right]}{\partial \eta} \right\} + \frac{\partial \omega c}{\partial \sigma} = \frac{d+\zeta}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}} \left\{ \frac{\partial}{\partial \xi} \left(D_H \frac{\sqrt{G_{\eta\eta}}}{\sqrt{G_{\xi\xi}}} \frac{\partial c}{\partial \xi} \right) + \frac{\partial}{\partial \eta} \left(D_H \frac{\sqrt{G_{\xi\xi}}}{\sqrt{G_{\eta\eta}}} \frac{\partial c}{\partial \eta} \right) \right\} + \frac{1}{d+\zeta} \frac{\partial}{\partial \sigma} \left(D_V \frac{\partial c}{\partial \sigma} \right) - \lambda_d (d+\zeta) c + S,$$
(24)

where D_H is the horizontal diffusion coefficient; D_V is the vertical diffusion coefficient; λ_d represents the 1st order decay process and *S* is the source and sink terms per unit area due to discharge (q_{in}) or withdrawal (q_{out}) of water (Deltares, 2011).

2.4.3 Boundary Conditions

A group of initial and boundary conditions for water levels and horizontal velocities must be specified to get a solution for the 3D and 2D depth-averaged shallow water equations applied in Delft3D-FLOW. The contour of the model domain consists of closed boundaries which are parts along "land-water" lines (river banks, coastlines) and open boundaries which are parts across the flow field. Closed boundaries are natural boundaries, while open boundaries are always artificial "water-water" boundaries. To limit the computational area and computational effort in a numerical model it is necessary to introduce open boundaries.

For Delft3D-FLOW the flow at the open boundaries is sub-critical, which means that the velocity of wave propagation is bigger than the magnitude of the flow. For subcritical flow there are two situations, inflow and outflow. At inflow, where the velocity component along the open boundary is set to zero, it is necessary to specify two boundary conditions while at outflow it is required to specify one boundary condition. The first boundary condition is external forced by the water level, the normal velocity, the discharge rate or the Riemann invariant. The second boundary condition is a built-in boundary condition.

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The reach of the built-in boundary condition is frequently restricted to only a few grid cells near to open boundary, in that case is recommended to specify the tangential velocity component, but for Delft3D-FLOW it is not possible yet to specify it at the input, therefore, it would be suitable to define the model boundaries at locations where the grid lines of the boundary are perpendicular to the flow with the purpose of obtaining a realistic flow pattern near the open boundary (Deltares, 2011). This should be accomplished in designing the grid, since all of the mesh should be orthogonal.

2.4.4 Turbulence

The turbulent scales of motion are solved as a "sub-grid" process since the vertical and horizontal grid is usually too coarse. The primitive variables are space and time averaged quantities. Filtering the equations leads to the need for appropriate closure assumptions.

The horizontal eddy viscosity coefficient v_H and the eddy diffusivity coefficient D_H are much larger than the vertical coefficients v_V and D_V . The horizontal coefficients are assumed to be a superposition of molecular viscosity, 2D-turbulence and 3D-turbulence.

The three-dimensional turbulence is computed following one of the turbulence closure models. The k-L turbulence closure model is used for the 3-D simulations performed in this research.

2.4.4.1 k-L Turbulence Model

The κ -*L* model is a first order turbulence closure scheme implemented in Delft3D-FLOW in which the mixing length *L* is given by the following equation:

$$L = \kappa \left(z+d\right) \sqrt{1 - \frac{z+d}{H}} F_L\left(Ri\right)$$
(25)

where κ is the turbulent kinetic energy; *d* is the depth below some horizontal plane of reference; *H* is the total water depth; *F*_L (*Ri*) is the damping function.

The velocity scale is supported on the kinetic energy of turbulent motion. The turbulent kinetic energy κ follows from a transport equation that contains an energy dissipation term D_k , a buoyancy term B_k and a production term P_k , assuming that these terms are the dominating terms and that the horizontal length scales are much larger than the verticals ones.

The transport equation is employed in a non-conservative form. The second assumption leads to simplification of the production term. The transport equation for κ is as follows:

$$\frac{\partial k}{\partial t} + \frac{u}{\sqrt{G_{\xi\xi}}} \frac{\partial k}{\partial \xi} + \frac{v}{\sqrt{G_{\eta\eta}}} \frac{\partial k}{\partial \eta} + \frac{\omega}{d+\zeta} \frac{\partial k}{\partial \sigma} = \\ + \frac{1}{(d+\zeta)^2} \frac{\partial}{\partial \sigma} \left(D_k \frac{\partial k}{\partial \sigma} \right) + P_k + P_{kw} + B_k - \varepsilon.$$
(26)

where D_k is an energy dissipation term, P_k is a production term, P_{kw} is a production term due to wave action; B_k is a buoyancy flux term and ε is the dissipation in transport equation for turbulent kinetic energy .

With,

$$D_k = \frac{\nu_{mol}}{\sigma_{mol}} + \frac{\nu_{3D}}{\sigma_k} \tag{27}$$

The horizontal gradients of the horizontal velocity and all the gradients of the vertical velocities are neglected in the production term P_k of turbulent kinetic energy, and then this term is given by:

$$P_{k} = \nu_{3D} \frac{1}{(d+\zeta)^{2}} \left[\left(\frac{\partial u}{\partial \sigma} \right)^{2} + \left(\frac{\partial v}{\partial \sigma} \right)^{2} \right]$$
(28)

A more extended production term P_{κ} of turbulent kinetic energy (option "partial slip", rough side wall) can be used for small-scale applications, given by:

$$P_{k} = 2\nu_{3D} \left[\frac{1}{2\left(d+\zeta\right)^{2}} \left\{ \left(\frac{\partial u}{\partial\sigma}\right)^{2} + \left(\frac{\partial v}{\partial\sigma}\right)^{2} \right\} + \left(\frac{1}{\sqrt{G_{\xi\xi}}} \frac{\partial u}{\partial\xi}\right)^{2} \right] + 2\nu_{3D} \left[\frac{1}{2} \left(\frac{1}{\sqrt{G_{\eta\eta}}} \frac{\partial u}{\partial\eta} + \frac{1}{\sqrt{G_{\xi\xi}}} \frac{\partial v}{\partial\xi}\right)^{2} + \left(\frac{1}{\sqrt{G_{\eta\eta}}} \frac{\partial v}{\partial\eta}\right)^{2} \right]$$
(29)

In this equation, v_{3D} is the vertical eddy viscosity, expressed by:

$$\nu_{3D} = c'_{\mu} L \sqrt{k} \tag{30}$$

where c'_{μ} is a constant determined by calibration, derived from the empirical constant c_{μ} in the κ - ξ model.

In the two previously $P\kappa$ equations expressed, it has been assumed that the gradients of the vertical velocity *w* can be neglected with respect to the gradients of the horizontal velocity components *u* and *v*. In the same way, has been neglected the horizontal and vertical (σ -grid) curvature of the grid.

The turbulent energy production due to wave action is given by P_{kw} , as is described in the following equation:

$$P_{kw}\left(z'\right) = \frac{4D_w}{\rho_w H_{rms}} \left(1 - \frac{2z'}{H_{rms}}\right) \quad \text{for} \quad 0 \le z' \le \frac{1}{2} H_{rms}$$
(31)

where z' is the vertical co-ordinate; D_w is the total depth-averaged due to wave breaking; ρ_w is the density of the water; $H_{rms}/2$ is the root-mean-square wave height.

Turbulent kinetic energy is converted into potential energy in stratified flows. This is represented by a buoyancy flux B_k expressed by:

$$B_k = \frac{\nu_{3D}}{\rho \sigma_\rho} \frac{g}{H} \frac{\partial \rho}{\partial \sigma}$$
(32)

using the Prandtl-Schmidt number $\sigma_{\rho} = 0.7$ for salinity and temperature and $\sigma_{\rho} = 1.0$ for suspended sediments.

For the κ -*L* model, it is assumed that the dissipation ε depends on the mixing length *L* and kinetic turbulent energy κ , according to:

$$\varepsilon = c_D \frac{k\sqrt{k}}{L} \tag{33}$$

where c_D , is a constant determined by calibration, derived from the constant c_{μ} :

$$c_D = c_\mu^{3/4} \approx 0.1925$$
 (34)

It is necessary to specify boundary conditions to obtain a solution from the transport equation. It is assumed a local equilibrium of production and dissipation of kinetic energy at the bed which leads to the following Dirichlet boundary condition:

$$k|_{\sigma=-1} = \frac{u_{*b}^2}{\sqrt{c_{\mu}}}$$
(35)

To determine the friction velocity u_{*b} at the bed from the magnitude of the velocity in the grid point nearest to the bed, it is assumed a logarithmic velocity profile, using the following expression:

$$\vec{u}_b = \frac{\vec{u}_*}{\kappa} \ln\left(1 + \frac{\Delta z_b}{2z_0}\right) \tag{36}$$

where \vec{u}_* is the vertically averaged friction velocity; Δz_b is the distance to the computational grid point closest to the bed ; z_0 is the bed roughness length. The bed roughness (roughness length) might be improved by the presence of wind generated short crested waves.

A similar Dirichlet boundary condition is prescribed, in case of wind forcing for the turbulent kinetic energy κ at the free surface:

$$k|_{\sigma=0} = \frac{u_{*s}^2}{\sqrt{c_{\mu}}}$$
(37)

where u_{*s} is the friction velocity at the free surface. The turbulent kinetic energy k at the surface is set to zero, in the absence of wind.

