Coupling Of Hydrodynamic And Wave Models For Storm Tide Simulations: A Case Study For Hurricane Floyd (1999)

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COUPLING OF HYDRODYNAMIC AND WAVE MODELS FOR STORM TIDE SIMULATIONS: A CASE STUDY FOR HURRICANE FLOYD (1999)

by

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A dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in the Department of Civil and Environmental Engineering in the College of Engineering and Computer Science at the University of Central Florida Orlando, Florida

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Major Professor: Scott C. Hagen
ABSTRACT

This dissertation presents the development of a two-dimensional St. Johns River model and the coupling of hydrodynamic and wave models for the simulation of storm tides. The hydrodynamic model employed for calculating tides and surges is ADCIRC-2DDI (ADvanced CIRCulation Model for Shelves, Coasts and Estuaries, Two-Dimensional Depth Integrated) developed by Luettich et al. (1992). The finite element based model solves the fully nonlinear shallow water equations in the generalized wave continuity form. Hydrodynamic applications are operated with the following forcings: 1) astronomical tides, 2) inflows from tributaries, 3) meteorological effects (winds and pressure), and 4) waves (wind-induced waves). The wave model applied for wind-induced wave simulation is the third-generation SWAN (Simulating WAves Nearshore), applicable to the estimation of wave parameters in coastal areas and estuaries. The SWAN model is governed by the wave action balance equation driven by wind, sea surface elevations and current conditions (Holthuijsen et al. 2004).

The overall work is comprised of three major phases: 1) To develop a model domain that incorporates the entire East Coast of the United States, Gulf of Mexico and Caribbean Sea, while honing in on the St. Johns River area; 2) To employ output from the SWAN model with the ADCIRC model and produce a uni-directional coupling of the two models in order to investigate the effects of the wave radiation stresses; 3) To couple the ADCIRC model with the SWAN model to describe the complete interactions of the two physical processes.
Model calibration and comparisons are accomplished in three steps. First, astronomical tide simulation results are calibrated with historical NOS (National Ocean Service) tide data. Second, overland and riverine flows and meteorological effects are included, and computed river levels are compared with the historical NOS water level data. Finally, the storm tides generated by Hurricane Floyd are simulated and compared with historical data. This research results in a prototype for real-time simulation of tides and waves for flash flood and river-stage forecasting efforts of the NWS Forecasting Centers that border coastal areas.

The following two main conclusions are reported: 1) regardless of whether one uses uni-coupling or coupling, wind-induced waves result in an approximately 10 – 15 % higher peak storm tide level than without any coupling; and 2) the wave-current interaction described by the coupling model results in decreasing peaks and increasing troughs in the storm tide hydrograph. Two main corollary conclusions are also drawn from a 122-day hindcast for the period spanning June 1 – October 1, 2005. First, wind forcing for the St. Johns River is equal to or greater than that of astronomic tides and generally supersedes the impact of inflows, while pressure variations have a minimal impact. Secondly, water levels inside the St. Johns River depend on the wind forcings in the deep ocean; however, if one applies an elevation hydrograph boundary condition from a large-scale domain model to a local-scale domain model the results are highly accurate.
ACKNOWLEDGMENTS

I would like to express my appreciation to those people whose assistance helped me finalize this research. First, I would like to thank Dr. Scott C. Hagen for his exceptional support and advice on this project as well as the many pleasant conversations I had with him during my stay at UCF. I also would like to thank Dr. Gour-Tsyh Yeh, Dr. Manoj B. Chopra, Dr. F. Necati Catbas, and Dr. Alain Kassab for agreeing to serve on my committee; Dr. Pedro Restrepo of NOAA/NWS/OHD and Ms. Reggina Cabrera of SERFC, for providing the vital information about the St. Johns River; Dr. Peter V. Sucsy of SJRWMD, for providing the bathymetric data associated with the St. Johns River; Andrew T. Cox of Oceanweather Inc., for providing the wind field information; R. E. Jensen of USACE, for providing the wave field information; Peter Bacopoulos, for checking the English usage in this dissertation; and many thanks to both current and past lab members: Daniel Dietsche, Derek Giardino, David Coggin, Juliano Elias, Michael Parrish, Mike Salisbury, Ryan Murray, Satoshi Kojima, Naeko Takahashi, and Qing Wang. Last but not least, I am very grateful to have such a wonderful family in my life.

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TABLE OF CONTENTS

LIST OF FIGURES ........................................................................................................................ x
LIST OF TABLES ........................................................................................................................ xvii
LIST OF ABBREVIATIONS .......................................................................................................... xviii

CHAPTER 1 INTRODUCTION ....................................................................................................1
  1.1 The Western North Atlantic Tidal (WNAT) Model Domain ............................................. 4
  1.2 The St. Johns River .......................................................................................................... 5
  1.3 Hurricane Floyd ............................................................................................................. 6
  1.4 Research Objective ........................................................................................................ 8

CHAPTER 2 WAVE MECHANICS AND DYNAMICS ............................................................ 11
  2.1 The Basic Types of Ocean Waves .................................................................................... 11
  2.2 Wind Waves ................................................................................................................... 14
    2.2.1 Statistical Treatment of Wind Waves .................................................................. 15
    2.2.2 Generation of Wind Waves ................................................................................. 19
  2.3 Swell .............................................................................................................................. 20
  2.4 The Governing Equation for Wind Waves and Swell .................................................... 21
  2.5 Tides and Tidal Currents ............................................................................................... 24
  2.6 Storm Surges ................................................................................................................ 27
  2.7 The Governing Equations for Tides and Storm Surges ................................................ 29
    2.7.1 The Depth-Integrated Equations ......................................................................... 32

CHAPTER 3 LITERATURE REVIEW .......................................................................................34
3.1 Coupling of Wave and Hydrodynamic Models ................................................................. 34
3.2 Coupling of Wave model and Atmospheric Models .......................................................... 37
3.3 Coupling of Hydrodynamic and Atmospheric Models ..................................................... 40
3.4 Ultimate Coupling Model and Discussion ....................................................................... 43

CHAPTER 4 MODEL DESCRIPTIONS ................................................................................. 45
4.1 ADCIRC-2DDI .............................................................................................................. 45
4.2 WAM and SWAN .......................................................................................................... 51
  4.2.1 Wave Radiation Stresses ......................................................................................... 54
4.3 Wind Field Model ......................................................................................................... 57
  4.3.1 Wind Stresses for ADCIRC-2DDI .......................................................................... 59
  4.3.2 Wind Stresses for WAM and SWAN ....................................................................... 60

CHAPTER 5 FINITE ELEMENT MESHES AND FINITE DIFFERENCE GRID
DEVELOPMENT ................................................................................................................... 64
5.1 St. Johns River Region ................................................................................................. 65
5.2 Finite Element Mesh Development .............................................................................. 67
  5.2.1 The Global-Scale ADCIRC Mesh (WNAT-SJR Mesh) ............................................. 67
  5.2.2 The Local-Scale ADCIRC Mesh (Pseudo-Operational Mesh) ................................. 74
5.3 Finite Difference Grid Development ............................................................................ 75
  5.3.1 The Global-Scale WAM Grid .................................................................................. 75
  5.3.2 The Local-Scale SWAN Grid .................................................................................. 76
5.4 Coupling Model Domain ............................................................................................. 77

CHAPTER 6 MODEL SETUP ............................................................................................... 79
7.2.3 Sensitivity Analysis ................................................................................................... 131
7.4 Quantitative Analysis ................................................................................................. 135
7.5 Creation of the Best Hydrograph .............................................................................. 138
CHAPTER 8 CONCLUSION AND FUTURE WORK ............................................................. 141
8.1 Conclusions ............................................................................................................... 141
8.2 Future Work .............................................................................................................. 145
APPENDIX A ADCIRC-2DDI INPUT FILE: MESH DESCRIPTION .................................. 146
APPENDIX B ADCIRC-2DDI INPUT FILE: MODEL PARAMETER ..................................... 148
APPENDIX C SWAN INPUT FILE: MODEL PARAMETER ................................................. 153
APPENDIX D NUMERICAL SIMULATION RESULTS: THE ADCIRC RESULTS .............. 155
APPENDIX E NUMERICAL SIMULATION RESULTS: THE UNI-COUPLING AND COUPLING RESULTS .............................................................................................................. 192
LIST OF REFERENCES ..................................................................................................... 207
LIST OF FIGURES

Figure 1.1: The WNAT model domain with boundary................................................................. 4
Figure 1.2: The St. Johns River (Sucsy and Morris 2002). ....................................................... 5
Figure 1.3: Hurricane Floyd track September 6 to 18, 1999 (NOAA). ................................. 7
Figure 1.4: Hurricane Floyd maximum wind speed (mph, blue line) and minimum pressure (mb, red line) September 8 to 17, 1999 (NOAA). ........................................................................ 7
Figure 2.1: Schematic distribution of wave energy in frequencies (Massel 1996).............. 12
Figure 2.2: Energy spectrum of waves (Bowden 1983). ........................................................ 17
Figure 2.3: Definition of a directional wave spectrum (Bowden 1983)................................. 18
Figure 2.4: Forces involved in the formation of a spring tide (PhysicalGeography.net)......... 25
Figure 2.5: Forces involved in the formation of a neap tide (PhysicalGeography.net)......... 26
Figure 3.1: A schematic of the storm tides (Graber et al. 2006).............................................. 36
Figure 3.2: A schematic of one- and two-way coupling of wave and hydrodynamic models..... 37
Figure 3.3: A schematic of coupling of wave and atmospheric models. ............................... 38
Figure 3.4: An image from the first ocean circulation/atmospheric coupling model (Manabe et al. 1975). ................................................................................................................................. 41
Figure 3.5: A schematic of coupling of wave and hydrodynamic models................................. 42
Figure 3.6: A schematic of coupling of wave, hydrodynamic, and atmospheric models......... 43
Figure 4.1: Hurricane Floyd wind field. .................................................................................. 58
Figure 5.1: St. Johns River region. .......................................................................................... 65
Figure 5.2: Lower St. Johns River and major drainage basins (Sucsy and Morris 2002). ....... 66
Figure 5.3: Finite element mesh for the WNAT-SJR model. ............................................................... 68
Figure 5.4: Bathymetry for the WNAT-SJR model........................................................................ 69
Figure 5.5: Finite element mesh and bathymetry for St. Johns River (maps and photos from
USGS)............................................................................................................................................ 70
Figure 5.6: Finite element mesh and bathymetry for the St. Johns River: inset α ...................... 71
Figure 5.7: Finite element mesh and bathymetry for the St. Johns River: inset β ...................... 72
Figure 5.8: Finite element mesh and bathymetry for the St. Johns River: inset γ ....................... 72
Figure 5.9: Finite element mesh and bathymetry for the St. Johns River: inset δ ...................... 73
Figure 5.10: Finite element mesh and bathymetry for the Pseudo-Operational model. ............. 74
Figure 5.11: Wave field of the WAM model and maximum significant wave height generated by
Hurricane Floyd (1999) .................................................................................................................. 75
Figure 5.12: Finite difference grid for the SWAN domain............................................................ 76
Figure 5.13: Bathymetry for the SWAN domain......................................................................... 77
Figure 5.14: Overlapped finite element mesh and finite difference grid and NOS tidal gauge
stations. ........................................................................................................................................... 78
Figure 6.1: NOAA\NOS tidal gauge locations for the Florida Atlantic Coast and the St. Johns
River.............................................................................................................................................. 83
Figure 7.1: Astronomical tide comparison at Mayport............................................................... 87
Figure 7.2: Astronomical tide comparison at I-295 Bridge West End. ......................................... 87
Figure 7.3: Astronomical tide comparison at Wekala. ............................................................... 88
Figure 7.4: a) USGS gauge and river inflow locations and b) a relationship between precipitation
[in] and average wind speed [mph] at Sanford. ............................................................................. 90
Figure 7.5: River level comparison at Mayport ................................................................. 91
Figure 7.6: River level comparison at I-295 Bridge West End ........................................ 91
Figure 7.7: River level comparison at Buffalo Bluff ......................................................... 92
Figure 7.8: a) The 2005 Atlantic storm tracks and timeline (Wikipedia) and b) precipitation [in] and average wind speed [mph] at Jacksonville during simulation period ............................... 95
Figure 7.9: River level comparison at Main Street Bridge ............................................... 96
Figure 7.10: Water level comparison (September 1 through 15, 2005) at Mayport ............ 97
Figure 7.11: Water level comparison (September 16 through 30, 2005) at Mayport .......... 97
Figure 7.12: Water level comparison (September 1 through 15, 2005) at I-295 Bridge .......... 98
Figure 7.13: Water level comparison (September 16 through 30, 2005) at I-295 Bridge .......... 98
Figure 7.14: Water level comparison (September 1 through 15, 2005) at Buffalo Bluff ........ 99
Figure 7.15: Water level comparison (September 16 through 30, 2005) at Buffalo Bluff ........ 99
Figure 7.16: Water level comparison (September 1 through 15, 2005) at Mayport .......... 102
Figure 7.17: Water level comparison (September 16 through 30, 2005) at Mayport .......... 102
Figure 7.18: Water level comparison (September 1 through 15, 2005) at I-295 Bridge .......... 103
Figure 7.19: Water level comparison (September 16 through 30, 2005) at I-295 Bridge .......... 103
Figure 7.20: Water level comparison (September 1 through 15, 2005) at Buffalo Bluff .......... 104
Figure 7.21: Water level comparison (September 16 through 30, 2005) at Buffalo Bluff .......... 104
Figure 7.22: Water level comparison based on the wind forcings at Fernandina Beach .......... 106
Figure 7.23: Water level comparison based on the wind forcings at Mayport ..................... 107
Figure 7.24: Water level comparison based on the wind forcings at St. Augustine Beach .......... 107
Figure 7.25: Water level comparison based on the wind forcings at Wekala ....................... 108
Figure 7.26: Water level comparison applying two domain sizes and hydrograph boundary conditions at Mayport. .............................................................................................................................. 109

Figure 7.27: Water level comparison with various bottom frictions at Mayport................. 111

Figure 7.28: Water level comparison with various bottom frictions at Fernandina Beach. ... 112

Figure 7.29: Water level comparison with several drag coefficients at Mayport............... 114

Figure 7.30: Water level comparison with several drag coefficients at Wekala. ................. 114

Figure 7.31: A diagram of uni-coupling SWAN and ADCIRC models. ................................. 118

Figure 7.32: Water level comparison in non- and uni-couplings at Fernandina Beach........ 119

Figure 7.33: Water level comparison in non- and uni-couplings at Mayport..................... 120

Figure 7.34: Water level comparison in non- and uni-couplings at St. Augustine Beach..... 120

Figure 7.35: Nested SWAN domain. .................................................................................. 122

Figure 7.36: Water level comparison using different boundary conditions at Mayport..... 122

Figure 7.37: Water level comparison applying the different modes in SWAN at Mayport. ... 124

Figure 7.38: The methodology of the coupling of SWAN and ADCIRC models.............. 127

Figure 7.39: Water level comparison among three models at Fernandina Beach............. 128

Figure 7.40: Water level comparison among three models at Mayport............................. 129

Figure 7.41: Water level comparison among three models at St. Augustine Beach............ 129

Figure 7.42: Water level comparison used several exchange times at Mayport............... 131

Figure 7.43: Water level comparison by applying the hydrograph BC at Mayport.......... 133

Figure 7.44: Maximum storm tide counters with the coupling model around Mayport.... 135

Figure 7.45: Water level comparison in three hydrographs at Fernandina Beach............. 138

Figure 7.46: Water level comparison in three hydrographs at Mayport......................... 139
Figure 7.47: Water level comparison in three hydrographs at St. Augustine Beach. ............... 139
Figure D.1.1: Simulation results (1 – 3, 1-4) at WWTD Mayport Naval Station.................. 157
Figure D.1.2: Simulation results (3, 4, 3-6) at WWTD Mayport Naval Station....................... 158
Figure D.1.3: Simulation results (3, 4, 7-8 - 8) at WWTD Mayport Naval Station. ............. 159
Figure D.2.1: Simulation results (1 – 3, 1-4) at Mayport. .................................................. 160
Figure D.2.2: Simulation results (3, 4, 3-6) at Mayport. .................................................... 161
Figure D.2.3: Simulation results (3, 4, 7-8 - 8) at Mayport. ............................................. 162
Figure D.3.1: Simulation results (1 – 3, 1-4) at Dame Point. ............................................. 163
Figure D.3.2: Simulation results (3, 4, 3-6) at Dame Point. ............................................. 164
Figure D.3.3: Simulation results (3, 4, 7-8 - 8) at Dame Point............................................ 165
Figure D.4.1: Simulation results (1 – 3, 1-4) at Longbranch (USE-DDP). ............................ 166
Figure D.4.2: Simulation results (3, 4, 3-6) at Longbranch (USE-DDP). ............................ 167
Figure D.4.3: Simulation results (3, 4, 7-8 - 8) at Longbranch (USE-DDP). ........................ 168
Figure D.5.1: Simulation results (1 – 3, 1-4) at Main Street Bridge....................................... 169
Figure D.5.2: Simulation results (3, 4, 3-6) at Main Street Bridge....................................... 170
Figure D.5.3: Simulation results (3, 4, 7-8 - 8) at Main Street Bridge.................................... 171
Figure D.6.1: Simulation results (1 – 4, 1-2) at I-295 Bridge, West End............................... 172
Figure D.6.2: Simulation results (3, 4, 3-6) at I-295 Bridge, West End............................... 173
Figure D.6.3: Simulation results (3, 4, 7-8 - 8) at I-295 Bridge, West End............................ 174
Figure D.7.1: Simulation results (1 – 4, 1-2) at Red Bay Point............................................. 175
Figure D.7.2: Simulation results (3, 4, 3-6) at Red Bay Point............................................. 176
Figure D.7.3: Simulation results (3, 4, 7-8 - 8) at Red Bay Point........................................ 177
Figure D.8.1: Simulation results (1 – 4, 1-2) at Racy Point........................................................ 178
Figure D.8.2: Simulation results (3, 4, 3-6) at Racy Point.......................................................... 179
Figure D.8.3: Simulation results (3, 4, 7-8 - 8) at Racy Point. ...................................................180
Figure D.9.1: Simulation results (1 – 4, 1-2) at Palakta. ............................................................ 181
Figure D.9.2: Simulation results (3, 4, 3-6) at Palakta. ............................................................. 182
Figure D.9.3: Simulation results (3, 4, 7-8 - 8) at Palakta. ..........................................................183
Figure D.10.1: Simulation results (1 – 4, 1-2) at Buffalo Bluff.................................................. 184
Figure D.10.2: Simulation results (3, 4, 3-6) at Buffalo Bluff.................................................... 185
Figure D10.3: Simulation results (3, 4, 7-8 - 8) at Buffalo Bluff. .............................................. 186
Figure D.11.1: Simulation results (1 – 4, 1-2) at Wekala........................................................... 187
Figure D.11.2: Simulation results (3, 4, 3-6) at Wekala............................................................. 188
Figure D11.3: Simulation results (3, 4, 7-8 - 8) at Wekala........................................................ 189
Figure D.12.1: Simulation results (1 – 8) at Fernandina Beach.................................................. 190
Figure D.13.1: Simulation results (1 – 8) at St. Augustine Beach.............................................. 191
Figure E.1: Simulation results (1 - 6) at WWTD Mayport Naval Station.................................194
Figure E.2: Simulation results (1 – 6) at Mayport. .....................................................................195
Figure E.3: Simulation results (1 – 6,) at Dame Point............................................................... 196
Figure E.4: Simulation results (1 – 6) at Longbranch (USE-DDP). ........................................... 197
Figure E.5: Simulation results (1 – 6) at Main Street Bridge. ..................................................... 198
Figure E.6: Simulation results (1 – 6) at I-295 Bridge, West End.............................................. 199
Figure E.7: Simulation results (1 – 6) at Red Bay Point............................................................ 200
Figure E.8: Simulation results (1 –6) at Racy Point. ................................................................. 201
Figure E.9: Simulation results (1 – 6) at Palakta................................................................. 202
Figure E.10: Simulation results (1 – 6) at Buffalo Bluff.......................................................... 203
Figure E.11: Simulation results (1 – 6) at Wekala................................................................. 204
Figure E.12: Simulation results (1 – 6) at Fernandina Beach.............................................. 205
Figure E.13: Simulation results (1 – 6) at St. Augustine Beach........................................... 206
LIST OF TABLES

Table 2.1: Waves, physical mechanisms, and periods (Massel 1996) .................................................. 13
Table 6.1: Tidal constituents used to force the ADCIRC model .............................................................. 80
Table 6.2: NOAA\NOS tidal gauge locations shown in Figure 6.1 ............................................................ 84
Table 7.1: Bottom friction values from various tidal studies ............................................................... 111
Table 7.2: Drag coefficient formulation from several studies ............................................................. 113
Table 7.3: Qualitative analysis comparison .......................................................................................... 137
LIST OF ABBREVIATIONS

ADCIRC-2DDI  Advanced Circulation Model for Oceanic, Coastal, and Estuarine Waters, Two-Dimensional Depth-Integrated

BG  British Gravitational

CPP  Carte Parallélogrammatique Projection

CHAMPS  Coastal Hydroscience Analysis, Modeling & Predictive Simulations Laboratory

CSI  Coastal Storm Initiative

ECMWF  European Centre for Medium-Range Weather Forecasts

EDT  Eastern Daylight Time

ERDC  Engineer Research and Development Center

FLDWAV  Dynamic, Generalized FLood WAVe Routing Model

GMT  Greenwich Mean Time

GWCE  Generalized Wave Continuity Equation

HEXOS  Humidity Exchange over the Sea

H*Wind  Hurricane Research Division Wind Analysis System

LTEA  Localized Truncation Error Analysis

MM5  Mesoscale Model, Fifth Generation

MSL  Mean Sea Level

NGDC  National Geophysical Data Center

NOAA  National Oceanic and Atmospheric Administration

NOS  National Ocean Service
<table>
<thead>
<tr>
<th>Acronym</th>
<th>Description</th>
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<tbody>
<tr>
<td>NOPP</td>
<td>National Oceanographic Partnership Program</td>
</tr>
<tr>
<td>NWS</td>
<td>National Weather Service</td>
</tr>
<tr>
<td>NWSRFS</td>
<td>National Weather Service River Forecast System</td>
</tr>
<tr>
<td>ODGP</td>
<td>Ocean Data Gathering Program</td>
</tr>
<tr>
<td>OHD</td>
<td>Office of Hydrologic Development</td>
</tr>
<tr>
<td>PBL</td>
<td>Planetary Boundary Layer</td>
</tr>
<tr>
<td>SERFC</td>
<td>SouthEast River Forecast Center</td>
</tr>
<tr>
<td>SI</td>
<td>System International</td>
</tr>
<tr>
<td>SJRWMD</td>
<td>St. Johns River Water Management District</td>
</tr>
<tr>
<td>SMS</td>
<td>Surface Water Modeling System</td>
</tr>
<tr>
<td>SST</td>
<td>Sea Surface Temperature</td>
</tr>
<tr>
<td>SWAN</td>
<td>Simulating WAves Nearshore</td>
</tr>
<tr>
<td>UCF</td>
<td>University of Central Florida</td>
</tr>
<tr>
<td>USACE</td>
<td>United States Army Corps of Engineers</td>
</tr>
<tr>
<td>USGS</td>
<td>United States Geological Survery</td>
</tr>
<tr>
<td>WAM</td>
<td>WAve Modeling</td>
</tr>
<tr>
<td>WCRP</td>
<td>World Climate Research Program of the World Meteorological Organization</td>
</tr>
<tr>
<td>WNAT</td>
<td>Western North Atlantic Tidal</td>
</tr>
<tr>
<td>WNAT-333K</td>
<td>Finite Element Mesh for WNAT which includes approximately 333,000 Computational Nodes</td>
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<tr>
<td>WANT-48K</td>
<td>Finite Element Mesh for WNAT which includes approximately 48,000 Computational Nodes</td>
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CHAPTER 1
INTRODUCTION

Hurricane storm tides are known to cause catastrophic damages to coastal communities, and therefore, it is necessary to better understand the cause of the storm tides in order to prepare for future events. The influences of individual factors that produce the storm tides are examined to better understand the underlying mechanisms. There are five basic mechanisms for the storm tide generation at or near the shoreline: 1) astronomical tides owing to the relative positions of the moon, sun and Earth; 2) inverted barometric effect (pressure surges); 3) wind-driven surges caused by strong onshore winds; 4) geostrophic tilt, a result of alongshore current; and 5) set-up from short waves (wind-induced waves) (Reid 1990). A storm surge is composed of only four (1–4) components while a storm tide consists of all the above (Graber et al. 2006).

Recent state-of-the-art models have an ability to operate at high spatial resolutions in the realm of hydrodynamic and wave modeling. This has led to a drastic increase in the accuracy of the computed physics and has offered the challenge to apply these models to appropriate areas in order to assess their performance. As a result, once neglected effects such as non-linear feedback between currents and waves are no longer ignored. Astronomical tides, surges caused by meteorological effects (wind and pressure) and wind-induced waves are components of the hurricane storm tides and can no longer be treated as independent entities. It must be considered as a combined mechanism.
Many hydrodynamic models are capable of simulating the storm surge (pressure surges, wind-driven surges, geostrophic tilt, and astronomical tides). Short wave set-up and run-up (wind-induced waves), however, are not described by these hydrodynamic models because of their inability to calculate wind-induced wave set-up and run-up (Panchang et al. 1999). In spite of this, many models can incorporate output information from a wave model (in form of the wave radiation stresses). Therefore, a coupling between the two models is possible. It is necessary to develop a practical application of coupling different models.