At open boundaries, the next equation is used to calculate the turbulent energy k without horizontal advection:

$$k = \frac{1}{\sqrt{c_{\mu}}} \left[\left(u_*^b \right)^2 \left(1 - \frac{z+d}{H} \right) + u_{*s}^2 \frac{z+d}{H} \right]$$
(38)

For a logarithmic velocity profile this will approximately lead to the next linear distribution based on the shear-stress at the bed and at the free surface:

$$k(z) = \frac{1}{\sqrt{c_{\mu}}} \left[u_{*b}^2 \left(1 - \frac{z+d}{H} \right) + u_{*s}^2 \frac{z+d}{H} \right]$$
(39)

where u_{*b} is the modified friction velocity near bed; u_{*s} is the friction velocity at the free surface.

2.4.5 Van Rijn (1984)

For the transport of fine sediments without waves, Van Rijn (Rijn, 1984a; 1984b; 1984c) proposes the following relations. The following expression gives the bead – load transport rate:

$$S_b = \begin{cases} 0.053 \sqrt{\Delta g D_{50}^3} D_*^{-0.3} T^{2.1} & \text{for } T < 3.0 \\ 0.1 \sqrt{\Delta g D_{50}^3} D_*^{-0.3} T^{1.5} & \text{for } T \ge 3.0 \end{cases}$$
(40) (41)

where
$$D_*$$
 is a non-dimensional particle diameter; D_{50} is the median diameter of

sediment; and T a dimensionless bead shear parameter is calculated with the following expression:

$$T = \frac{\mu_c \tau_{bc} - \tau_{bcr}}{\tau_{bcr}} \tag{42}$$

According to Shields, τ_{bc} this is normalized with the critical bed shear stress using the following expressions:

$$\tau_{bc} = \frac{1}{8} \rho_w f_{cb} q^2 \tag{43}$$

$$f_{cb} = \frac{0.24}{\left(\frac{10\log\left(12h/\xi_c\right)\right)^2}{\left(\frac{10\log\left(12h/\xi_c\right)\right)^2}}$$
(44)

$$\mu_c = \left(\frac{18^{10} \log\left(12h/\xi_c\right)}{C_{g,90}}\right)^2 \tag{45}$$

where $C_{g,90}$ can be defined as the Chézy coefficient, related to the grain and defined by this expression:

$$C_{g,90} = 18^{-10} \log\left(\frac{12h}{3D_{90}}\right)$$
(46)

According to Shields, the critical shear is written:

$$\tau_{bcr} = \rho_w \Delta g D_{50} \theta_{cr} \tag{47}$$

From this, θ_{cr} defined as the Shields parameter and a function of the dimensionless particle parameter, D_* obtained with the following expression:

$$D_* = D_{50} \left(\frac{\Delta g}{\nu^2}\right)^{\frac{1}{3}} \tag{48}$$

On the other hand, the expression for the suspended transport is:

$$S_s = f_{cs}qhC_a \tag{49}$$

where, C_a is a reference concentration, given by:

$$C_a = 0.015\alpha_1 \frac{D_{50}}{\xi_c} \frac{T^{1.5}}{D_*^{0.3}}$$
(50)

q is the depth averaged velocity; h is the water depth; f_{cs} is the shape factor with only and approximate solution:

$$f_{cs} = \begin{cases} f_0(z_c) & \text{if } z_c \neq 1.2\\ f_1(z_c) & \text{if } z_c = 1.2 \end{cases}$$

$$f_0(z_c) = \frac{(\xi_c/h)^{z_c} - (\xi_c/h)^{1.2}}{(1 - \xi_c/h)^{z_c} (1.2 - z_c)}$$
(51)

$$f_1(z_c) = \left(\frac{\xi_c/h}{1 - \xi_c/h}\right)^{1.2} \ln(\xi_c/h)$$
(52)

where, ξ_c is the reference level or roughness height (can be interpreted as the bed-load layer thickness); z_c is the suspension number given by:

$$z_c = \min\left(20, \frac{w_s}{\beta\kappa u_*} + \phi\right) \tag{53}$$

$$u_* = q\sqrt{\frac{f_{cb}}{8}} \tag{54}$$

$$\beta = \min\left(1.5, 1+2\left(\frac{w_s}{u_*}\right)^2\right) \tag{55}$$

$$\phi = 2.5 \left(\frac{w_s}{u_*}\right)^{0.8} \left(\frac{C_a}{0.65}\right)^{0.4}$$
(56)

"The bed-load transport rate is imposed as bed-load transport due to currents, S_{bc} , while the computed suspended load transport rate is converted into a reference concentration equal to $f_{cs}C_a$ " (Deltares, 2011).
2.4.6 Non-cohesive sediment dispersion

The vertical sediment mixing coefficient can be calculated using the algebraic or *k*-L turbulence model, being computed from the vertical fluid mixing coefficient. When it is non-cohesive sediment, the Van Rijn's "beta factor" multiplies the fluid mixing coefficient. The beta factor describes the different diffusivity of a fluid "particle" and a sand grain, and the mathematical representation is:

$$\varepsilon_s^{(\ell)} = \beta \varepsilon_f^{(\ell)} \tag{57}$$

where $\varepsilon_s^{(\ell)}$ is the vertical sediment mixing coefficient for sediment fraction; β is the Van Rijn's "beta" factor for the sediment fraction; $\varepsilon_f^{(\ell)}$ is the vertical fluid mixing coefficient calculated by the selected turbulence closure model (Deltares, 2011).

2.5 Lacey Regime Equations

The Lacey Regime Equations are used for the design of channels, stating a set of stable channel dimensions for each given flow and silt load (Davis, 2010). The depth and width of the channel are given based on the wetted perimeter and hydraulic radius.

The width is represented by the expression below,

$$P = 2.67 Q^{1/2} \tag{58}$$

The depth of the channel is represented by R which is given by,

$$R = \left[\frac{Q}{1.17Pf_{s}^{1/2}}\right]^{2/3}$$
(59)

$$f_s = 8\sqrt{D_{50}}$$
 (60)

To estimate the velocity in the channel the next expression is used,

$$v = 1.17\sqrt{f_s R} \tag{61}$$

The cross-sectional area of the channel can be found by the continuity equation,

$$A = Q/v = PR \tag{62}$$

where *P* is the wetted perimeter which represents the width, ft; *Q* is the flow rate, ft³/s; f_s is the silt factor used to incorporate sediment effect; D_{50} is the median grain size, in; *v* is the velocity, ft/s; *R* is the hydraulic radius which represents the depth, ft; *A* is the cross-sectional area, ft².

To obtain the dimensions of the equivalent channels, the cumulative width and depth of the cuts or bifurcated channels are used.

2.6 Statistical Analysis

The root mean square error (RMSE), the coefficient of determination of r and the bias between the observations and simulated results were obtained using the following equations:

Root Mean Square Error (RMSE) =
$$\sqrt{\frac{\sum_{i=1}^{N} (O_i - P_i)^2}{N}}$$
 (63)

Bias Error =
$$-\frac{\sum_{i=1}^{N} (O_i - P_i)}{N}$$
 (64)

$$r^{2} = \left(\frac{\sum_{i=1}^{n} (O_{i} - \bar{O}) (P_{i} - \bar{P})}{\sqrt{\sum_{i=1}^{n} (O_{i} - \bar{O})^{2}} \sqrt{\sum_{i=1}^{n} (P_{i} - \bar{P})^{2}}}\right)^{2}$$
(65)

where O_i is the observed value; P_i is the modeled value; \overline{O} is the average of the observed value, \overline{P} is the average of the modeled value; N is the number of observations (Krause, Boyle, & Base, 2005; Pereira, 2011).

3. METHODOLOGY

3.1 Model Selection

Different three dimensional models had to be evaluated based on their capabilities to decide what the best option was for the hydrodynamics and sediment transport simulations on the modeling domain. After a preselecting process, two models were considered for this application, mainly based on their availability, ECOMSED and Delft3D.

3.1.1 ECOMSED

ECOMSED is a sigma coordinate, free surface model, designed to realistically simulate time-dependent distribution of waters levels, currents, temperature, salinity, tracers, cohesive and non-cohesive sediments and waves in marine and freshwater systems. It is based on the Princeton Ocean Model developed by Alan Blumberg and George Mellor (1987) with modifications for its applicability in estuaries and coastal oceans and subsequent additions from many other contributors. The major assumption in this code and most others is that the vertical pressure distribution is hydrostatic (McCorquodale & Georgiou, 2006)

3.1.2 Delft3D

Delft3D offers the Delft3D-FLOW module, which is a multidimensional (2D and 3D) hydrodynamic and transport simulation model which calculates non-steady flow and transport phenomena resulting from tidal and meteorological forcing on a curvilinear, boundary fitted domain. In 3D simulations, the vertical grid is defined following the sigma transformation. This results in a high computing efficiency because of the constant number of vertical layers over the whole computational domain (McCorquodale & Georgiou, 2006).

3.1.3 Selection Criteria

The model selection criteria are always subjected to the problem that is to be solved but some common aspects to evaluate are: the availability of the model; what processes can be simulated; cost of obtaining and implementing the code; assumption and limitations; ease of utilization; quality of documentation and user manual; hardware and software requirements; grid system; formulation; graphic user interface; order of accuracy; among other important parameters for the domain to be studied (McCorquodale & Georgiou, 2006).