It is submitted herein that a coupling of hydrodynamic and wave models can be facilitated to understand the complex mechanisms of the storm tides. The hydrodynamic model employed for calculating tides and surges is ADCIRC-2DDI (ADvanced CIRCulation Model for Shelves, Coasts and Estuaries, Two-Dimensional Depth Integrated) developed by Luettich et al. (1992) (See Chapter 4.1). The finite element based model solves the fully nonlinear shallow water equations in the generalized wave continuity form. The wave model applied for wind-induced wave simulation is the third-generation SWAN (Simulating WAves Nearshore), applicable to the estimation of wave parameters in coastal areas and estuaries. The SWAN model is governed by the wave action balance equation driven by wind, sea surface elevations and current conditions (Holthuijsen et al. 2004) (See Chapter 4.2). The coupling models developed in this research are capable of describing not only the wave-current interaction but also non-linear relationships among tides, wind surges and wind-induced waves.
Hurricane Floyd is chosen as the subject storm for this dissertation, since Hurricane Floyd passed along the Florida Atlantic Coast without making a direct landfall and presents an ideal case to study the influence of wind-induced set-up and run-up on the storm tides. In fact, Hurricane Floyd had a direct impact on the St. Johns River region and caused severe damage. Therefore the St. Johns River provides a focal point for the following research and is modeled along with the Western North Atlantic Tidal model domain. The following begins with an overview of the Western North Atlantic Tidal (WNAT) model domain, the St. Johns River, and Hurricane Floyd. This introductory chapter concludes with the research objectives.
1.1 The Western North Atlantic Tidal (WNAT) Model Domain

The Western North Atlantic Tidal (WNAT) model domain encompasses the Gulf of Mexico, the Caribbean Sea, and the entire portion of the North Atlantic Ocean found west of the 60ºW meridian (Figure 1.1). The open-ocean boundary lying along the 60ºW meridian extends from the area of Glace Bay, Nova Scotia, Canada to the vicinity of Corocora Island in eastern Venezuela and is situated almost entirely in the deep ocean. This large scale computational domain covers an area of approximately 8.4 million km².
1.2 The St. Johns River

The St. Johns River, with a length of 500 km, is the longest river contained wholly within the state of Florida. The St. Johns River drainage basin encompasses over 22,000 km$^2$ within portions of 16 counties (Figure 1.2). The St. Johns River is a slow-moving river with low slope. The river drops, on average, 2.2 cm per kilometer (Toth 1993). The low slope of the river allows tidal effects to extend at least 170 km from the river mouth in Duval County to Lake George in Volusia County (Sucsy and Morris 2002).
1.3 Hurricane Floyd

Hurricane Floyd impacted the East Coast of the United States from September 14 to 17, 1999. Torrential rains fell from the Carolinas to New England, resulting in major river and urban flooding. There were 56 deaths in the United States directly attributed to Floyd, and flood damage estimates ranged from $4.5 billion to more than $6 billion (in 2000). The hurricane path resulted in the evacuation of nearly 3 million people from the coastal areas of Florida, Georgia, and the Carolinas (NOAA 1999).

Floyd’s origin can be traced to a tropical wave that emerged from western Africa on September 2, 1999. Tropical Depression Eight, Hurricane Floyd, formed September 7 about 1100 miles (1600 km) east of the Lesser Antilles (Figure 1.3). Floyd became a hurricane at 8 am Eastern Daylight Time (EDT) on September 10. Floyd came within 110 miles (177 km) of Cape Canaveral as it paralleled the Florida coast on September 15. Floyd then moved slightly north-east and increased in forward speed, coming ashore near Cape Fear, North Carolina, at 2:30 am on September 16. At the time of landfall, Floyd was a Category 2 hurricane on the Saffir–Simpson Hurricane Scale with maximum winds of 104 mph (167 km/s). Sustained tropical storm force winds and gusts close to hurricane strength were recorded at many locations ranging from the Florida Keys to New York. Sustained winds of 96 mph (154 km/s) with gusts to 122 mph (196 km/s) were measured near Topsail Beach, North Carolina (See Figure 1.4) (NOAA 1999).
Figure 1.3: Hurricane Floyd track September 6 to 18, 1999 (NOAA).

Figure 1.4: Hurricane Floyd maximum wind speed (mph, blue line) and minimum pressure (mb, red line) September 8 to 17, 1999 (NOAA).
1.4 Research Objective

Forecasting river levels in the Southeastern United States is the responsibility of the National Weather Service (NWS) Southeast River Forecast Center (SERFC). Existing river forecasts in the St. Johns River are generated using hydrologic models defined within the National Weather Service River Forecast System (NWSRFS) in conjunction with a hydraulic, one-dimensional, dynamic, generalized flood wave routing model (FLDWAV) (Garza et al. 2005). When the National Oceanic and Atmospheric Administration (NOAA) started the Coastal Storm Initiative (CSI), the St. Johns River was selected for a demonstration project to showcase improved forecasting capabilities in coastal river systems. The NWS Office of Hydrologic Development (OHD), SERFC, University of Central Florida (UCF) and the Coastal Hydroscience Analysis, Modeling & Predictive Simulation Laboratory (CHAMPS Lab) have collaborated toward the development of the St. Johns River model envisioned by CSI. The combined effort involved expansion of the existing one-dimensional model to the two-dimensional model to predict flows, tides (astronomical and meteorological) and waves. In the future, real-time storm tide simulation will be operated to generate the flood forecast map.

Three issues arise when developing the St. Johns River model. First, the domain size of the coastal model needs to be determined. Previous research has shown that a large-scale domain is ideal for simulating hurricane storm surge (Blain et al. 1994a). However, a large-scale computational domain may include upwards of 100,000 nodal points, which can be computationally intensive for an operational model system. Furthermore, a shelf-based model
has proven to be adequate in reproducing hurricane flow conditions (Dietsche et al. In Press). This study compares two results of different finite element meshes by performing the Hurricane Floyd storm surge simulation along the Florida Atlantic Coast. Model results are compared to historical water level data at the National Ocean Service (NOS) tide gauge stations.

Second, the effects of meteorological forcings (wind and pressure) on the river levels need to be investigated. The St. Johns River is such a flat river that the meteorological effects could be significant to forecast the river levels when tropical cyclones and hurricanes pass near or over the river. Therefore, it is necessary to elucidate the effect of the meteorological forcings on the river. To accomplish this, a numerical simulation is performed by employing two periods, short- and long-term simulations. A short period is a length of a day or week, while a long period is a length of a month or more.

Third, the effect of the wind-induced wave set-up and run-up on the overall storm tides is a topical and important subject. The wind-induced waves could be left out of numerical models for computational efficiency, when it is not a significant player. On the other hand, for conditions when the wind-induced waves are significant, it is of interest to know how the wind-induced waves behave, i.e. if they linearly increase or if there are more complicated relationships. Therefore, it is necessary to explore the influences of the wind-induced waves on the storm tides through the coupling of hydrodynamic and wave models.
Chapter 2 elucidates the wave mechanisms and dynamics associated with this study. In Chapter 3, the literature review, an overview is presented of previous coupling models, including research discussion. The numerical codes used in this study (hydrodynamic and wave models) are presented in Chapter 4, and the model domains and study area are presented in Chapter 5. Chapter 6 presents the simulation parameters used in this study. The results are presented in Chapter 7, with a sensitivity analysis and quantitative analysis. Finally, the conclusions and future work are discussed in Chapter 8.
This chapter discusses the fundamental concepts of ocean wave mechanisms and dynamics. It provides a brief description of the basic types of ocean waves. It also provides a description of the wind waves, swell, tides and tidal currents, and storm surges associated with this research. In order to render the subject amenable to exact mathematical treatment, we adopt some simplifying assumptions on the medium and its motion, i.e. water is an inviscid and incompressible fluid, the density differences are sufficiently small so that the Boussinesq approximation may be applied and motion is irrotational.

2.1 The Basic Types of Ocean Waves

In general, five basic types of ocean waves can be distinguished in the oceanography: sound, gravity, internal, capillary, and planetary waves. Sound waves are due to water compressibility and are very small. Gravity forces, acting on water particles displaced from equilibrium at the ocean surface or at an internal geopotential surface in a stratified fluid, induce gravity and internal waves. At the contact surface between air and water, the combination of the turbulent wind and surface tension gives rise to short, high frequency capillary waves. On the other hand, very slow, large-scale planetary (or Rossby) waves are induced by the variation of the equilibrium potential vorticity, due to change in depth or latitude. All of the above wave types can occur together, producing complicated patterns of oscillations (Massel 1996).
The frequency range associated with external forces is very wide and the corresponding ocean surface responses occupy an extraordinarily broad range of wave lengths and periods, from capillary waves, with period of less than a second, through wind-induced waves and swell, with periods on the order of a few seconds, to tidal oscillations, with periods on the order of several hours and days. In Figure 2.1 and in Table 2.1, the schematic representation of energy contained in surface waves, and the physical mechanisms generating these waves, are listed. Figure 2.1 gives some impression of the relative importance of the various kinds of surface oscillations, but does not necessarily reflect the actual energy content (Massel 1996).

![Figure 2.1: Schematic distribution of wave energy in frequencies (Massel 1996).](image-url)
Table 2.1: Waves, physical mechanisms, and periods (Massel 1996).

<table>
<thead>
<tr>
<th>Wave type</th>
<th>Wave name</th>
<th>Physical mechanism</th>
<th>Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capillary waves</td>
<td>Capillary waves</td>
<td>Surface tension</td>
<td>$&lt; 10^{-1}$ s</td>
</tr>
<tr>
<td>Wind waves &amp; Swell</td>
<td>Wind waves</td>
<td>Wind shear, gravity</td>
<td>$&lt; 15$ s</td>
</tr>
<tr>
<td></td>
<td>Swell</td>
<td>Wind waves</td>
<td>$&lt; 30$ s</td>
</tr>
<tr>
<td>Infra-gravity waves</td>
<td>Surf beat</td>
<td>Wave groups</td>
<td>1 - 5 min</td>
</tr>
<tr>
<td></td>
<td>Seiche</td>
<td>Wind variation</td>
<td>2 - 40 min</td>
</tr>
<tr>
<td></td>
<td>Harbor resonance</td>
<td>Surf beat</td>
<td>2 - 40 min</td>
</tr>
<tr>
<td>Long period waves</td>
<td>Tsunami</td>
<td>Earthquake</td>
<td>10 min - 2 h</td>
</tr>
<tr>
<td></td>
<td>Storm surge</td>
<td>Wind stresses and atmospheric pressure variation</td>
<td>1 - 3 days</td>
</tr>
<tr>
<td></td>
<td>Tides</td>
<td>Gravitational action of the moon and sun, Earth rotation</td>
<td>12 - 24 h</td>
</tr>
</tbody>
</table>

In Table 2.1, bold-faced wave names are associated with the generation of the storm tides. The following chapters describe detailed information related to wind waves including statistical treatment and generation of wind waves, swell, tides, and storm surge related to this research. Furthermore, the governing equations for these waves are expressed.
2.2 Wind Waves

The occurrence of waves on the surface of the sea and their association with winds blowing over it are features which are familiar to everyone. The practical importance of wind waves in all aspects of sea travel, offshore engineering activities and the maintenance of coastal defenses is well recognized. Although it is obvious that wind stress is the primary cause of wind waves, the actual generation process has only recently received a satisfactory physical explanation. The main characteristics of wind waves may be summarized as follows: 1) they are of relatively short period, mostly within the range 1 to 30 seconds; 2) in deep water, their influence is restricted to a comparatively shallow layer, unlike tidal waves which extend throughout the whole depth; and 3) the water movements associated with them are of similar magnitude in the vertical and horizontal directions. This is in contrast to tides or wind-driven currents in which the vertical movement is small compared with the horizontal flow.

The classical theory of wind waves, with a history going back more than a hundred and fifty years, deals with trains of waves of uniform amplitude, wavelength and period, traveling in a fixed direction. A casual observation of actual sea waves, however, shows that the sea surface is very irregular, with waves of different heights and periods, changing in character as they move. In order to bring some degree of order out of chaos, it is necessary to treat an actual wave field statistically. The statistical approach is a recent scheme for wave forecasting, including the definition of significant waves, energy spectra and directional spectra and the estimation of extreme conditions, and will be considered next (Bowden 1983).
2.2.1 Statistical Treatment of Wind Waves

A record of waves passing a fixed point would normally have an irregular appearance. Groups of high waves alternate with intervals of lower waves, and it becomes apparent that wave trains of a number of different periods have been superimposed. A useful way of describing such a record would be to consider the highest one third of all the waves as being the ‘significant waves’ and to take the average height and period of these waves as the ‘significant wave height’ and ‘significant wave period’ respectively. These procedures gives more weight to the higher waves, which are of greater importance in relation to their effect on forecasting waves, but retain a certain amount of averaging and avoids extreme values.

The first stage in representing a wave field is to formulate the wave spectrum, which takes into account the superposition of many wave trains of different wavelength and period. Considering only waves traveling in the $x$ direction, the resultant elevation of the sea surface may be written

$$\xi_0(x,t) = \sum_{n=1}^{\infty} a_n \cos(k_n x - \sigma_n t + \epsilon_n)$$

where $a_n$ is the amplitude of the $n$th component which has a wavenumber $k_n$ and angular frequency $\sigma_n$. $\epsilon_n$ is a phase lag which varies randomly within the range 0 and $2\pi$ radians from one component to another and the summation is carried out over all of the components present.
It may be shown that, for waves of small amplitude, the total energy of any number of superposed wave trains is given by the sum of their individual energies. This enables an energy spectrum to be defined. Let \( E(\sigma)d\sigma \) be defined as the energy per unit area of all waves trains with angular frequencies between \( \sigma \) and \( \sigma + d\sigma \). Then

\[
E(\sigma)d\sigma = \frac{1}{2}g\rho \sum_\sigma a_n^2 \hspace{1cm} \text{...................................................................................................(2.2)}
\]

where the summation of \( a_n^2 \) is carried out for all components with angular frequency between \( \sigma \) and \( \sigma + d\sigma \). \( E(\sigma) \) is the spectral density, which may be plotted as a function of \( \sigma \), as in Figure 2.2. \( E(\sigma)d\sigma \) is represented by the area between the ordinate of \( E(\sigma) \) at \( \sigma \) and \( \sigma + d\sigma \). The total energy of the wave field is obtained by summation over the whole spectrum. Thus

\[
E = \int_0^\infty E(\sigma)d\sigma \hspace{1cm} \text{...................................................................................................(2.3)}
\]

From equations (2.2) and (2.3) it follows that \( E = (1/2)g\rho \sum_\sigma a_n^2 \) showing that the total energy per unit area of all the wave trains present is proportional to the sum of the squares of their amplitudes.
The energy spectrum has been defined above with the angular frequency $\sigma$ as the independent variable. It is possible to define the desired spectrum in terms of the frequency $f$, period $T$, wavenumber $k$ or wavelength $\lambda$, since all these quantities are related to $\sigma$.

The energy spectrum represents the distribution of energy among the waves of different frequencies but it does not account for the direction of travel of the waves. A more complete description of the wave field specifies the direction of propagation of the various trains of waves as well as their frequency. Referring to Figure 2.3, let $E(\sigma, \theta) d\sigma d\theta$ be the energy per unit area of waves with angular frequencies between $\sigma$ and $\sigma + d\sigma$ traveling in directions between $\theta$ and $\theta + d\theta$, where $\theta$ is measured from a fixed direction. The distribution of $E(\sigma, \theta)$ may be represented by contours drawn on a diagram in which $\sigma$ is denoted by the radial distance from the original and $\theta$ is drawn in the appropriate direction.
By integrating the value of $E(\sigma, \theta)$ over all value of $\theta$ for a given $\sigma$, the spectral density $E(\sigma)$ for the energy spectrum, as described above, is obtained. Thus

$$E(\sigma) = \int_0^{2\pi} E(\sigma, \theta) d\theta$$

(2.4)

One definition of wave distribution is that of the ‘zero up-cross’ wave proposed by Longuet-Higgins (1952) and Cartwright (1958). The concept of the zero up-cross waves is an objective way of focusing attention on the larger waves and it has been found to be amenable to statistical treatment. Let $N$ be the total number of zero up-cross waves in the record of duration $T_L$, and then the root mean square wave height $H_{rms}$ is defined by
\[ H_{\text{rms}}^2 = \frac{1}{N} \sum_{n=1}^{N} H_n^2 \] ...........................................(2.5)

where \( H_n \) is the height of the \( n \)th wave. In order to determine the significant wave height, the heights \( H_n \) are arranged in decreasing order of magnitude from \( H_1 \) to \( H_N \). Then the significant waves are those from \( H_1 \) to \( H_{N/3} \), taking \( N/3 \) to the nearest integer, and the significant wave height \( H_s \) is given by

\[ H_s = \frac{3}{N} (H_1 + H_2 + \cdots + H_{N/3}) \] ...........................................(2.6)

The statistical properties of the distribution of zero up-cross waves are related to the energy spectrum in a way described by Cartwright (1962), based on original papers by Longuet-Higgins (1952) and by Cartwright (1958).

2.2.2 Generation of Wind Waves

There are three aspects of the problem of the generation of wind waves by wind:

(1) Why does the surface of a body of water become wavy when a wind blows across it?
(2) How is energy transferred from the wind to the waves, so that they grow and develop the characteristic features of a rough sea?
(3) How many occurrences of waves, their heights, periods and spectrum, must be forecast for practical purposes?

The first theory of wave generation is physically based and defined by Jeffreys in 1924; however, it did not consider the formation of waves from an initially flat surface, rather it assumed that wave-like perturbation of small amplitude wave present and consider how they could grow by extracting energy from a steady wind blowing over them. Two papers were published in 1957 which form the basis of present-day theory of the generation waves. The first paper, by Phillips (1957), described a resonance mechanism which postulated the presence of a random distribution of pressure fluctuations. The second paper, by Miles (1957), described an instability mechanism which had some similarity with the Jeffreys theory. The two processes were combined in a mechanism described by Miles (1960). When a wind starts to blow over a calm sea, the resonance mechanism would come into action first, producing an initial rate of growth of wave energy linearly with time. As the wave become larger, the instability mechanism becomes more effective and the wave energy increases exponentially (Bowden 1983).

2.3 Swell

Within the area of a storm, waves of many different wavelengths, traveling at varying angles to the wind direction are present. Once generated, waves of each component wavelength will continue to travel at their own velocity. The waves will travel beyond the storm area into previously undisturbed water, with the energy being propagated at the appropriate group velocity.
The waves of longer period, and thus of longer wavelength, will travel faster and arrive at a distant coast before shorter waves from the same storm area. Pioneer work on the propagation of swell was carried out by Barber and Ursell (1948), using the spectra of waves recorded on the coast of Cornwall, Southwest England, and originating from storms in the North Atlantic. Waves at the long period end of the spectrum arrived first, followed some hours later by the shorter period, but higher energy, waves at the peak of the generated spectrum.

Several changes take place in the properties of the waves as they travel away from the storm area. In the first place, the angular spread of their directions of travel is reduced. The spread is related to the angle subtended at a point by the dimensions of the generating area and this angle is reduced when the storm generating area is at a greater distance. The energy density of the waves is decreased by geometrical spreading as they move further from the source. For waves from a point source, the energy per unit length of wave form would be inversely proportional to the distance traveled, independent of the wavelength. Dissipative processes, of which wave breaking is probably the most important, although air resistance and turbulent friction may play some part, affect the waves of shorter wavelength more severely than the longer ones.

2.4 The Governing Equation for Wind Waves and Swell

The governing equation for the growth of wind waves may be represented by the radiative transfer equation. Taking into account the random character of wave motion, it seems that the most appropriate method for the evaluation of waves, in time and space are the spectral methods.
Resulting functions of these methods (i.e., the action density spectrum \( N(k,x,t) \)) provide the distribution of wave energy in the frequency or wave number space. In order to develop an action balance equation we assume for a moment that the dispersion relation does not depend on time, but is rather a slowly changing function in space.

\[
\omega_n(x,t) = \Omega(k_n) \tag{2.7}
\]

If the medium itself is moving with velocity \( U \), the frequency of waves passing a field point is:

\[
\omega = \Omega(k) = \sigma(k) + k \cdot U \tag{2.8}
\]

Usually the quantity \( \omega \) is called the observed or apparent frequency, while \( \sigma \) is the intrinsic angular frequency. Willebrand (1975) noted that the conservation of wave action holds for every wave component separately:

\[
\frac{\partial}{\partial t} N_n + \nabla_x \cdot [(\nabla_x \omega_n) N_n] = 0 \tag{2.9}
\]

where \( N_n = 2|a_n|^2 / \sigma_n \), where \( a_n \) is amplitude of the \( n \)th component.

The action density is a function of time and this leads to an extra term involving \( \nabla_x \cdot \Omega \) in the equation for \( N(k,x,t) \). Thus equation (2.9) becomes
\[
\frac{\partial}{\partial t} N(k,x,t) + (\nabla_x \Omega) \cdot \nabla_x N(k,x,t) - (\nabla_x \Omega) \cdot \nabla_k N(k,x,t) = 0 \tag{2.10}
\]

Explicitly, in terms of the energy density spectrum \( N(k,x,t) = F(k,x,t)/\sigma \), one has

\[
\left[ \frac{\partial}{\partial t} + (\nabla_x \Omega) \cdot \nabla_x - (\nabla_x \Omega) \cdot \nabla_k \right] \left( \frac{F(k,x,t)}{\sigma} \right) = 0 \tag{2.11}
\]

Another useful form for this equation is obtained by using the trivial identity

\[
\nabla_x \cdot (\nabla_x \Omega) - \nabla_k \cdot (\nabla_x \Omega) = 0 \tag{2.12}
\]

This leads to the ‘flux form’

\[
\frac{\partial}{\partial t} \left( \frac{F}{\sigma} \right) + \nabla_x \cdot \left[ c_g + U \left( \frac{F}{\sigma} \right) \right] - \nabla_k \cdot \left[ \nabla_x \Omega \left( \frac{F}{\sigma} \right) \right] = 0 \tag{2.13}
\]

in which \( c_g = \nabla_x \sigma(k) \) is a group velocity.

If the wave field is subjected to processes of generation, dissipation, nonlinear interaction between spectral components and other possible interactions with atmospheric boundary layer and various ocean movements, equation (2.13) should be supplemented by a source – sink term at the right-hand side,
\[
\left\{ \frac{\partial}{\partial t} + (c_g + U) \cdot \frac{\partial}{\partial x} \nabla_x - \nabla_x \Omega \cdot \frac{\partial}{\partial k} \right\} \left( \frac{F}{\sigma} \right) = \frac{S}{\sigma}
\]

(2.14)

In equation (2.14), the first term in the left-hand side expresses the local evolution of the spectrum in time, while the second term represents the evolution of the spectrum for the horizontally non-homogeneous wave field. This term shows that energy is transported at the group velocity. The third term reflects the effects of refraction and shoaling due to a non-horizontal bottom or due to currents. The right-hand sides consist of source and sink terms. The WAM and SWAN models solve equation (2.14) with the numerical techniques described in Chapter 4.2

### 2.5 Tides and Tidal Currents

An ocean tide refers to the cyclic rise and fall of seawater. Tides are predominantly caused by slight variations in the gravitational attractions between the Earth and the moon and the sun. Tides are periodic primarily because of the cyclical influence of the Earth's rotation. The moon is the primary factor controlling the temporal rhythm and height of the tides. The moon produces two tidal bulges somewhere on the Earth through the effects of gravitational attraction. The height of these tidal bulges is controlled by the moon's gravitational force and the Earth's gravity pulling the water back toward the Earth. At the location on the Earth closest to the moon, seawater is drawn toward the moon because of the greater strength of gravitational attraction. On the opposite side of the Earth, another tidal bulge is produced away from the moon. However,
this bulge is due to the fact that at this point on the Earth, the force of the moon's gravity is at its weakest (Pidwirny 2006).

The timing of tidal events is related to the Earth's rotation and the revolution of the moon around the Earth. If the moon was stationary in space, the tidal cycle would be 24 hours long. However, the moon revolves around the Earth. One revolution of the moon around the Earth takes about 27 days and adds about 50 minutes to the tidal cycle. As a result, the tidal period is 24 hours and 50 minutes in length. The second main factor controlling tides on the Earth's surface is the sun's gravity. The height of the average solar tide is about 50% the average lunar tide. At certain times during the moon's revolution around the Earth, the direction of its gravitational attraction is aligned with the sun's (Figure 2.4). During these times the two tide producing bodies act together to create the highest and lowest tides. These spring tides occur every 14-15 days during full and new moons.

Figure 2.4: Forces involved in the formation of a spring tide (PhysicalGeography.net).
When the gravitational pull of the moon and sun are at right angles to each other, the daily tidal variations on the Earth are at their least (Figure 2.5). These events are called neap tides and they occur during the first and last quarter of the moon.

![Figure 2.5: Forces involved in the formation of a neap tide (PhysicalGeography.net).](image)

Although the rise and fall of the water level is the most obvious effect, the primary tidal phenomenon consists of the induced horizontal current; so, the sea level variations at the coast are a consequence of the divergence and convergence of seawater, occurring when tidal currents flow away from or toward the shore. The current associated with a rising water level is termed the flood and that with a falling level, the ebb current. The effects of tidal currents are twofold: on one hand, they may cause large daily changes in the volume of water in a bay, and on the other hand they may promote vertical mixing, thus breaking down the stratification of the water column.
In other open waters of the continental shelf, or in shallow open seas, tidal currents are characterized by a changing speed, often never decreasing to zero, and rotating direction, usually with a dominating semi-diurnal period. In narrow waterways, such as estuaries, the common tidal pattern is composed of a flood current in one direction as the tide rises, and ebb current in the opposite direction while it falls. Typical values of tidal current speeds are given in Pond and Pickard (1989) as less than about 0.1 m/s away from the coast, but these authors point out that much higher values are common in straits and passages, as in the Seymour Narrows (British Columbia, Canada), where tidal currents of up to 8 m/s have been measured.