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Based on the most important characteristics to simulate the hydrodynamics and noncohesive sediment transport on the Lower Mississippi River, Table 1 is constructed to visualize in a more practical way which model is more suitable for this purpose.

Table 1. Mouel Characteristics for Selection						
Characteristic	ECOMSED	Delft3D				
Public Domain	Yes	Yes				
Distributor	HydroQual	Deltares				
Formulations	Finite Volume Method	Finite Volume Method				
Grid	Structured	Structured				
Sediment Module	Yes	Yes				
Wetting/Drying	No	Yes				
Pre-processing Tool	No	Yes				
Post Processing Tool	No	Yes				
Cost	Free	Free				

Table 1. Model Characteristics for Selection

After analyzing different aspects of models available, Delft3D turned out to be a more convenient and powerful option to perform the processing, noting that the graphical user interface provides pre-processing and post-processing tools that ease the modeling process. The modeled physics are similar in both models.

Delft3D was developed to solve the shallow wave equations in 2-D and 3-D. It has been successfully applied to coastal areas, estuaries and rivers. It is a public domain model with a large user group. It was selected for the Mississippi River because it includes: riverine and estuarine hydrodynamics, sediment transport and channel morphology. It has an excellent graphics interface (Teran, et al., 2013).

3.1.4 Delft3D Capabilities

The Delft3D main capabilities that are suited for this project research involve:

- 3-D hydrostatic numerical model
- Orthogonal curvilinear grid
- Based on a sigma (σ) level coordinates for the vertical distribution
- Perform non-cohesive sediment transport
- Provide morphology updating options during simulations
- Possibility of performing parallel computations

3.2 General Modeling Setup

A 3-D Delft3D numerical model was developed for hydrodynamics and non-cohesive sediment transport simulations on the Lower Mississippi River.

3.2.1 General Considerations

The study area extends from Bonnet Carré (RM 127) to the Head of Passes (RM 0). The model was applied to simulate the hydrodynamics and non-cohesive sediment transport of the modeling domain for high flow periods. After experimentation with a range of time steps, the simulations were performed using a time step of 0.5 minutes. The sigma levels were variable according to a parabolic distribution with the smallest layers near the bed. A variable roughness was used over the domain. Van Rijn's 1984 sediment formulations were used for the sediment transport simulations. The basic model developed at the beginning of the research project was applied to periods under hurricane storm surge conditions.

3.2.2 Modeling Domain

The modeled reach in this study extends from Bonnet Carré (RM 127) to the Head of Passes (RM 0), which is shown in Figure 3. Along the reach there are some continuous outflows such as West Bay and Main Pass, and the east bank of the river downstream of Bohemia (RM 47) has a natural levee that overtops in periods of high flow. Some other outlets are: Grand Pass and Tiger Pass, Baptiste Collette, Fort St Philip, Caernarvon and Davis Pond and are also represented in the grid corresponding to this domain.



Figure 3. Modeling Domain (USGS, 2007)

3.2.3 Model Development

To start up the model, a 2-D depth-averaged model in the study reach was set up based on a 2-D hydrodynamic model built by Dr. Pereira (Pereira, 2-D Regional Delft3D model for the Mississippi River Hydro-study, 2012) based on discharge boundary conditions for all outlets for the same reach. Figure 4 shows the original grid indicating all outlets boundary conditions being discharge type. First, a uniform bathymetry and uniform roughness for the main channel and outlets were used. For this stage, the modeled was to be transformed from a discharge based boundary condition for the outlets to a stage based model for the most important outlets in the domain. This process had to be done step by step, since those changes lead to instabilities.



Figure 4. Initial Grid map Indicating Boundary Conditions Type

After the main outlets (West Bay, Main Pass, Bohemia Spillway, Grand Pass and Tiger Pass, Baptiste Collette and Fort St Philip) were set to stage boundary conditions, the model was converted into a three dimensional model, with 10 layers under a parabolic scheme, going from thinner layer at the bottom to thicker at the surface. Along with the conversion to a 3-D model, the bathymetry and roughness Manning's n were converted to a variable distribution to obtain a more realistic setting of the model. Figure 5 shows the transitional grid between the original model grid and the final grid with the extended grid.



Figure 5. Transitional Grid map Indicating Boundary Conditions Types

Moreover, some overflow zones were extended to account for channels present between the Bohemia and Fort St Philip area. Figure 6 shows the last grid for the extended and refined grid.



Figure 6. Final Grid map Indicating Boundary Conditions Types

3.2.3.1 Grid Generation

The generation of the grid by using the Delft3D was a simple but time consuming process. The tool used for this purposed was the RGFGRID. The grid was built on a map sample that was imported into the grid generation tool. Figure 7 shows the samples being imported to the grid generation tool.



Figure 7. Sample importing

The next step consisted in creating splines following the shape of the river section. Figure 8 displays the spline creation on a section of the main stem of the river.



Figure 8. Spline creation

After the splines are defined, they must be converted into grid. Figure 9 shows the splines being converted to grid.



Figure 9. Grid creation

Then splines must be deleted, which is shown on Figure 10.



Figure 10. Splines deletion

Depending on the way the splines are drawn, it might be necessary to refine or derefine the grid generated. Figure 11 shows both options on the main menu.



Figure 11. Refinement and Derefinement

Once the grid has been developed, it is necessary to orthogonalise it, this process is shown in Figure 12.



Figure 12. Grid Ortogonalisation

After the grid is created, the file must be exported as a .grd file.

3.2.3.2 Bathymetry Interpolation

The bathymetry interpolation was done by using the QUICKIN tool. The generated grid file (.grd) and depth samples (.xyz file) must be imported into the interface. Figure 13 shows the grid being imported into the program, and Figure 14 presents the bathymetry and the grid being superposed.



Figure 13. Grid Importing into QUICKIN

DD	elft3D-QUICKIN - [unnam	ed]	
File	Edit Operations View C	o-ordinate System Settings Help	
D	New Project		
Ê	Open Project		
	Save Project		
	Save Project As		
	Attribute Files	🕨 😅 Open Land Boundary	
	Import	Save Land Boundary	
	Export	Open Polygons	
	Delft3D-FLOW Attributes	Save Polygons	
È	Open Colour Map	😅 Open Samples	
È	Open Settings	😅 Open TEKAL Samples	
	Save Settings	Save Samples	$\langle \chi \gamma$
	Exit	൙ Open Text File	
	- Photostallar		

Figure 14. Depth and Grid Superposition

Once both files have been imported into the tool, it is necessary to interpolate the depths into the grid as shown in Figure 15.



Figure 15. Bathymetry Interpolation

Finally, the depths must be exported as a .dep file to create the bathymetry file, as displayed in Figure 16.



Figure 16. Depth file Generation

3.2.3.3 Roughness File Generation

The roughness file for a spaced varied distribution is generated using the QUICKIN tool. As done for the bathymetry interpolation, the grid must be imported to the tool. Once the grid has been displayed, a polygon around the desired area must be drawn to define the value needed. Figure 17 shows a polygon around an area of the grid.



Figure 17. Polygon definition

Once the polygon was drawn, the value for the roughness must be defined. Figure 18 shows the option to insert the roughness value, which is created like a depth file.

dit Op	erations View Co-ordinate System Setting	is Help
•	Change Centre of Projection	
	Delete •	
	Create Rectangular Grid Data in Cell Centre	D Give a value
	Grid Cell Averaging Triangular Interpolation	Uniform depth = 0.02
Z	Internal Diffusion Smoothing	OK Cancel
	Courant Numbers	
H h	Actual and Maximum Data Dimensions Check Interpolation Volumes and Areas	
	Combine Depth and Uniform Value	Fill Missing Depth with Uniform Value
	Combine Depth and Second Depth Combine Samples and Uniform Value Copy Depth to Samples	Change Depth to Max of Depth and Uniform Value Change Depth to Min of Depth and Uniform Value Add Uniform Value to Depth
	Copy Polyline to Samples Copy Polyline to Samples	Multiply Depth with Uniform Value
 0.0	Undo Depth Undo Samples	

Figure 18. Roughness Definition

Once the .dep file has been exported, an .rgh file must be created containing the values assigned as depth on this last .dep file. It is important to highlight that the roughness file needs to be filled with a component on the u direction, and one for the v direction. As recommended by Deltares, both components were defined with the same value, meaning the values defined as depth for the roughness are pasted twice on the roughness file.

3.2.3.4 Boundary Condition Definition

The boundary conditions are defined on the Flow input tool. Once the grid has been generated, the .mdf file (main input file) can be created to define all the variables needed, including the boundary conditions. The grid file must be imported into this input file. Figure 19 shows the main options on the menu of the .mdf generation file, where the Boundaries button can be observed. Under this option, using the visualization area, the boundaries are defined. Figure 20 shows the created boundary.

Delft3D-FLOW *		Delft3D-FLOW *
le Table View Help		File Table View Help
Description	Boundaries	De Attribute-files ries
Domain	*	Domain
Time frame	Add Open	Visualisation Area
Processes	Carting same	× 8582 ✓ Add [8.3 [m]
Initial conditions	Secon name	< 2.0 Modify
Boundaries	" MI NI	< 3.5 View < 5.0 < 5.6
Physical parameters	M2 N2	< 8.1 < 9.7
Numerical parameters	Flow conditions	(12.7)
Operations	Reflection parameter alpha:	<pre> 13.0</pre>
Monitoring	Forcing type:	
Additional parameters		
Output	Edit flow conditions	

Figure 19. Boundary Creation



Figure 20. Boundary Definition

The flow input tool only allows the specification of two values for each boundary, one initial and one final value. However, the data were introduced externally on the ASCII file generated (.bct file).