### 2.6 Storm Surges

A storm surge is defined as a disturbance of sea level, relative to that due to tides alone, produced by meteorological causes (Bowen 1983). The height of surges is given by:

\[
\text{Surge height} = \text{recorded level} - \text{tidally predicted level}
\]

A surge may be either positive or negative, i.e. the actual sea level may be either higher or lower than that expected from tidal predictions. The time scale of a storm surges may range from a few hours to several days. A surge of several days duration could be identified by subjecting the sea level data to a low-pass numerical filter which would eliminate oscillations of frequencies within the diurnal, semidiurnal and higher harmonic tidal bands. This procedure, however, would eliminate surge components on a time scale of less than a day, including changes in the
amplitude and phase of the tidal constituents which can arise from interactions between surge and tide. The alternative is to subtract the predicted tide from the recorded levels directly (Bowen 1983).

The wind blowing over the sea exerts an effective tangential stress on the surface. If the processes acting on the wave-covered surface are considered in detail, it is probable that much of the stress is contributed by normal pressures on the deformed sea surface. From the point of view of the storm surges, however, we consider the shearing stress in the air above the seas to be communicated to the layer of water below the surface, without considering in detail what happens at the surface itself. A proportion of the wind stress is used directly in generating the surface waves and some of the wave momentum is probably passed on to the drift current by breaking waves. The corresponding stress will be included in the effective tangential stress due to the wind. The wave field also reacts on the stress by determining the effective roughness of the sea surface. These various effects are assumed to have been taken into account in defining the effective tangential stress of the wind on the sea surface, denoted by $\tau_s$.

In specifying the stress $\tau_s$, it is usually assumed that it acts in the direction of the wind relative to the sea surface and that its magnitude is proportional to the square of the wind speed relative to the sea surface. Thus

$$\tau_s = C_{D} \rho_a W |W|$$  \hfill (2.15)
where $W$ is the wind speed measured at a given height, usually taken as 10 m, above the sea surface, $\rho_a$ is the density of the air and $C_D$ is a drag coefficient. The value of $C_D$ depends on (a) the height at which $W$ is measured, (b) the stability of the lowest few meters of the atmosphere and (c) the roughness of the sea surface, as affected by waves. The value of $C_D$ also depends on $W$ itself.

### 2.7 The Governing Equations for Tides and Storm Surges

The equations governing the tides and storm surges comprise two equations: one is the equations of motion and the other is the continuity equation. Over a limited region of the Earth, for which the curvature of the Earth’s surface may be neglected, right-handed rectangular axes will be taken with the origin in the mean sea surface, the $x$ and $y$ axes horizontal and the $z$ axis vertically upward. The velocity components parallel to the $x$, $y$ and $z$ axes, a point $x$, $y$, $z$, will be denoted by $u$, $v$ and $w$. The pressure is denoted by $p$, the density of the water by $\rho$ and $F_x$, $F_y$ denote the components of force per unit mass (other than the pressure force) in the $x$ and $y$ directions. It is assumed that the only significant force in the $z$ direction is that due to apparent gravity, $g$, which includes the centrifugal force due to the rotation of the Earth, and that vertical accelerations are negligible. Then the equations of motion in the $x$, $y$ and $z$ directions respectively are
\[
\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} + w \frac{\partial u}{\partial z} - f v = -\frac{1}{\rho} \frac{\partial p}{\partial x} + F_x \tag{2.16}
\]

\[
\frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} + w \frac{\partial v}{\partial z} + f u = -\frac{1}{\rho} \frac{\partial p}{\partial y} + F_y \tag{2.17}
\]

\[
0 = -\frac{1}{\rho} \frac{\partial p}{\partial z} - g \tag{2.18}
\]

where \( f = 2\omega \sin \phi \) is the Coriolis parameter, \( \omega \) is the angular rate of rotation of the Earth \((= 7.29 \times 10^{-5} \text{ radians per second})\) and \( \phi \) is the latitudinal position, positive to the north of the equator.

To these equations must be added the equation of continuity of volume

\[
\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \tag{2.19}
\]

Integrating equation (2.18) with respect to \( z \), assuming \( \rho \) to be independent of \( z \),

\[
p = p_a + g \rho (\zeta - z),
\]

where \( p_a \) is the atmospheric pressure and \( \zeta \) is the elevation of the sea surface above its undisturbed value, taken as the zero for the \( x \), \( y \) plane. If \( p \) is independent of \( x \) and \( y \) as well as of \( z \),

\[
\frac{1}{\rho} \frac{\partial p}{\partial x} = \frac{1}{\rho} \frac{\partial p_a}{\partial x} + g \frac{\partial \zeta}{\partial x}, \quad \frac{1}{\rho} \frac{\partial p}{\partial y} = \frac{1}{\rho} \frac{\partial p_a}{\partial y} + g \frac{\partial \zeta}{\partial y} \tag{2.20}
\]
In the case of tidal motions and storm surge, the components of horizontal force, $F_x$ and $F_y$ of equations 2.16 and 2.17, will include the tide-generating forces and also frictional stresses in the water, where they are significant. For the tide-generating forces

$$F_x = g \frac{\partial \eta}{\partial x}, \quad F_y = g \frac{\partial \eta}{\partial y} \tag{2.21}$$

where $\eta$ is the elevation in the equilibrium tide.

Frictional effects arise from the shearing stress of wind acting on the sea surface or the shearing stress at the bottom caused by the flow of water over the sea bed. These stresses are communicated to the rest of the water column by internal shearing stresses due to turbulence. The direct effect of molecular viscosity is usually small. In most cases only shearing stresses acting across horizontal planes need be considered. It may easily be shown that the additional force per unit mass acting on an element of water has components

$$\frac{1}{\rho} \frac{\partial \tau_x}{\partial z}, \quad \frac{1}{\rho} \frac{\partial \tau_y}{\partial z} \tag{2.22}$$

where $\tau_x$ and $\tau_y$ represent the stress tensors in the $x$ and $y$ directions, respectively.

From equations (2.16) - (2.22), the complete momentum equations, valid at any point $x$, $y$, $z$ in the water are
\[
\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} + w \frac{\partial u}{\partial z} - f v = -\frac{\partial}{\partial x} \left[ \frac{p_a}{\rho} + g(\zeta - \eta) \right] + \frac{1}{\rho} \frac{\partial \tau_x}{\partial z} \quad \text{(2.23)}
\]

\[
\frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} + w \frac{\partial v}{\partial z} + f u = -\frac{\partial}{\partial y} \left[ \frac{p_a}{\rho} + g(\zeta - \eta) \right] + \frac{1}{\rho} \frac{\partial \tau_y}{\partial z} \quad \text{(2.24)}
\]

The hydrostatic equation (2.18) and continuity equation (2.19) are unchanged.

### 2.7.1 The Depth-Integrated Equations

Let \( h \) be the depth of water below the undisturbed level, \( z = 0 \). Then by integrating equations (2.23) and (2.24) from the bottom, \( z = -h \), to the surface, \( z = \zeta \), putting

\[
\bar{u} = \frac{1}{h + \zeta} \int_{-h}^{\zeta} u dz, \quad \bar{v} = \frac{1}{h + \zeta} \int_{-h}^{\zeta} v dz,
\]

so that \( \bar{u} \) and \( \bar{v} \) are components of the depth-integrated velocities, and making certain assumptions, the following equations may be derived:

\[
\frac{\partial \bar{u}}{\partial t} + \bar{u} \frac{\partial \bar{u}}{\partial x} + \bar{v} \frac{\partial \bar{u}}{\partial y} - f \bar{v} = -\frac{\partial}{\partial x} \left[ \frac{p_a}{\rho} + g(\zeta - \eta) \right] + \frac{\tau_{xx} - \tau_{hn}}{\rho(h + \zeta)} \quad \text{(2.25)}
\]

\[
\frac{\partial \bar{v}}{\partial t} + \bar{u} \frac{\partial \bar{v}}{\partial x} + \bar{v} \frac{\partial \bar{v}}{\partial y} + f \bar{u} = -\frac{\partial}{\partial y} \left[ \frac{p_a}{\rho} + g(\zeta - \eta) \right] + \frac{\tau_{xy} - \tau_{hv}}{\rho(h + \zeta)} \quad \text{(2.26)}
\]
In these equations $\tau_{sx}$, $\tau_{sy}$ are the components of the stress on the surface and $\tau_{bx}$, $\tau_{by}$ are the components of the stress at the bottom. The wind stresses $\tau_{sx}$, $\tau_{sy}$ is derived from equation (2.15): $\tau_{sx} = |\tau_x| \cos \theta$, $\tau_{sy} = |\tau_y| \sin \theta$ in the $x$ and $y$ directions, if we adopt a coordinate system normal to a coastline, and the wind blows at an angle $\theta$ to the coast normal. Similarly, the resultant bottom stress $\tau_b$ may be related to the bottom current $U_b$ by a quadratic law,

$$\tau_b = C_B \rho U_b |U_b|$$

(2.27)

where $U_b$ is the measured at a standard reference height, usually taken as 1 m, $\tau_b$ is assumed to be in the direction of $U_b$ and $C_B$ is a coefficient of bottom friction. If $U_b$ has components $u_b$, $v_b$ in the $x$ and $y$ directions, then $\tau_b$ has components: $\tau_{bx} = C_B \rho U_b u_b$, $\tau_{by} = C_B \rho U_b v_b$, in which $U_b = (u_b^2 + v_b^2)^{1/2}$. Integration the continuity equation (2.19) in the same way gives

$$\frac{\partial \zeta}{\partial t} + \frac{\partial}{\partial x} [(h + \zeta)u] + \frac{\partial}{\partial y} [(h + \zeta)v] = 0$$

(2.28)

Although these equations (2.25), (2.26), and (2.28) are applied as frequently to the atmosphere as to the ocean, they bear the name shallow water equations. ADCIRC-2DDI numerically solves the shallow water equations by applying the finite element method (See Chapter 4.1).
This chapter represents a literature review and research discussion. For this literature review, there are three cases of coupling procedures relevant to this research: 1) a coupling of wave and hydrodynamic models in order to simulate the storm surges and investigate the wave-current interaction, 2) a coupling of wave and atmospheric models in order to consider the interaction between wind-induced wave and ocean surface roughness, and 3) a coupling of hydrodynamic and atmospheric models in order to simulate the global circulation corresponding to a change of the level of CO$_2$ and the Sea Surface Temperature (SST).

3.1 Coupling of Wave and Hydrodynamic Models

To begin, the coupling of wave and hydrodynamic models is discussed. The theory of wave-current interactions by the concept of wave radiation stresses was introduced and developed by Longuet-Higgins and Stewart (1960). This was the concept of momentum transfer from wave to currents through the gradient of additional stresses due to excess momentum flux of wave motion. Wolf et al. (1988) reported on a first attempt to dynamically couple the wave model and the hydrodynamic model. Wolf used the theory of Kitaigorodskii (1973) to calculate the drag from wave parameters. This theory provided values that were too high for the drag coefficients. Mastenbroek et al. (1993) investigated the wave-current interaction and exhibited that the normal
bulk law of sea surface stresses proposed by Smith and Banke (1975) underestimates the surge height by 20% compared to those computed by a wave-dependent drag coefficient.

There are five basic mechanisms for the storm tide generation at or near the shoreline: 1) astronomical tide owing to relative positions of the moon, sun and Earth, 2) inverted barometric effect (pressure surge), 3) a wind-driven surge caused by a strong onshore wind, 4) geostrophic tilt, a result of alongshore current, and 5) set-up from a short wave (wind-induced wave) (Reid, 1990). A storm surge is composed of only four (1–4) components while a storm tide consists of all the above. Figure 3.1 shows the schematic of a storm tide that represents the different contribution for the storm tide generation.

Many hydrodynamic models are capable of simulating the storm surge (pressure surge, wind-driven surge, geostrophic tilt, and astronomical tide). Short wave set-up and run-up (wind-induced wave), however, are not described by these hydrodynamic models because of their inability to calculate wind-induced wave set-up and run-up (Panchang et al. 1999). In spite of this, many models can incorporate output information from a wave model (in the form of the wave radiation stresses). Therefore, a coupling between the two models is possible.
Wave-current interaction is incorporated into the simulation by iteratively coupling wave and hydrodynamic models. The one-way interaction or the two-way interaction is applied for the coupling procedure. In the one-way interaction (Mastenbroek 1993, Ozer et al. 2000, Pandoe et al. 2005), shown in Figure 3.2, the wave radiation stresses, which are used as the surface stress forcing for the hydrodynamic model, computed by the wave model are provided to the hydrodynamic model. The hydrodynamic model calculates the currents and surface water levels by employing the wave radiation stresses. There is no feedback from the hydrodynamic model to the wave model. While in the two-way interaction (Zhang 1996, Cobb et al. 2002, Zundel et al. 2002, Weaver et al. 2004, Choi 2004, Moon 2005), shown in Figure 3.2, the wave radiation stresses are computed and then passed in the same way as the one-way interaction, the currents and surface water levels computed by the hydrodynamic model are fed back and utilized to
calculate new wave radiation stresses in the wave model. This procedure dynamically couples the two models through interchanging wave radiation stresses with surface water levels and currents.

Figure 3.2: A schematic of one- and two-way coupling of wave and hydrodynamic models.

### 3.2 Coupling of Wave model and Atmospheric Models

The coupling of wave and atmospheric models is utilized to simulate the air-sea interaction. The idea of a coupled atmosphere-wave model was proposed by Klaus Hasselmann (Hasselmann 1991), in the context of climate modeling. As waves are the “gearbox” between the atmosphere and the ocean, a detailed understanding of waves can significantly improve the parameterization of air-sea fluxes and surface processes (Fabrice 2005). This idea was more firmly established by the works of Janssen (1989, 1991). Janssen (1991) reported on the quasi linear theory of the
wind-wave generation that discussed the effect on the roughness of sea surface. Janssen concluded that the growth rate of waves generated by wind depends on a number of additional factors, such as wind gustiness and wave age that are neglected in the Charnock parameterization (1955).

Wave-atmosphere interaction, shown in Figure 3.3, is achieved by passing wind stresses $\tau$ (or wind speed $U_{10}$, 10 m above ocean surface) to drive the wave model and returning wave-induced stresses to the atmospheric model. Wave-induced stresses are computed within the wave model by the Janssen formulation (1991) or the original Charnock formulation (1955). This presents a modification of the Charnock parameter. The impact of this coupling mechanism is best estimated through the utilization of a fine resolution grid.

Figure 3.3: A schematic of coupling of wave and atmospheric models.
The coupling of wave and atmospheric models is capable of representing surface momentum fluxes that are enhanced due to young ocean waves in fetch-limited conditions, which yield surface roughness lengths that significantly depart from the conventional Charnock formulation. In general, the impact of ocean-wave-induced stresses on the central pressure of a tropical cyclone is quite variable, with ocean wave feedback resulting in changes ranging from 8 hPa in deeper waters to 3 hPa in shallower waters. The increased low-level stresses due to the ocean waves reduces the near-surface winds by 2-3 ms$^{-1}$, with local differences in excess of 10 ms$^{-1}$, which directly leads to a 10% reduction in the significant wave height maximum (Doyle 2002).

The coupling technique using the WAM (Wave Modeling) model is currently being applied operationally at the European Centre for Medium-Range Weather Forecasts (ECMWF) and the results indicate a substantial positive impact on the skill of the wave and atmospheric model forecasts (Janssen et al. 2002). This coupling technique has been applied to investigate the air-sea interaction in extratropical cyclones (Doyle 2002) and tropical cyclones (Bao et al. 2000). Tenerelli et al. (2001) examined the impact of coupling the fifth generation NCAR/PSU meso-scale model (MM5) to the third generation wave mode (Wavewatch III) on a simulation of Hurricane Floyd (1999). Perrie et al. (2001) describes an alternative coupling technique using the formulation of Smith et al. (1992), as derived in the HEXOS (Humidity Exchange over the Sea) experiment.
As a result, the coupling between wave models and atmospheric models is possible; moreover, these coupling techniques are utilized in forecast systems such as weather forecasts and ocean wave forecasts.

### 3.3 Coupling of Hydrodynamic and Atmospheric Models

The coupling of hydrodynamic and atmospheric models is employed to study the climate system, its natural variability, and its response to external forcings. The most important utilization of the coupling model has been to study how Earth’s climate might respond to a doubling of CO$_2$ in the atmosphere. Much of the literature on climate change is based on studies with such a coupling model. Other important utilizations of the coupling model include studies of El Niño and the potential modification of the major patterns for oceanic heat transport as a result of increasing greenhouse gases. The former varies over periods of a few years; the latter varies over a period of a few centuries (Stewart 2005).

Bryan and Manabe (1975) published results from the first coupled ocean-atmosphere circulation model that had a roughly Earth-like geography. Looking at their crude map, shown in Figure 3.4, one could make out continents like North America and Australia; however, smaller features, like Japan or Italy, are indistinguishable. The supercomputer ran for fifty straight days, simulating the movements of air and the sea over nearly three centuries. Development of the work tends to be coordinated through the World Climate Research Program of the World Meteorological
Organization (WCRPWMO), and recent progress is summarized in the Climate Change 1995 report by the Intergovernmental Panel on Climate Change (Gates et al. 1996).

Ocean-atmosphere interaction, shown in Figure 3.5, is achieved by passing the SST to drive the atmospheric model and returning surface fluxes and surface stresses to the hydrodynamic model. Salinity fluxes associated with evaporation (including sea spray evaporation) and freshwater influx by precipitation at the air-sea interface, and enthalpy flux modulation by sea spray are taken into account in the surface flux calculation. Note that ADCIRC-2DDI doesn’t include the salinity and temperature components; thus, the interaction is only the surface stress that is represented by the normal bulk formulation (Figure 3.5 displays the case of a hurricane event).
Figure 3.5: A schematic of coupling of wave and hydrodynamic models.

With an increase in computing power, the model has been developed to simulate more aspects of the real world. Boville and Gent (1998) developed the Climate System Model that includes physical and biogeochemical influence on the climate system. The Princeton Coupled Model consists of an atmospheric model with a horizontal resolution of 7.5° longitude by 4.5° latitude and 9 levels in the vertical, an ocean model with a horizontal resolution 4° and 12 levels in the vertical, and a land-surface model. Yet, it is still difficult to establish a simplified integration framework, particularly on a global scale, as present capabilities for modeling the Earth system are rather limited. However, models hereafter will make more advances by depending on the means of computational innovation.
3.4 Ultimate Coupling Model and Discussion

A coupled system of a wave, an atmospheric, and a hydrodynamic model is the ultimate goal in storm tide simulation and coastal modeling research. The image of three-way coupling is shown in Figure 3.6. Bao et al. (2000) studied air-sea interaction using such a system for the first time and successfully introduced the roles of ocean mixing, sea spray and sea-surface waves although the two-way coupling between the hydrodynamic and wave models was neglected. Yamashita et al. (2004) studied this coupling system under typhoon conditions; however, the interaction between the atmospheric model and the hydrodynamic model was disregarded. While a three-way coupled system of a wave, an atmospheric, and a hydrodynamic is a desirable goal in storm tide simulation and coastal modeling research, it is beyond the scope of a single PhD dissertation.

Figure 3.6: A schematic of coupling of wave, hydrodynamic, and atmospheric models.
Before attempting to couple three models, the present research is focused on the coupling of wave and hydrodynamic models. A substantial effort has been made to develop algorithms that promote the convergence criteria for velocity and the interpolation procedures for different grid configurations.

In order to achieve the coupling of the wave and hydrodynamic models, the following five issues must be discussed: 1) Evaluating the effects of the one- and two-way coupling methods, it is necessary to verify the influence of transferred values; 2) Establishing the convergence criteria in two-way coupling. This meant to minimize error of the transferred values during the computation; 3) Determining the coupling order, time scale, time step, and duration, i.e., which model drives first, when surface water levels, currents, and wave radiation stresses transfer during the computation, how long of a time step should be taken for each model, or when the simulation ends; 4) Determining how to interpolate each value through the different grid configuration. What kind of interpolation procedure should be employed, e.g., linear interpolation, inverse distance weighted interpolation? Also one must consider how much the interpolation error affects the results; and 5) Examining the distinct drag laws and parameterizations employed by the hydrodynamic model and wave model. A careful assessment and analysis of these five problem issues will result in a contribution to engineering science.
CHAPTER 4
MODEL DESCRIPTIONS

This chapter provides an overview of the hydrodynamic, wave and wind field models. The hydrodynamic model employed for calculating tides and storm surges is ADCIRC-2DDI (Luettich et al. 1992). The ADCIRC-2DDI model has been proven successfully for tidal and storm surge studies in coastal waters. The wave models applied for wind-induced wave simulations are the third-generation wave models, WAM and SWAN. The WAM model (WAMDI 1988) is one of the most sophisticated and validated models in the world. The SWAN model (Holthuijsen et al. 2004) is well recognized as being more suitable for the nearshore region than the WAM model. Wave fields computed by WAM are provided by Robert Jensen at U.S. Army Corps of Engineers (USACE), Engineer Research and Development Center (ERDC). Oceanwether, Inc. generously provided the computed tropical wind fields for Hurricane Floyd (1999).

4.1 ADCIRC-2DDI

ADCIRC-2DDI has been developed for the specific purpose of generating long time periods of two dimensional hydrodynamic calculations along shelves, coasts, and within estuaries (Luettich et al. 1992). ADCIRC-2DDI applies the depth integrated equations of mass and momentum conservation, subject to the hydrostatic pressure, incompressibility, and Boussinesq approximations. Under these assumptions, a continuity equation (4.1) and two momentum
equations, (4.2) and (4.3), expressed in a spherical coordinates system (Flather 1988; Kolar et al. 1992), are set up into the ADCIRC-2DDI computer code to solve hydrodynamic problems in order to describe shallow water tidal flow.

\[
\frac{\partial \zeta}{\partial t} + \frac{1}{R \cos \phi} \frac{\partial U H}{\partial \lambda} + \frac{1}{R} \frac{\partial V H}{\partial \phi} - \frac{V H \tan \phi}{R} = 0 \quad \text{(4.1)}
\]

\[
\frac{\partial U}{\partial t} + \frac{U}{R \cos \phi} \frac{\partial U}{\partial \lambda} + \frac{V}{R} \frac{\partial U}{\partial \phi} = \left( \frac{\tan \phi}{R} U + f \right) V
\]

\[
= -\frac{1}{R \cos \phi} \frac{\partial}{\partial \lambda} \left[ \frac{p_s}{\rho_0} + g(\zeta - \eta) \right] + \frac{\tau_{s \lambda}}{\rho_0 H} - \frac{\tau_{b \lambda}}{\rho_0 H} + D_\lambda - B_\lambda \quad \text{(4.2)}
\]

\[
\frac{\partial V}{\partial t} + \frac{U}{R \cos \phi} \frac{\partial V}{\partial \lambda} + \frac{V}{R} \frac{\partial V}{\partial \phi} + \left( \frac{\tan \phi}{R} U + f \right) U
\]

\[
= -\frac{1}{R} \frac{\partial}{\partial \phi} \left[ \frac{p_s}{\rho_0} + g(\zeta - \eta) \right] + \frac{\tau_{s \phi}}{\rho_0 H} - \frac{\tau_{b \phi}}{\rho_0 H} + D_\phi - B_\phi \quad \text{(4.3)}
\]

where \( t \) is time, \( \lambda \) is degrees longitude, east of Greenwich positive, \( \phi \) is degrees latitude, north of Equator positive, \( \zeta \) is free surface elevation, relative to the geoid, \( U \) and \( V \) are depth averaged velocities in the \( \lambda \) and \( \phi \) directions, \( R \) is radius of the Earth, \( H \) \((= h + \zeta)\) is total height of the water column, \( h \) is bathymetric depth, relative to the geoid, \( f = 2\Omega \sin \phi \) is the Coriolis parameter, in which \( \Omega \) is angular speed of the Earth, \( p_s \) is atmospheric pressure at the free surface, \( \rho_0 \) is reference density of water, \( g \) is acceleration due to gravity, \( \eta \) is Newtonian equilibrium tide potential, \( \tau_{s \lambda} \) and \( \tau_{s \phi} \) are applied free surface stresses (e.g., wind stresses and wave radiation stresses) in the \( \lambda \) and \( \phi \) directions, \( \tau_{b \lambda} \) and \( \tau_{b \phi} \) are applied bottom stresses in
the $\lambda$ and $\phi$ directions, $B_{\lambda}$ and $B_{\phi}$ are depth integrated baroclinic pressure gradient terms in the $\lambda$ and $\phi$ directions and $D_{\lambda}$ and $D_{\phi}$ are depth integrated momentum diffusion/dispersion terms in the $\lambda$ and $\phi$ directions. These equations have singularities at the poles and therefore will not behave well in these geographical regions.

The spherical equations are mapped to a rectilinear, distance-based coordinates system ($x, y$) using a Carte Paralleleogrammatique Projection (CPP) that is centered in longitude and latitude at $(\lambda_0, \phi_0)$: $x = R(\lambda - \lambda_0)\cos\phi_0$ and $y = R\phi$. In CPP coordinates, the continuity equation and the primitive momentum equations in nonconservative form are

$$\frac{\partial \zeta}{\partial t} + S \frac{\partial UH}{\partial x} + \frac{\partial VH}{\partial y} - \frac{VH \tan \phi}{R} = 0 \quad \text{..................................................(4.4)}$$

$$\frac{\partial U}{\partial t} + SU \frac{\partial U}{\partial x} + V \frac{\partial U}{\partial y} = \left( \frac{U \tan \phi}{R} + f \right) V$$

$$= -S \frac{\partial}{\partial x} \left[ \frac{p_z + g(\zeta - \eta)}{\rho_0} \right] + \frac{\tau_{xs}}{\rho_0 H} - \frac{\tau_{by}}{\rho_0 H} + D_x - B_y \quad \text{..................................................(4.5)}$$

$$\frac{\partial V}{\partial t} + SU \frac{\partial V}{\partial x} + V \frac{\partial V}{\partial y} = \left( \frac{\tan \phi}{R} U + f \right) U$$

$$= -\frac{\partial}{\partial y} \left[ \frac{p_z + g(\zeta - \eta)}{\rho_0} \right] + \frac{\tau_{xx}}{\rho_0 H} - \frac{\tau_{by}}{\rho_0 H} + D_x - B_y \quad \text{..................................................(4.6)}$$

Equations (4.4) – (4.6) are identical to the depth integrated governing equations in Cartesian coordinates with the exception that all terms containing $x$-derivatives are multiplied by a factor
$S \equiv \cos \phi / \cos \phi$ and an additional term containing $\tan \phi / R$ appears in each equation. In the momentum equations, $U \tan \phi / R$ can be treated as a modification to the Coriolis parameter. However, a scaling analysis shows that away from the immediate vicinity of the poles this term is several orders of magnitude smaller than the Coriolis parameter, and therefore, it is neglected in ADCIRC. No similar scaling argument can be made for the continuity equation, and therefore, the additional term should be retained in the numerical solution (Kolar et al. 1994).