3.3 Hydrodynamics and Sediment Transport Set up

3.3.1 Grid Resolution

The grid has a varied resolution. The total grid consists of 2004 points in the M direction and 117 points in the N direction. The main channel consists of about 20 cells across. The typical grid dimension is 50m. Figure 21 shows a portion of the grid where the number of cells across the channel can be observed.



Figure 21. Final Grid View

3.3.2 Bathymetry

Setting up the bathymetry for this model was a very wearying task. The construction of the bathymetry file consisted on compiling data from different sources, such as LIDAR 2003 bathymetry data, Corps of Engineers Multibeam, Lake Pontchartrain Basin Foundation (LPBF) surveyed height of land data and Google Earth data.

The surveyed depths were used to build the bathymetry where available; otherwise, the depths were estimated by using Lacey Regime equations based on the measured widths of the channels (Google Earth). For the overflow areas, such as Fort St Philip and Ostrica, some equivalent channels were used to replace the cuts present in those areas (based on $\Sigma AR^{2/3}$) that have the capability to extract flow from the main channel. Figure 22 displays the dimensions for the equivalent channels, and Figure 23 shows the bathymetry and the grid around that area.

Also, the Gras Pass, which is a channel that formed in the Bohemia area during the spring flood of 2011, was added to the bathymetry by using the same approach.



Figure 22. Cuts at Fort St Philip (Google Earth)



Figure 23. Equivalent Channels at Fort St Philip

The high land that works as a natural levee in the Bohemia area was treated as a broad-crested weir, which was built based on data provided by LPBF.

The bathymetry distribution for the modeling domain is shown in Figure 24.



Figure 24. Bathymetry distribution along the domain

3.3.3 Layer Distribution

The vertical sigma coordinates consist on 10 layers being 11 sigma levels along the entire domain. They are distributed under a parabolic profile, going from the thinner layer at the bottom to the thicker one at the surface. Figure 25 shows the vertical profile for the layer distribution.



Figure 25. Parabolic Profile for Vertical Layer Distribution

3.3.4 Roughness

The roughness was defined under Manning's n formulation. It was a varied roughness along the entire domain. The Manning's n was the main parameter to calibrate the hydrodynamics in the model. Tables 2 and 3 present the Manning's n used in the different areas of the modeling domain.

Table 2. Manning's n roughness for main channel				
Area – Main Channel	Manning's n			
Bonnet Carré (RM 127) to New Orleans (RM 103)	0.02600			
New Orleans (RM 103) to IHNC Lock (RM 93)	0.02700			
IHNC Lock (RM 93) to West Point a la Hache (RM 49)	0.01680			
West Point a la Hache (RM 49) to Bohemia area (RM 44)	0.01450			
Bohemia area (RM 44) to Venice (RM 11)	0.01485			
Venice (RM 11) to Head of Passes (RM 0)	0.01750			

Table 3. Manning's n roughness for outlets

Area – Outlets	Manning's n
Bohemia Spillway	0.06
Bohemia 2	0.06
Ostrica 1	0.08
Ostrica 2	0.07
Equivalent Channels @ Ostrica	0.05
Fort St Philip	0.10
Fort St Philip 2	0.10
Equivalent Channels	0.03
Baptiste Collette	0.03
Grand + Tiger Pass	0.04
West Bay	0.06
Main Pass	0.03
Gras Pass	0.05

Figure 26 displays the Manning's n roughness distribution map in the modeling domain.



Figure 26. Manning's n Roughness distribution along the domain

3.3.5 Boundary Conditions

The upstream boundary condition at Bonnet Carré (RM 127) and Caernarvon and Davis Pond diversions corresponds to daily discharge flows obtained from a calibrated 1-D HEC-RAS model from Tarbert Landing to Gulf of Mexico (Gurung T. , 2012). The downstream end, Head of Passes (RM 0), and outlets (Main Pass, West Bay, Baptiste Collette, Grand Pass + Tiger Pass, Fort St, Philip and Bohemia Spillway) boundary conditions consist on daily stage values obtained from USACE data (USACE:rivergages, 2012) and NOAA data (NOAA:Tides&Currents, 2012). The outlets data corresponds to data at the Gulf of Mexico for the corresponding periods. Figure 27 shows the modeling domain and boundaries along the domain.



Figure 27. Boundaries along the modeling domain

3.3.6 Sediment Transport Main Settings

The sediment transport formulations correspond to the Van Rijn (1984) equations. Three sediment classes are included based on the particle size. Table 4 shows the grain sizes and settling velocities corresponding to the different classes of sand particles simulated in the model.

The sediment size distribution was completed based on a USACE report (Nordin & Queen, 1992).

	Tuble 4. Seument Classes, Furthere Size and Fun Verbetiles				
Sediment	Sediment Size, D ₅₀	Settling Velocity			
Class	(mm)	(m /s)			
Very Fine Sand	0.08833	0.0053			
Fine Sand	0.16667	0.0180			
Medium Sand	0.33333	0.0430			

 Table 4. Sediment Classes, Particle Size and Fall Velocities

3.3.7 Other important parameters and settings

For the hydrodynamics and non-cohesive sediment transport simulations some parameters can be summarized as follows:

- Initial water level: 1.5m
- Initial Sediment Concentration (all classes): 0.0Kg/m³
- Specific sediment density: 2650Kg/m³
- Horizontal Eddy Viscosity: 1.0m²/s
- Vertical Eddy Viscosity: 0.1m²/s
- Horizontal Eddy Diffusivity: 10.0m²/s
- Turbulence 3-D Model: k-L
- Update bathymetry during FLOW simulation: Enabled
- Equilibrium sand concentration profile at inflows boundaries: Enabled
- Reference height (.tra files): 2.5m
- Alpha Coefficient (.tra files): 1.0

It is important to highlight that both the Alpha Coefficient and the Reference height were the main parameters to obtain the sediment transport calibration. In this project many different setting combinations were tested; however, for simplification purposes, only calibrated results will be presented.

3.4 Hurricanes Application Setup

The model used for the Hurricane application consisted on the two dimensional model based on the original grid and stage boundary conditions. Two hurricane periods were simulated to obtain an estimate of the behavior of the model for storm surge propagation analysis: Hurricane Isaac and Hurricane Gustav.

3.4.1 Grid Resolution

The grid presented a varied resolution. The first grid developed in general terms is a 100mx100m curvilinear grid. The main channel consists of 9 cells across with widths of about 1000m in average. Figure 28 shows a section of the grid where the number of cells across the channel can be observed for the original grid.



Figure 28. Original Coarser Grid View

3.4.2 Boundary Conditions

The upstream boundary condition corresponds to the hourly discharge flows at Bonnet Carré (RM 127) obtained from 1-D HEC-RAS model on storm surge developed by Terán et al. (2013). The downstream end, Head of Passes (RM 0), and outlets (Main Pass, West Bay, Baptiste Collette, Grand Pass + Tiger Pass, Fort St, Philip and Bohemia Spillway) boundary conditions consist on hourly stage values. Figure 29 shows the grid and locations where boundary conditions are given.



Figure 29. Original Coarse Grid with Boundary Condition Locations

3.4.3 Other important parameters

Other important parameters to set up the hurricane model were the bathymetry, initial conditions, time step and the roughness.

The bathymetry file used consisted on a varied depth distribution going from 10m to 25m in different areas of the reach. Most outlets have a 10m depth, except for Bohemia which has 5m. Figure 30 shows the bathymetry along the domain.



Figure 30. Depth varied distribution along the domain for first grid

For the initial condition a 2m uniform water level was set. The time step used for the hydrodynamic simulations was 0.4 min. The roughness was set to a 0.02 uniform value for the Manning's roughness coefficient in both components.

4. **RESULTS**

This chapter presents the results obtained for the different periods simulated, showing hydrodynamics by looking at stage, discharge and velocity profiles; and sediment transport of non-cohesive sediment in different stations along the modeling domain. The results presented also include the application of the model to hurricane storm surge simulations under a two dimensional approach.

4.1 Hydrodynamics

The hydrodynamic simulation results are presented for the different periods evaluated. Results for water levels, discharges and velocities are shown along different stations of the river.

The hydrodynamic simulations were mainly run for three periods. The first period simulations were performed from 03/25/2011 to 04/10/2011, referred to as March/April 2011. The second one corresponds to simulations run from 05/10/2011 to 06/01/2011, referred to as May 2009; and the simulations for the third one started on 05/10/2009 and ended on 05/30/2009. Some other periods were run in order to obtain velocity profiles and non-cohesive sediment transport results based on the data available.

4.1.1 Stage Results

The model stage results are plotted against observed data for the different stations. The observed data is obtained from the U.S. Army Corps of Engineers (Rivergages) website (USACE:rivergages, 2012).

4.1.1.1 March/April 2011

Results for some stations along the domain for the March/April 2011 period are presented in this section. Figure 31 shows the observed and simulated stage values at the upstream end, Bonnet Carré (RM 127) of the modeling domain, Figure 32 presents the observed and simulated water levels at New Orleans (RM 103) and Figure 33 shows the stage at Venice (RM 11)



Figure 31. Simulated and Observed Water Level at Bonnet Carré - March/April 2011



Figure 32. Simulated and Observed Water Level at New Orleans - March/April 2011



Figure 33. Simulated and Observed Water Level at Venice - March/April 2011

A water level longitudinal profile is presented on Figure 34, displaying the observed and modeled stage results along the modeling reach.



Figure 34. Water Level Profile along the Main Channel - March/April 2011

The metrics analysis was performed to observe the agreement in observed and simulated data. The metrics for stage values is presented on Table 5, displaying the values for the different station and determining the overall efficiency of the model.

Station	RMSE (%)	Bias (ft)	Overall r
Bonnet Carré (RM 127)	1	-0.03	
New Orleans (RM 103)	5	-0.60	
IHNC Lock (RM 93)	8	-0.89	0.00
West Point a la Hache (RM	2	0.04	0.99
49)			
Venice (RM 11)	3	0.03	

Table 5. Metrics for stage results – March/April 2011

4.1.1.2 May 2011

The observed and simulated stage values for the different stations during these period simulations are presented next. Figure 35 shows the stage values at Bonnet Carré; Figure 36 presents the observed and simulated stage values at New Orleans; and Figure 37 displays the stage values at Venice.



Figure 35. Simulated and Observed Water Level at Bonnet Carré - May 2011



Figure 36. Simulated and Observed Water Level at New Orleans - May 2011



Figure 37. Simulated and Observed Water Level at Venice - May 2011

A longitudinal profile for the stage values is shown in Figure 38, presenting the observed and simulated data along the channel for the May 2011 period.



Figure 38. Water Level Profile along the Main Channel - May 2011

The metrics were determined for the period of May 2011; the results are presented on Table 6. Root mean square error and Bias error were determined for each station analyzed in the model, and the overall efficiency of the model was also calculated.

Table 6. Whether for stage results – Whay 2011						
Station	RMSE (%)	Bias (ft)	Overall r			
Bonnet Carré (RM 127)	2	-0.39				
New Orleans (RM 103)	6	-0.90				
IHNC Lock (RM 93)	9	-1.25	0.00			
West Point a la Hache (RM	3	-0.17	0.99			
49)						
Venice (RM 11)	4	-0.10				

 Fable 6. Metrics for stage results – May 2011

4.1.1.3 May 2009

The observations and simulated values of stage for the different stations during May 2009 period are plotted in this section. Stage values at Bonnet Carré are shown in Figure 39; observed and simulated stage values at New Orleans are presented in Figure 40; and the stage values at Venice are in Figure 41.

Also, a longitudinal profile was developed to show the stage values along the main channel for the observed and modeled data. The water level profile for May 2009 is presented in Figure 42.



Figure 39. Simulated and Observed Water Level at Bonnet Carré - May 2009



Figure 40. Simulated and Observed Water Level at New Orleans - May 2009



Figure 41. Simulated and Observed Water Level at Venice - May 2009



Figure 42. Water Level Profile along the Main Channel - May 2009

The metrics for stage results on the May 2009 period are presented in Table 7.

Station	RMSE (%)	Bias (ft)	Overall
			r
Bonnet Carré (RM 127)	3	-0.09	
New Orleans (RM 103)	4	-0.48	
IHNC Lock (RM 93)	7	-0.78	0.00
West Point a la Hache (RM	12	0.58	0.99
49)			
Venice (RM 11)	7	0.24	

Table 7. Metrics for stage results - May 2009

4.1.2 Discharge Results

The results obtained for the flow on the main outlets and some important points in the modeling domain are presented in this section.

4.1.2.1 March/April 2011

The upstream end flow corresponding to Bonnet Carré is shown in Figure 43. Also, the downstream end discharge is shown in figure 44. Flow at Fort St. Philip, one of the main outlets in the modeling domain, is presented in Figure 45.



Figure 43. Flow at Bonnet Carré – March/April 2011



Figure 44. Flow at Head of Passes - March/April 2011



Figure 45. Flow at Fort St Philip – March/April 2011

The flow distribution for main outlets and other important points is shown in Figure 46. Two set of estimated data for the discharge were used. One is based on the estimate provided by Lake Pontchartrain Basin Foundation (Lopez & Lake Pontchartrain Basin Foundation , 2008); and the second one corresponds to a set of data based on an estimate provided by USACE (U.S. Army Corps of Enginners, 2013).



Figure 46. Estimated and Modeled Discharge for U/S, D/S and outlets - March/April 2011

Based on the two different estimates for discharge a range of error percentage is presented in Table 8. Also the overall efficiency and RMSE are shown.

1	Table 6. Estimated and Wodeled Discharge Values – Warch/April 2011							
	LPBF Estimated	USACE Estimated	Modeled	LPBF	USACE	Mean	Overall	Overall
River Station	Flow (ft ³ /s)	Flow (ft ³ /s)	Flow (ft ³ /s)	%Difference	%Difference	%Difference	r	RMSE %
Bonnet Carré	993159	993159	993159	0%	0%	0%		
Head Of Passes	481154	482505	499098	-4%	-3%	-4%		
Venice		765469	748118		2%	2%		
Grand+Tiger Pass	92486	84202	90599	2%	-8%	-3%	0.00	50/
Baptiste Collette	130130	76547	97447	25%	-27%	-1%	0.99	576
Main Pass	97571	91856	95044	3%	-3%	0%		
Bohemia Spillway	86094	130483	101913	-18%	22%	2%		
Fort St Philip	70386	85034	76748	-9%	10%	0%		

Table 8. Estimated and Modeled Discharge Values – March/April 2011

4.1.2.2 May 2011

Similarly to the previous period presented, the flows corresponding to Bonnet Carré, Head of Passes and Fort St Philip are shown in Figure 47, Figure 48 and Figure 49, respectively for May 2011.



Figure 47. River Flow downstream of Bonnet Carré Spillway- May 2011


Figure 48. Flow at Head of Passes - May 2011



Figure 49. Flow at Fort St Philip – May 2011

Figure 50 displays the 2 sets of estimated values previously mentioned, and the modeled values for the discharges during the May 2011 simulated period.



Figure 50. Estimated and Modeled Discharge for U/S, D/S and outlets - May 2011

The difference percentage between estimated and simulated data is shown in Table 9 for the main outlets and some important points of the main channel. Also, the mean difference based on the two sets of observed data; the overall efficiency coefficient, r; and the overall RMSE percentage is shown.

Table 9. Estimated and Modeled Discharge Values – May 2011									
	LPBF Estimated	USACE Estimated	Modeled	LPBF	USACE	Mean	Overall	Overall	
River Station	Flow (ft ³ /s)	Flow (ft ³ /s)	Flow (ft ³ /s)	%Difference	%Difference	%Difference	r	RMSE %	
Bonnet Carré	1150010	1150010	1150010	0%	0%	0%			
Head Of Passes	557144	540195	604306	-8%	-12%	-10%			
Venice		856992	834419		3%	3%			
Grand+Tiger Pass	107092	94269	97915	9%	-4%	2%	0.00	70/	
Baptiste Collette	150682	85699	103981	31%	-21%	5%	0.99	1 70	
Main Pass	112980	102839	100927	11%	2%	6%			
Bohemia Spillway	99691	169056	142168	-43%	16%	-13%			
Fort St Philip	81502	110171	95230	-17%	14%	-2%]		

Tabla A	Fatime of a			Diaskawas	Valuea	Mar	2011
гаріе ч.	Esumaleo	and v	ioaeiea	Discharge	vames –	VINV	2011
1 4010 > 1	1350111140004		roucieu	Discharge	, and the op		

4.1.2.3 May 2009

For May 2009, flows at Bonnet Carré (downstream), Head of Passes and Fort St Philip are shown in Figures 51, 52 and 53, respectively.



Figure 51. Model Discharge at Bonnet Carré - May 2009



Figure 52. Model Discharge at Head of Passes - May 2009



Figure 53. Model Discharge at Fort St. Philip - May 2009

The flow distribution showing estimated values and simulation results are displayed in Figure 54 for the period May 2009.



Figure 54. Estimated and Modeled Discharge for U/S, D/S and outlets - May 2009

The estimated flows for the most important outlets of the outlets in the modeling domain and some other point in the main channel are presented against the values obtained for the May 2009 simulations are displayed in Table 10. Moreover, the difference percentage, mean difference percentage, overall efficiency and RMSE percentage is shown for the results obtained.

	LPBF Estimated	USACE Estimated	Modeled	LPBF	USACE	Mean	Overall	Overall
River Station	Flow (ft ³ /s)	Flow (ft ³ /s)	Flow (ft ³ /s)	%Difference	%Difference	%Difference	r	RMSE %
Bonnet Carré	1172010	1172010	1172010	0%	0%	0%		
Head Of Passes	567802	547883	578946	-2%	-6%	-4%		
Venice		869189	835433		4%	4%		
Grand+Tiger Pass	109141	95611	96975	11%	-1%	5%	0.00	10/
Baptiste Collette	153564	86919	102718	33%	-18%	7%	0.99	4 70
Main Pass	115142	104303	96343	16%	8%	12%		
Bohemia Spillway	101598	174858	165968	-63%	5%	-29%		
Fort St Philip	83061	113952	113167	-36%	1%	-18%]	

Table 10. Estimated and Modeled Discharge Values - May 2009

4.1.3 Velocity Results

For the velocities a depth average map is presented just to give an idea of the range of velocities found. Moreover, velocity profiles are presented for different periods where observed data is available (Allison M., 2012)

4.1.3.1 March/April 2011

Figure 55 displays the depth average velocity map for the March/April 2011 period for the entire domain. Observed data for this period was not available for this period; consequently, the depth average profile and vertical velocity profile for observed and simulated data are not shown.