To avoid well known numerical problems using a Galerkin finite element spatial discretization of this set of equations, the primitive continuity equations is replaced by a Generalized Wave Continuity Equation (GWCE). The GWCE is formed by taking the time derivative of the primitive continuity equation, reordering the terms, adding a parameter, $\tau_o$, and applying the chain rule:

$$\frac{\partial^2 \zeta}{\partial t^2} + \tau_o \frac{\partial \zeta}{\partial t} + S \frac{\partial A_x}{\partial x} + \frac{\partial A_y}{\partial y} - UHS \frac{\partial \tau_o}{\partial x} - VH \frac{\partial \tau_o}{\partial y} - \frac{A_y \tan \phi}{R} = 0 \tag{4.7}$$

where

$$A_x \equiv \frac{\partial UH}{\partial t} + \tau_o UH = \frac{\partial Q_x}{\partial t} + \tau_o Q_x \tag{4.8}$$

$$A_y \equiv \frac{\partial VH}{\partial t} + \tau_o VH = \frac{\partial Q_y}{\partial t} + \tau_o Q_y \tag{4.9}$$
Using the chain rule on the time derivative terms in the expressions for \( A_x \), \( A_y \) and substituting the momentum equations for \( \partial U / \partial t \), \( \partial V / \partial t \) results in:

\[
A_x = U \frac{\partial H}{\partial t} + H \left\{ -US \frac{\partial U}{\partial x} - V \frac{\partial U}{\partial y} + fV - S \frac{\partial}{\partial x} \left[ \frac{p_x}{\rho_0} + g(\zeta - \eta) \right] + \frac{\tau_{xx}}{\rho_0 H} - \frac{\tau_{by}}{\rho_0 H} + D_x - B_y + \tau_o U \right\} \tag{4.10}
\]

\[
A_y = V \frac{\partial H}{\partial t} + H \left\{ -US \frac{\partial V}{\partial x} - V \frac{\partial V}{\partial y} - fU - \frac{\partial}{\partial y} \left[ \frac{p_y}{\rho_0} + g(\zeta - \eta) \right] + \frac{\tau_{xx}}{\rho_0 H} - \frac{\tau_{by}}{\rho_0 H} + D_x - B_y + \tau_o V \right\} \tag{4.11}
\]

where

\[
B_x = \frac{g}{H} \int_{z_h}^{\zeta} \left\{ S \frac{\partial}{\partial x} \left[ \frac{\rho - \rho_0}{\rho_0} \right] dz \right\} dz = gS \left\{ \left( \frac{\bar{\rho} - \rho_0}{\rho_0} \right) \frac{\partial \zeta}{\partial x} + \frac{H}{2} \frac{\partial}{\partial x} \left( \frac{\bar{\rho} - \rho_0}{\rho_0} \right) \right\} \tag{4.12}
\]

\[
B_y = \frac{g}{H} \int_{z_h}^{\zeta} \left\{ \frac{\partial}{\partial y} \left[ \frac{\rho - \rho_0}{\rho_0} \right] dz \right\} dz = g \left\{ \left( \frac{\bar{\rho} - \rho_0}{\rho_0} \right) \frac{\partial \zeta}{\partial y} + \frac{H}{2} \frac{\partial}{\partial y} \left( \frac{\bar{\rho} - \rho_0}{\rho_0} \right) \right\} \tag{4.13}
\]

\[
D_x = \frac{E_h}{H} \left[ S^2 \frac{\partial^2 U H}{\partial x^2} + \frac{\partial^2 U H}{\partial y^2} \right] \tag{4.14}
\]

\[
D_y = \frac{E_h}{H} \left[ S^2 \frac{\partial^2 V H}{\partial x^2} + \frac{\partial^2 V H}{\partial y^2} \right] \tag{4.15}
\]
in which $E_h$ is horizontal eddy viscosity. The GWCE is obtained by substituting equation (4.10) and (4.11) for $A_x$, $A_y$ into equation (4.7). In spherical coordinates, ADCIRC solves the resulting GWCE together with the nonconservative momentum equations, (4.5) and (4.6).

Tidal forcing is normally imposed in ADCIRC via time and spatially varying levels along the open (elevation specified) boundaries of the model domain. However, ADCRIC also includes “extra” terms representing the Newtonian tidal potential and corrections due to the effect of the Earth tides. This extra term forces tides throughout the model domain. This term appears in the momentum equations, (4.5) and (4.6), as spatial gradients that are subtracted from the spatial gradient of the free surface elevation. In continental shelf areas, the free surface elevation gradient is typically much larger than the extra term and therefore they are safely neglected. However, the free surface gradient can be very small in the deep ocean, and therefore, when significant areas of the deep ocean are included in the model domain; this extra term may be significant. The Newtonian tidal potential and Earth tides are expressed as (Reid 1990):

$$\eta(\lambda, \phi, t) = \sum_{n,j} \alpha_{jn}C_{jn}f_{jn}(t_0)L_j(\phi)\left[\frac{2\pi(t-t_0)}{T_{jn} + j\dot{\lambda} + \nu_{jn}(t_0)}\right]$$ ...........................................(4.16)

where $t_0$ is reference time, $\alpha_{jn}$ is reduction in the field of gravity due to Earth tide, $C_{jn}$ is Newtonian equilibrium tidal potential amplitude, $f_{jn}$ is time-dependent nodal factor, $T_{jn}$ is tidal period, $\nu_{jn}$ is time-dependent astronomical argument, $j$ ($j = 0,1,2$) is tidal species, in which
\[ j = 0 \text{ (declinational), } j = 1 \text{ (diurnal) and } j = 2 \text{ (semidiurnal)}: L_0 = 3 \sin^2(\phi) - 1, \quad L_1 = \sin(2\phi) \]
\[ \text{and } L_2 = \cos^2(\phi) \] In addition, Reid (1990) consolidated the value of the effective earth elasticity factor, \( \alpha_{jn} \), which is typically applied as 0.69 for all tidal constituents (Schwiderski 1980; Hendershott 1981) even though the value has been shown to be slightly constituent dependent (Wahr 1981).

### 4.2 WAM and SWAN

The third generation wave models, WAM and SWAN, are described with the two-dimensional action balance equation (2.14). Equation (2.14) is described with the action density spectrum \( N(\sigma, \theta, \phi, \lambda, t) \) as a function of relative angular frequency \( \sigma \), wave direction, \( \theta \), latitude, \( \phi \), longitude, \( \lambda \), and time \( t \). \( \sigma = [(gk) \tanh(kd)]^{1/2} \) in which \( k = \frac{2\pi}{L} \), \( L \) being the wavelength) is the wave number, \( g \) is acceleration due to gravity and \( d \) is the water depth (sum of mean water depth, \( H \) and sea level elevation, \( \xi \)). The action density spectrum is defined as the energy density spectrum \( F(\sigma, \theta, \phi, \lambda, t) \) divided by \( \sigma \) observed in a frame moving with the ocean current velocity, which is \( N(\sigma, \theta, \phi, \lambda, t) = F(\sigma, \theta, \phi, \lambda, t)/\sigma \). The action density is chosen because it is conserved in the presence of time-dependent water depths and currents whereas the energy density spectrum is not. In general, the conservation equation for \( N \) in flux form in spherical coordinates and in frequency-direction space is given in the form:
\[
\frac{\partial N}{\partial t} + (\cos \phi)^{-1} \frac{\partial}{\partial \phi} \left( C_\phi \cos \phi N \right) + \frac{\partial}{\partial \lambda} (C_\lambda N) + \frac{\partial}{\partial \sigma} (C_\sigma N) + \frac{\partial}{\partial \theta} (C_\theta N) = \frac{S}{\sigma} \quad \text{.................(4.17)}
\]

where

\[
C_\phi = \frac{C_g \cos \theta + U}{R} \quad \text{............................................(4.18)}
\]
\[
C_\lambda = \frac{C_g \sin \theta + V}{R \cos \phi} \quad \text{............................................(4.19)}
\]
\[
C_\sigma = \frac{\partial}{\partial t} \left( \sqrt{gk \tanh(kd)} - \mathbf{k} \cdot \mathbf{U} \right) \quad \text{............................................(4.20)}
\]
\[
C_\theta = \frac{C_g \sin \theta \tan \phi}{R} + \frac{1}{kR} \left( \sin \theta \frac{\partial}{\partial \phi} - \cos \theta \frac{\partial}{\partial \lambda} \right) \left( \sqrt{gk \tanh(kd)} - \mathbf{k} \cdot \mathbf{U} \right) \quad \text{............................................(4.21)}
\]
\[
S = S_{in} + S_{nl} + S_{ds} + S_{bf} + S_{br} \quad \text{............................................(4.22)}
\]

In equation (4.17) the first term of the left hand side represents the local rate of change of action density in time, the second and third terms are propagation of action density in geographical space (with propagation velocities, \(C_\phi\) and \(C_\lambda\) in latitude and longitude space, respectively), the fourth term is the shifting of the relative frequency due to variations in depths and currents (with propagation velocity \(C_\sigma\) in \(\sigma\) space) and fifth term relates to the depth-induced and current-induced refraction (with propagation velocity \(C_\theta\) in \(\theta\) space). In equations (4.18) - (4.21), \(R\) is the radius of the Earth, \(U\), \(V\) are the current velocities in latitude and longitude space, respectively, \(\mathbf{k}\) is the vector of wave number and \(\mathbf{U}\) is the vector of current velocity.
The term $S = S(\sigma, \theta, \phi, \lambda, t)$ on the right hand side of equation (4.17) is the net source term expressed in terms of energy density. It is the sum of a number of source terms given in equation (4.22) representing the effects of wave generation by wind ($S_{in}$), nonlinear wave-wave interaction ($S_{nl}$), dissipation due to whitecapping ($S_{ds}$), depth induced breaking ($S_{bf}$) and bottom dissipation refraction ($S_{br}$).

The WAM model solves the energy balance form of equation (4.17) for no currents and fixed water depths on a spherical grid and in frequency-direction space. WAMDI Group (1988) describes the Cycle-3 version of WAM (WAM 3) in which $S_{in}$ and $S_{ds}$ are based on the formulations of Komen et al. (1984). In the WAM Cycle-4 version (WAM 4), $S_{in}$ and $S_{ds}$ are based on the formulations of Janssen (1989, 1991), in which the winds and waves are coupled, i.e., there is a feedback of growing waves on the wind profile. The effect of this feedback is to enhance the wave growth of younger wind waves over that of older wind waves for the same wind. The WAM 4.5 is an update of WAM Cycle-4. It uses the first order upwind explicit propagation scheme which results in the propagation time step being limited by the CFL condition and a fully implicit source term integration. To ensure that WAM remains numerically stable, a limitation on wave growth is imposed. WAM 4.5 is used in this study.

The SWAN model solves the action balance equation on a spherical grid and in $\sigma-\theta$ space. Because of the assumptions of time independent water depths and no currents, the solution of equation (4.17) is equivalent to the solution of the energy balance equation as in WAM 4.5. The
propagation scheme is fully implicit and for the source term integration scheme, the fully implicit option is chosen. SWAN has the option of using WAM 3 or WAM 4 physics for the $S_{in}$ and $S_{do}$ source terms. The SWAN (Cycle 3, version 40.41) is used in this study. In this research, WAM is employed for the global-scale simulation to provide the boundary conditions for SWAN, while SWAN is used in the local-scale simulation which is more close to the coast.