Figure 55. Depth averaged velocity map – March/April 2011

Some important areas are zoomed in to observe in more detail the velocity distribution. Figure 56 shows the velocity distributions around the Myrtle Grove area; Figure 57 exposes the velocity distribution around the Bohemia area; Figure 58 presents the velocity distribution for the Fort St Philip area; and Figure 59 displays the velocity distribution around the Main Pass area.



Figure 56. Depth average velocity map (ft/s) for Myrtle Grove area – March/April 2011



Figure 57. Depth average velocity map (ft/s) for Bohemia area – March/April 2011



Figure 58. Depth average velocity map (ft/s) for Fort St Philip area – March/April 2011



Figure 59. Depth average velocity map (ft/s) for Main Pass area – March/April 2011

4.1.3.2 May 2011

A depth average map for the May 2011 period is shown in Figure 60 to observe the range of velocities in the domain.



Figure 60. Depth average velocity map – May 2011

To observe in more detail the velocity distribution, some areas of the map are amplified in the next figures. Figure 61 illustrates the velocity distributions around the Myrtle Grove area; Figure 62 presents the velocity distribution around the Bohemia area; Figure 63 shows the velocity distribution for the Fort St Philip area; Figure 64 displays the velocity distribution around the Main Pass area.



Figure 61. Depth average velocity map (ft/s) for Myrtle Grove area – May 2011



Figure 62. Depth average velocity map (ft/s) for Bohemia area – May 2011



Figure 63. Depth average velocity map (ft/s) for Fort St Philip area – May 2011



Figure 64. Depth average velocity map (ft/s) for Main Pass area – March/April 2011

4.1.3.3 May 2009

The data available for the velocities on this period corresponds to the area around Magnolia (RM 47). Table 11 displays the overall RMSE and efficiency coefficient r for this period.

Table 11. Metrics for Velocity Profiles – May 2009							
Profile	Overall % DMSE	Overall r					
	70 KIVISE						
Cross-sectional	7	0.98					
Vertical	22	0.97					

Figure 65 shows the cross-sectional profile of the measurements and simulated depth averaged velocity.



Figure 65. Cross-Sectional Velocity Profile, RM 47 – May 2009

The vertical velocity profile for the Magnolia area during the May 2009 run is presented on Figure 66.



Figure 66. Vertical Velocity Profile, RM 47 – May 2009

The depth average velocity map for May 2009 simulation is shown in Figure 67.



Figure 67. Depth average velocity map – May 2009

4.1.3.4 September 2009

In order to observe the behavior on velocities compared to available observations on the Empire area (RM 31) the velocity profiles were obtained. Table 12 shows the overall RMSR and r for this period around the Empire area.

Table 12. Metrics for Velocity Profiles – September 2009							
Profile	Overall	Overall r					
	%RMSE						
Cross-sectional	8	0.84					
Vertical	23	0.83					

e •

Figure 68 present the depth averaged velocity profile, and Figure 69 displays the vertical velocity profile for this period.



Figure 68. Cross-Sectional Velocity Profile, RM 31 – September 2009



Figure 69. Vertical Velocity Profile, RM 31 – September 2009

4.1.3.5 April 2010

The depth average velocity profile is shown in Figure 70 and the vertical velocity profile in Figure 71 for the Magnolia area (RM 46) for April 2010. The observations and simulation results are plotted to observe the agreement. Table 13 presents the overall RMSE and coefficient of efficiency

	v clocity i romes h	pin 2010
Profile	Overall	Overall r
	%RMSE	
Cross-sectional	23	0.80
Vertical	33	0.97

Table 13. Metrics for Velocity Profiles – April 2010



Figure 70. Cross-Sectional Velocity Profile, RM 46 – April 2010



Figure 71. Vertical Velocity Profile, RM 46 – April 2010

4.2 Sediment Transport

The non-cohesive sediment transport results are presented in this section. Figures 72 and 73 show the locations where observed data (Allison, 2011) is available for the periods evaluated.



Figure 72. Sediment measurement locations at Myrtle Grove area (RM 61)



Figure 73. Sediment measurement locations at Magnolia area (RM 47)

4.2.1 March/April 2011

The non-cohesive sediment run for this period was performed from 03/27/2011 to 04/02/2011. The suspended sand concentrations and loads measurements (Allison, 2011) were plotted against the modeled results to calibrate the model.

Figures 74, 75 and 76 show the observed and simulated suspended sand concentrations for Myrtle Grove area (RM 61) for the March/April 2011 period.



Figure 74. Simulated and Observed Suspended Sand Concentration at Myrtle Grove, MGup2 (RM 61). March/April 2011



Figure 75. Simulated and Observed Suspended Sand Concentration at Myrtle Grove, MGup3 (RM 61). March/April 2011



Figure 76. Simulated and Observed Suspended Sand Concentration at Myrtle Grove, MGup4 (RM 61). March/April 2011

Figures 77, 78 and 79 show the observed and simulated suspended sand concentrations for Magnolia area (RM 47) for the March/April 2011 period.



Figure 77. Simulated and Observed Suspended Sand Concentration at Magnolia, MAG1 (RM 47). March/April 2011



Figure 78. Simulated and Observed Suspended Sand Concentration at Magnolia, MAG2 (RM 47). March/April 2011



Figure 79. Simulated and Observed Suspended Sand Concentration at Magnolia, MAG3 (RM 47). March/April 2011

Table 14 exhibits the measurements for non-cohesive sediment load against the model results obtained for Myrtle Grove area (RM 61) and Magnolia area (RM 47) for the March/April 2011 period.

2011							
(Tonnes/day)	Area	Observed	Simulated	Difference	Overall r	Overall %RMSE	
Podlood	Myrtle Grove	15094	16750	-11%	0.99		
Bed Load	Magnolia	12403	17011	-37%		5%	
Suspended	Myrtle Grove	199533	201725	-1%			
Load	Magnolia	190874	177242	7%		570	
Total Load	Myrtle Grove	214627	218474	-2%			
	Magnolia	203277	194253	4%			

Table 14. Measured and Modeled Bed Load, Suspended Load and Total Load – March/April2011

4.2.2 May 2011

The non-cohesive sediment simulation for this period was performed from 05/11/2011 to 05/15/2011. The suspended sand concentrations and loads measurements were plotted against the modeled results to observe the agreement in both data series.

Figures 80, 81 and 82 display the observed and simulated suspended sand concentrations for Myrtle Grove area (RM 61) for the May 2011 period.



Figure 80. Simulated and Observed Suspended Sand Concentration at Myrtle Grove, MGup2 (RM 61). May 2011



Figure 81. Simulated and Observed Suspended Sand Concentration at Myrtle Grove, MGup3 (RM 61). May 2011



Figure 82. Simulated and Observed Suspended Sand Concentration at Myrtle Grove, MGup4 (RM 61). May 2011

The observed and simulated suspended sand concentrations for the Magnolia area (RM 47) during the May 2011 period are presented on Figures 83, 84 and 85.



Figure 83. Simulated and Observed Suspended Sand Concentration at Magnolia, MAG1 (RM 47). May 2011



Figure 84. Simulated and Observed Suspended Sand Concentration at Magnolia, MAG2 (RM 47). May 2011



Figure 85. Simulated and Observed Suspended Sand Concentration at Magnolia, MAG3 (RM 47). May 2011

Table 15 shows the measurements for non-cohesive sediment load against the modeled values obtained for Myrtle Grove area (RM 61) and Magnolia area (RM 47) for the May 2011 period.

(Tonnes/day)	Area	Observed	Simulated	Difference	Overall r	Overall %RMSE	
Pod Lood	Myrtle Grove	13686	18801	-37%		22%	
Bed Load	Magnolia	47488	22882	52%			
Suspended	Myrtle Grove	155541	118626	24%	0.00		
Load Total Load	Magnolia	111058	109111	2%	0.98	23%	
	Myrtle Grove	169227	137427	19%			
	Magnolia	158546	131993	17%			

Table 15. Measured and Modeled Bed Load, Suspended Load and Total Load – May 2011

4.2.3 May 2009

The non-cohesive sediment run for this period was performed from 05/01/2009 to 05/07/2009. The suspended sand concentrations and loads measurements were plotted against the modeled results to calibrate the model.

Figures 86, 87 and 88 show the observed and simulated suspended sand concentrations for Myrtle Grove area (RM 61) for the May 2009 period.



Figure 86. Simulated and Observed Suspended Sand Concentration at Myrtle Grove, MGup2 (RM 61). May 2011



Figure 87. Simulated and Observed Suspended Sand Concentration at Myrtle Grove, MGup3 (RM 61). May 2009



Figure 88. Simulated and Observed Suspended Sand Concentration at Myrtle Grove, MGup4 (RM 61). May 2009

The observed and simulated suspended sand concentrations for the Magnolia area (RM 47) for the May 2011 period are presented in Figures 89, 90 and 91.



Figure 89. Simulated and Observed Suspended Sand Concentration at Magnolia, MAG1 (RM 47). May 2011



Figure 90. Simulated and Observed Suspended Sand Concentration at Magnolia, MAG2 (RM 47). May 2009



Figure 91. Simulated and Observed Suspended Sand Concentration at Magnolia, MAG3 (RM 47). May 2009

The observed and simulated bed load, suspended load and total load for the May 2009 period are presented in Table 16.

(Tonnes/day)	Area	Observed	Simulated	Difference	Overall r	Overall %RMSE	
Podlood	Myrtle Grove	6684	6632	1%	0.88		
Bed Load	Magnolia	5943	7802	-31%			
Suspended	Myrtle Grove	26824	28580	-7%		0.88	>E0%
Load	Magnolia	103568	40349	61%		>30%	
Total Load	Myrtle Grove	33508	35212	-5%			
	Magnolia	109511	48151	56%			

 Table 16. Measured and Modeled Bed Load, Suspended Load and Total Load – May 2009

4.3 Application for Hurricanes

The 2-D hydrodynamic model developed at the first stage was applied to hurricane periods corresponding to Isaac and Gustav to test the robustness of the model, since a two dimensional model is well suited for storm surge applications.

4.3.1 Hurricane Isaac

The simulation for Hurricane Isaac was performed in the period starting from 08/27/2012 to 09/01/2012. The results obtained for the 2-D hydrodynamic model at different stations along the domain compared to the observations by the US Army Corps of Engineers (Personal Communication) are presented in this section.

Figure 92 displays the stage values obtained from the model at New Orleans (RM 103) station compared to the measured values for the same period. Similarly, Figure 93 shows the stage at Harvey Lock (RM 93).



Figure 92. Simulated and Observed Stage at New Orleans (RM 103) for Isaac

The surge height was determined along the channel during Hurricane Isaac and results are represented on Figure 94 as a longitudinal profile.

Table 17 shows the %RMSE and bias for the peak stage value for the Hurricane Isaac.



Figure 93. Simulated and Observed Stage at Harvey Lock (RM 98) for Isaac



Figure 94. Surge height along the Main Channel for Isaac

Station	Stag	e (ft)		Riac (ft)	
Station	Modeled Observed		%RIVIJE	Dias (IL)	
New Orleans (RM 103)	11.4	11.9			
Harvey Lock (RM 98)	11.4	11.6	20/	0.1	
IHNC Lock (RM 93)	11.28	11.03	570		
Algiers Lock (RM 88)	11.02	10.97			

Table 17. Metrics for Peak Stage Prediction – Hurricane Isaac

4.3.2 Hurricane Gustav

The simulation for Hurricane Gustav was performed in the period starting from 08/30/2008 to 09/01/2008. The results for the 2-D hydrodynamic model and the observed values by the USACE at different stations are shown in the next figures. Figure 95 plots the simulated and observed stage for New Orleans during Gustav Hurricane; Figure 96 displays the values for West Point a la Hache; and Figure 97 shows results and observations for Venice.



Figure 95. Simulated and Observed Stage at New Orleans (RM 103) for Gustav



Figure 96. Simulated and Observed Stage at West Point a la Hache (RM 49) for Gustav



Figure 97. Simulated and Observed Stage at Venice (RM 11) for Gustav

Figure 98 displays the longitudinal profile of surge height for Hurricane Gustav indicating the stations that were evaluated in the simulations. Table 18 displays the %RMSE and bias for the peak stage value for the Hurricane Gustav.



Figure 98. Surge height along the Main Channel for Gustav

Station	Stag	e (ft)		Dies (ft)	
Station	Modeled Observed		70KIVIJE	Dids (IL)	
Bonnet Carré (RM 127)	9.90	10.40		0.5	
New Orleans (RM 103)	9.85	10.30	69/		
Harvey Lock (RM 98)	9.61	10.34	0%		
Algiers Lock (RM 88)	9.31	9.90			

Table 18. Metrics for Peak Stage Prediction – Hurricane Gustav

5. DISCUSSION

5.1 Hydrodynamics

5.1.1 Stage Results

Based on the observations available for stage values (USACE:rivergages, 2012), graphical and numerical comparisons were performed in order to determine the agreement between the model and the measurements.

5.1.1.1 March/ April 2011

For the graphical comparison, there is a very good agreement for most stations with the observations. Based on the metrics analysis, for New Orleans (RM 103) the model is underpredicting a 5% and for IHNC Lock (RM 93) an 8%, being this last the station with the highest %RMSE. All stations %RMSE is under 9%, being lower than the target which is <15% (Meselhe E. , 2013), indicating a satisfactory prediction based on water level evaluation. The overall efficiency coefficient is 0.99 and the overall RMSE is 4%.

The highest bias for this period is 0.89ft which is less than 1ft that is satisfactory based to the high target of <1ft for all stations (Meselhe E. , 2013).

From the longitudinal profile it can be observed that the model is following the trend of the observations along the main channel.

5.1.1.2 May 2011

Similarly, the validation period results corresponding to May 2011 are in good agreement with the observations for that period, which can be observed from the graphical comparison. Moreover, based on the metrics analysis, the highest %RMSE, which corresponds to 9% for IHNC Lock is still under the defined target high limit of <15%. The overall efficiency coefficient is 0.99 and the overall %RMSE is 5%, representing all satisfactory results.

The highest bias corresponds to IHNC Lock with a 1.25ft value (absolute value) which is above the high limit target of 1ft; however, the 80% of the stations meet the low limit target being under the <1ft.

The simulation results follow the trend of the observations, which can be seen on the water level longitudinal profile for this period.

5.1.1.3 May 2009

For the May 2009 period there is also very good agreement between the measurements and the simulation results. The metrics analysis showed satisfactory results for the water level. The highest %RMSE corresponds to West Point a la Hache (RM 49) with a 12%, being this value lower than the high limit target of <15%. Additionally, the overall efficiency is 0.99 and the overall %RMSE is 7%.

The highest bias found in this simulation period was 0.78ft (absolute value), which meets the target on the higher limit of <1 ft.

Similarly to the previous simulations, the results for the water levels follow the trend of the measurements, which can be observed on the longitudinal profile for the stage presented on the results section for this period.

5.1.2 Discharge Results

To determine the performance of the model for discharge distribution prediction, the results obtained from the simulations were compared to estimated values from two different sources; one is based on the estimate provided by Lake Pontchartrain Basin Foundation (Lopez & Lake Pontchartrain Basin Foundation , 2008); and the second one is based on an estimate provided by USACE (U.S. Army Corps of Enginners, 2013).

When looking at the two different sources for the estimated values, for some areas it is found a difference between the observations of about 20%. Moreover, it is important to consider that there is about a 5% to 8% of error that is carried in the measurements based on the river flow, and that based on Tarbert Landing there is about an 8% error on the measurements.

For the overflow areas in the domain, such as Bohemia; West Bay; Fort St Philip; among others, it is significant to observe that the flow estimation depends on the exact reach where the flow was measured, and if the cuts present on the nearby areas were taken into account. Moreover, equivalent channels are being used in the model on the overflow areas to represent some of the cuts on the nearby zones.

It is not possible to rely only on one source, for this reason; by considering all the previous information, the calibration of the model for the discharge was performed to target the mean difference of the two set of observation based estimated flows

5.1.2.1 March/April 2011

A graphical comparison is presented to visually appreciate the agreement of the estimated values and the model results. For most of the outlets there is a good agreement based on the estimated average.

The overall comparison between the simulated values and the estimated values shows satisfactory results, being the overall efficiency 0.99 and the overall %RMSE 5%. For the evaluation on the main outlets it was found that a mean difference percentage of -4% was the highest value.

For most of the outlets the flows fall under the mean difference of the estimated values. West Bay (RM 4) is a particular case, the estimated values range between 38273cfs and 42882cfs being the model result 69677cfs, which indicates that the model is overpredicting by a significant amount. However, as it was stated before, there are some cuts on the west bank of the river that represent an important flow extraction, and West Bay is extracting the flow for those cuts to ensure the mass conservation in the domain.

5.1.2.2 May 2011

For this validation period the overall efficiency was 0.99 and the overall %RMSE was 7%. The highest mean difference corresponds to Bohemia Spillway with a -13%. Once more, it is important to consider the error present in the observations, and the existence of cuts that might not be included in one of the measurement estimates.

The West Bay outflow, as explained for the previous period is expected to be higher than the observed since the model is including some of the cuts on the west bank. The estimated flow by LPBF for West Bay was 49655cfs; the estimated by USACE was 42850cfs, while the one determined by the model was 73400cfs.

5.1.2.3 May 2009

Similarly for this validation period, there was an overall efficiency of 0.99 and an overall %RMSE of 4%. The highest mean difference corresponds to Bohemia Spillway area; however, it is important to remember that this is one of the main areas where cuts are being represented by equivalent channels, meaning this that the flow extraction by the model is expected to be higher than the measured value. The same behavior is observed for Baptiste Collette and Fort St Philip in a lower scale.

For West Bay during this period the value based on the LPBF estimate was 50604cfs; the one based on the USACE estimate was 43459cf, and the value obtained from the simulation was 71000cfs.

5.1.3 Velocity Results

The velocity distribution in the domain was obtained by representing the depth average velocity in a map for some periods (no observations available), and by obtaining depth averaged velocity profiles and vertical velocity plotted against available observed data (Allison M., 2012) for some other periods.

5.1.3.1 March/ April 2011

For this period the depth averaged velocity distribution was presented in a map format. The expected range for the velocity based on observations for other periods is between 3ft/s to 7ft/s. It can be observed on the map that the velocities in the main channel for Myrtle Grove (RM 61) and Bohemia (RM 45) are about 6.5ft/s; for Fort St Philip (RM 20) is about 5ft/s; and for the area near Main Pass (RM 4) is about 4.5ft/s, being all in the expected range.
5.1.3.2 May 2011

In this period, based on the map distribution, velocities around Myrtle Grove and Bohemia area in the main channel were about 7ft/s. The velocities for the Fort St Philip area and near Main Pass were about 6ft/s. Once more, the velocities were on the expected range.

5.1.3.3 May 2009

For this period there was available data and it was possible to build a depth averaged velocity profile and a vertical velocity profile plotting the measurements and the model results for the Magnolia area (RM 47). Based on the profiles it can be observed a very good agreement of the model results with the measurements. Furthermore, the metrics show satisfactory results based on the target. The efficiency for the cross-sectional profile was 0.98; the efficiency for the vertical profile was 0.97, and the target is an efficiency >0.75 (Meselhe E. , 2013). The target for the %RMSE is <30%, the %RMSE was 7% for the depth averaged velocity; and 22% for the vertical profile, meaning the model has a satisfactory performance for velocity prediction.

Based on the map of depth averaged velocities, the velocities fall into the expected range of 3ft/s to 7ft/s.

5.1.3.4 September 2009

This period was run to test the performance of the model on velocity prediction, since there was available measured data. Based on the graphical comparison, there is a good agreement between observations and model results. Moreover, the metrics show an efficiency of 0.84 for the depth averaged velocity and a 0.83 for the vertical velocity, being both above the target of >0.75. The %RMSE for the cross-sectional velocity was 8% and for the vertical velocity was 23%, being both over the target of >30% (Meselhe E. , 2013).

5.1.3.5 April 2010

Based on the comparison of available observed data and the model for this period, the performance of the model on velocity prediction was evaluated. The graphical comparison shows a fairly good agreement between observed and model values. The efficiency for the

depth averaged velocity was 0.80 and for the vertical velocity 0.97, meaning the target of >0.75 is being achieved. For the depth averaged velocity the %RMSE was 23%, which meets the target value statement of %RMSE>30%.For the vertical velocity the %RMSE was 33% which is a above the target value; however, it is still acceptable since the velocities fall under the expected range of values of 3ft/s to 7ft/s.

5.2 Sediment Transport Results

The non-cohesive sediment transport prediction of the model was evaluated by comparing the model results with measurements (Allison, 2011) corresponding to the same periods.

5.2.1 March/April 2011

There is a good agreement on the suspended sand concentration profiles for both Myrtle Grove area (RM 61) and Magnolia area (RM 47). Moreover, the model showed a 0.99 overall efficiency and an overall %RMSE of 5%. There is a very good agreement in the total load for both Myrtle Grove and Magnolia. The suspended load prediction is also very satisfactory for this period on the two areas analyzed; and the bed load prediction for Myrtle Grove is fairly good, while for Magnolia the model is over-predicting for a 37%, which is still a satisfactory value based on the target of <50% (Meselhe E. , 2013).

5.2.2 May 2011

For this period based on the suspended sand concentration profiles there is a good agreement between observations and model results. The overall efficiency is 0.94 and the overall %RMSE is 20%. The total load prediction gave very satisfactory results for both the Myrtle Grove and Magnolia area. The suspended load estimation was fairly good being the highest difference percentage for the Myrtle Grove area with an over-prediction of 24%. The bed load for the Magnolia area was under-predicted by a 52%, while the bed load for the Myrtle Grove area was over-predicted by a 37%, being these values still satisfactory for the sand transport prediction.

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5.2.3 May 2009

There is a good agreement on the suspended sand concentration for Myrtle Grove, and a fairly good agreement for Magnolia based on the profiles for this period. Furthermore, for the Myrtle Grove sediment transport prediction results were very satisfactory. The bed load, suspended load and total load %difference were below 15% for the Myrtle Grove area. However, the performance for Magnolia on this period was not as good as expected still being acceptable. The bed load estimation for Magnolia resulted in very satisfactory results, not obtaining so plausible for suspended and total load with a 61% and 56% of underprediction respectively.

5.3 Application for Hurricanes Results

The 2-D hydrodynamic model was applied to the analysis of stage during Hurricanes Isaac and Gustav.

5.3.1 Hurricane Isaac

From the graphical comparison between available observed data (USACE:rivergages, 2012) and the model results there is a good agreement on the peak prediction. Moreover, the shape of the simulated stage hydrographs follows the tendency of the observations hydrograph.

Based on the peak for the stage the overall %RMSE was 3% and the bias 0.1ft, which shows a very good performance on the peak prediction.

Also, the surge height was predicted along the main channel showing a value of about 8.5ft for New Orleans. Based on a 1-D HEC-RAS model for storm surge prediction (Gurung T., 2012) the storm surge propagated as far as Tarbert Landing (RM 302) with a surge height of about 4ft.

5.3.2 Hurricane Gustav

Based on the graphical comparison, the model had a fairly good performance for the peak stage prediction, also following the tendency of the observations available.

The overall %RMSE based on the peak stage was 6% and the bias was 0.6ft, showing once more a good performance of the model for the peak stage prediction. The surge height based on the Gustav data was about 5ft for the upstream end of the model of Bonnet Carré (RM 127). For the hurricane Gustav, the storm surge also propagated as far as Tarbert Landing with a surge height (Gurung T. , 2012) was about 2ft.

6. CONCLUSIONS

The following conclusions have been derived from this study:

- The three dimensional hydrodynamic model of the Lower Mississippi river was calibrated for the March/April 2011 period for stage prediction with an overall efficiency of 0.99, an overall %RMSE of 4% and an overall bias -0.29ft. The %RMSE for all stations meets the high target of <15% for all stations, being the highest %RMSE the IHNC Lock result corresponding to an 8%, and the lowest %RMSE corresponds to Bonnet Carré with a 1%.
- The May 2011 validation period for the hydrodynamic model lead to satisfactory results on stage prediction. The overall efficiency was 0.99; the overall %RMSE was 5%; and the overall bias -0.74. The %RMSE for all stations meets the high limit of the target of <15%. 80% of the stations meet the target of <1ft for the bias, being only one station above the value with 1.25ft corresponding to IHNC Lock.
- For the May 2009 hydrodynamic validation period, stage results were also satisfactory. Finding an overall efficiency of 0.99; an overall %RMSE of 6%; and the overall bias -0.1ft. All stations presented a %RMSE <15% meeting the target. Additionally, the bias for all stations was below 1ft, meeting once more the high limit of the target for stage prediction.
- The discharge prediction for all periods was satisfactory based on the mean difference percentage for the estimates from LPBF and USACE. The overall efficiency for all the periods was 0.99, and the %RMSE was below 10%.
- The velocities were in the expected range between 3ft/sec to 7ft/sec along the domain for all the periods evaluated. Moreover, the period with available observed data showed a very good agreement between the measurements and the simulated values, meeting the target for a %RMSE <30% and an efficiency r > 0.75.
- The non-cohesive sediment prediction in the Myrtle Grove and Magnolia area, lead to satisfactory results. For the calibration period the overall %RMSE was 5%, obtaining a good prediction of the total load, bed load and suspended load.

- For the validation period of May 2011, the total load prediction resulted in very satisfactory results. The model showed acceptable values for bed load and suspended load in both Magnolia and Myrtle Grove areas.
- For May 2009, the model had a very good performance for the Myrtle Grove area. The bed load prediction for the Magnolia area was very good, while suspended and total load showed an under-prediction over 70%.
- The application of the 2-D model for hurricane storm surge simulation was satisfactory for both Hurricane Isaac and Hurricane Gustav. The model showed a very good prediction of the peak stage during these events, and followed the hydrograph shape based on the observations available.

7. RECOMMENDATIONS

- For further research on the hurricane application, better results might be obtained by adapting all outlets depths to more realistic values and also varying roughness along the modeling domain, for instance; depth and Manning's n roughness files from the three dimensional model could be used to improve this model.
- The bedform option in Delft3D should be calibrated and evaluated in comparison to Chezy and Manning's roughness approaches.

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APPENDIX A: Boundary Conditions

This appendix shows plots for the boundary conditions used in the different periods simulated.



Hydrodynamics and Sediment Transport Simulation















Application for Hurricanes

































APPENDIX B: Other results

Hydrodynamics

• Stage













• Flow





















































Hurricane Application











APPENDIX C: Original Grid and Expanded Grid Comparisons



<u> Original Grid for Main Channel – Upstream Bohemia</u>

<u>Expanded Grid for Main Channel – Upstream Bohemia</u>



Original Grid for Bohemia Area



Expanded Grid for Bohemia Area



Original Grid for Fort St Philip Area



Expanded Grid for Fort St Philip Area


VITA

Grecia Alejandra Terán Gonzalez was born in Maracaibo, in Zulia state, Venezuela on November 24, 1985. In 2002, she graduated from High School in her hometown. In December 2008 she obtained a 5-year degree of Bachelor in Chemical Engineering at the Universidad del Zulia (University of Zulia) in Venezuela.

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