4.2.1 Wave Radiation Stresses

For basic fluid flow, some problems are best solved using the energy equation and others (such as a hydraulic jump where there is a strong concentrated dissipation of energy) the impulse-momentum principle. Similarly, for waves, some problems are best addressed considering the flux of momentum. This approach was first applied to waves by Longuett-Higgins and Stewart (1960, 1964) who introduced the term “radiation stress,” which they defined as “the excess flow of momentum due to the presence of waves.” (Sorensen 1993)

The radiation stress components presented below are useful for analyzing a number of wave phenomena, including mean water level set up in the surf zone, wave-current interaction, and the alongshore currents generated in the surf zone by waves that obliquely approach the shore. The instantaneous horizontal flux of momentum at a given location consists of the pressure force on a vertical plane plus the transfer of momentum through that vertical plane. The latter is the product of the momentum in the flow and the flow rate across the plane. Dividing by the area of the
vertical plane yields the momentum flux for the $x$ direction which is $P + \rho u^2$. The resulting radiation stress $S_{xx}$ for a wave propagating in the $x$ direction is

$$S_{xx} = \int_{-d}^{d} (P + \rho u^2)dz - \int_{-d}^{d} \rho gzdz ..............................................................(4.23)$$

where the subscript $xx$ denotes the $x$–directed momentum flux across a plane defined by $x = \text{constant}$. In the equation shown above $p$ is the total static plus dynamic pressure so that the static pressure must be subtracted to obtain the radiation stress for the wave. The over bar denotes that the term is averaged over the wave period. Inserting the pressure and particle velocity terms from small amplitude theory yields

$$S_{xx} = \frac{\rho g H^2}{8} \left( \frac{1}{2} + \frac{2kd}{\sinh 2kd} \right) = E \left( 2n - \frac{1}{2} \right) ..........................................................(4.24)$$

Likewise $S_{yy}$, the $y$–directed momentum flux across a plane defined by $y = \text{constant}$ is

$$S_{yy} = \frac{\rho g H^2}{8} \left( \frac{kd}{\sinh kd} \right) = E \left( n - \frac{1}{2} \right) ..........................................................(4.25)$$
The radiation stress components $S_{xy}$ and $S_{yx}$ are both equal to zero. Note, in deep water $S_{xx} = E / 2, S_{yy} = 0$ and in shallow water $S_{xx} = 3E / 2, S_{yy} = E / 2$. If a wave is propagating in a direction that is at an angle with the $x$ direction, the radiation stress components become

$$
S_{xx} = \overline{E} \left[ n(\cos^2 \theta + 1) - \frac{1}{2} \right]
$$

$$
S_{yy} = \overline{E} \left[ n(\sin^2 \theta + 1) - \frac{1}{2} \right]
$$

$$
S_{xy} = S_{yx} = \frac{\overline{E}}{2} n \sin^2 \theta = \overline{E} n \sin \theta \cos \theta \tag{4.26}
$$

WAM and SWAN output the momentum transfer from the wave field to the depth-averaged currents by integrating the radiation stresses over the water direction and frequency spectrum. The $x$ and $y$ components of the momentum transfer are

$$
\tau_{xx} = \frac{1}{\rho} \left[ - \frac{\partial S_{xx}}{\partial x} - \frac{\partial S_{xy}}{\partial y} \right] \tag{4.27}
$$

$$
\tau_{xy} = \frac{1}{\rho} \left[ - \frac{\partial S_{yx}}{\partial x} - \frac{\partial S_{yy}}{\partial y} \right] \tag{4.28}
$$

The wave radiation stresses are used as surface stresses conditions in the ADCIRC-2DDI model (See Chapter 4.1).
4.3 Wind Field Model

The state-of-art wind data for the storm tide simulation is provided by Oceanweather, Inc. using a tropical wind model, TC96, developed by Cox and Cardone (2000). This model was first developed into a practical tool in the Ocean Data Gathering Program (ODGP) (Cardone et al. 1976). It can provide a fairly compete description of the time-space evolution of the surface winds in the boundary layer of a tropical cyclone. The model is an application of a theoretical model of the horizontal airflow in the boundary layer of a moving vortex, where this model solves, by numerical integration, the vertically averaged equations of motion that govern a boundary layer subject to horizontal and vertical shear stresses. Based on this principle, the numerical model provides a fairly thorough description of the time-space evolution of the wind speeds within the planetary boundary layer (PBL) during a tropical cyclone event (Thompson and Cardone 1996).

The wind field model is driven by “snapshots” in time of the storms intensity and is based on the assumption that the structure of the hurricane changes relatively slowly (Cox and Cardone 2000). In addition to the TC96 model, these snapshots are also obtained from the Hurricane Research Division Wind Analysis System (H*Wind), a distributed system that uses real-time tropical cyclone observations as input (Powell et al. 1998). The entire wind field history is interpolated from the snapshots. The wind field model is free of arbitrary calibration constants that might link the model to a particular storm type or region. The variations in structure between tropical storm types manifest themselves basically in the characteristics of the pressure field of the vortex itself.
and of the surrounding region. The interaction of a tropical cyclone and its environment, therefore, can be accounted for by a proper specification of the input parameters. The assignable parameters of the PBL formulation, namely PBL depth and stability, and of the sea surface roughness formulation, can safely be taken from a number of studies (Cox and Cardone 2002).

Figure 4.1: Hurricane Floyd wind field.
4.3.1 Wind Stresses for ADCIRC-2DDI

The wind speeds that are computed by the wind field model are used as surface stress conditions in the ADCIRC-2DDI model. In order to convert wind speed to wind stress, ADCIRC employs the following formulation:

\[ \tau_{sx} = C_D \rho_{air} |W| W_x \] ..............................................................(4.29)

\[ \tau_{sy} = C_D \rho_{air} |W| W_y \] ..............................................................(4.30)

where \( \rho_{air} \) is air density, \( C_D \) is frictional drag coefficient, \((0.75 + 0.067 \times W) \times 10^{-3}\) developed by Garrett (1977), \(|W|\) is the magnitude of wind velocity and \( W_x \) and \( W_y \) are components of wind velocity in the \( x \) and \( y \) directions at 10 m height above the mean water level. Furthermore, the wind field model also provides a pressure gradient to ADCIRC. A conversion is then performed within ADCIRC to convert the pressure gradient to an equivalent water column height through the transformation \( p / \rho_g g \) (Blain et al, 1994b). In the end, the wind stress and the equivalent water column height are linearly interpolated to the each computational node of the finite element mesh used by ADCIRC.
4.3.2 Wind Stresses for WAM and SWAN

Transfer of wind energy to the waves is described in SWAN with a resonance mechanism (Phillips 1957) and a feed-back mechanism (Miles 1957). The corresponding source term for these mechanisms is commonly described as the sum of linear and exponential growth:

\[ S_{\theta}(\sigma, \theta) = A + BE(\sigma, \theta) \] ...................................................(4.31)

in which \( A \) describes linear growth and \( BE \) exponential growth. It should be noted that the SWAN model is driven by the wind speed at 10 m elevation \( W_{10} \) whereas the computations use the friction velocity \( W_* \). For the WAM Cycle 3 formulation the transformation from \( W_{10} \) to \( W_* \) is obtained with

\[ U_*^2 = C_D W_{10}^2 \] ...................................................(4.32)

in which \( C_D \) is the drag coefficient from Wu (1982).

\[
C_D(W_{10}) = \begin{cases} 
1.2875 \times 10^{-3} & \text{for } W_{10} < 7.5 \text{ m/s} \\
(0.8 \times 0.065 \text{ m/s} \times W_{10}) \times 10^{-3} & \text{for } W_{10} \geq 7.5 \text{ m/s}
\end{cases} \] ...................................................(4.33)

For the WAM Cycle 4 formulations, the computation of \( W_* \) is an integral part of the source term.
For the linear growth term $A$, the expression due to Cavaleri and Malanotte-Rizzoli (1981) is used with a filter to eliminate wave growth at frequencies lower than the Pierson-Moskowitz frequency (Tolman 1992)

\[
A = \frac{1.5 \times 10^{-3}}{g^2 2n} \left[ W_* \max[0, \cos(\theta - \theta_w)] \right]^4 H
\]

\[
H = \exp\left(-\left(\frac{\sigma}{\sigma_{PM}^*}\right)^4\right) \quad \text{with} \quad \sigma_{PM}^* = \frac{0.13 g}{28 W_*}
\]

in which $\theta_w$ is the wind direction, $H$ is the filter and $\sigma_{PM}^*$ is the peak frequency of the fully developed sea state according to Pierson and Moskowitz (1964).

Two expressions for exponential growth by wind are optionally available in the SWAN model. The first expression is due to Komen et al. (1984). Their expression is a function of $W_*/c_{ph}$:

\[
B = \max\left[ 0, 0.25 \frac{\rho_a}{\rho_w} \left[ 28 \frac{W_*}{c_{ph}} \cos(\theta - \theta_w) - 1 \right] \right] \sigma \quad \text{........................................................(4.35)}
\]

in which $c_{ph}$ is the phase speed and $\rho_a$ and $\rho_w$ are the density of air and water, respectively. This expression is also used in WAM Cycle 3. The second expression is due to Janssen (1989, 1991). It is based on a quasi-linear wind-wave theory and is given by:
\[
B = \beta \left( \frac{\rho_a}{\rho_w} \left( \frac{W_*}{c_{ph}} \right)^2 \text{max}[0, \cos(\theta - \theta_w)] \right) \sigma
\]

where \( \beta \) is the Miles “constant”. In the theory of Janssen (1991), this Miles “constant” is estimated from the non-dimensional critical height \( \lambda \):

\[
\begin{align*}
\beta &= \frac{1.2}{\kappa^2} \lambda \ln^4 \lambda & \lambda \leq 1 \\
\lambda &= \frac{g z_e}{u} e^r & r = \kappa c / W_* \cos(\theta - \theta_w)
\end{align*}
\]

where \( \kappa \) is the Von Karman constant, equal to 0.41 and \( z_e \) is the effective surface roughness. If the non-dimensional critical height \( \lambda > 1 \), the Miles constant \( \beta \) is set equal to 0. Janssen (1991) assumes that the wind profile is given by:

\[
W(z) = \frac{W_*}{\kappa} \ln \left( \frac{z + z_e - z_o}{z_e} \right)
\]

in which \( W(z) \) is the wind speed at the height \( z \) (10 m in the SWAN model) above the mean water level, \( z_o \) is the roughness length. The effective roughness length \( z_e \) depends on the roughness length \( z_o \) and the sea state through the wave induced stress \( \tau_w \) and the total surface stress \( \tau \):
\[ z_e = \frac{z_o}{\sqrt{1 - \tau_w / \tau}} \quad \text{and} \quad z_o = \hat{\alpha} \frac{W^2_s}{g} \] .................................................................(4.39)

The second of these two equations is a Charnock-like relation in which \( \hat{\alpha} \) is a constant equal to 0.01. The wave stress \( \tau_w \) vector is given by:

\[ \tau_w = \rho_w \int_0^{2\pi} \int_0^1 \sigma E(\sigma, \theta) \frac{k}{k_d} d\sigma d\theta \] .................................................................(4.40)

The value of \( W \), can be determined for a given wind speed \( W_{10} \) and a given wave spectrum \( E(\sigma, \theta) \) from the above set of equations. In the SWAN model the iterative procedure of Mastenbroek et al. (1993) is used. This set of expressions (equations (4.36) - (4.40)) is also used in WAM Cycle 4.5 (Komen et al. 1994).
CHAPTER 5

FINITE ELEMENT MESHES AND FINITE DIFFERENCE GRID DEVELOPMENT

A numerical model for storm tides must resolve the physical equations that affect the astronomical tide and storm surge generation as well as the wave propagation. The period, wavelength, amplitude, and direction features of storm tides rely on the geometric properties of the water body and the characteristics of the meteorological forcings (Blain et al. 1994b). Therefore, in order to obtain reliable simulation results a model domain has to incorporate complex coastal geometries, account for quickly changing bathymetry in the continental slope and shelf areas, and permit reasonable boundary condition specifications. The finite element mesh and the finite difference grid supply a means by which to satisfy these requirements.

Presented in this chapter is the development of the finite element mesh and the finite difference grid for two different domain scales (global-scale and local-scale domain). First, a newly generated finite element mesh that represents detailed topographic features in the St. Johns River is incorporated into WNAT model domain to produce a global-scale domain mesh. Second, for the local-scale domain mesh, a semicircular mesh encompassing the Florida Atlantic Coast and continental shelf is extracted from the global-scale domain mesh. Third, a global-scale finite difference grid is provided by USACE. Finally, a local-scale finite difference grid is produced along the Florida Atlantic Coast including Fernandina Beach, Mayport, and St. Augustine Beach.
5.1 St. Johns River Region

The study area is located along the Florida Atlantic Coast in the northeast portion of the state of Florida and the southern portion of the Georgia coast. The main focus of this study is on a riverine system that is strongly influenced by inflows, astronomical and meteorological tides, and waves. The system includes Fernandina Beach, St. Augustine Beach, and the lower St. Johns River, from Lake George down to the mouth near Mayport (Figure 5.1).

Figure 5.1: St. Johns River region.
The lower (northern) St. Johns River has a length of 170 km (Figure 5.2). The lower St. Johns River receives 60% of its total annual freshwater flow from sources upstream of Buffalo Bluff. The surrounding local watersheds of the lower St. Johns River encompass 6000 km², about 27% of the total watershed area of 22,000 km². The local watersheds, downstream of Buffalo Bluff, contribute 40% of the total annual flow to the river and also contribute significantly to peak flow at the river mouth because of the relatively rapid delivery of surface runoff from the local watersheds following rainfall events (Sucsy and Morris 2002).

Figure 5.2: Lower St. Johns River and major drainage basins (Sucsy and Morris 2002).
5.2 Finite Element Mesh Development

5.2.1 The Global-Scale ADCIRC Mesh (WNAT-SJR Mesh)

Two different model domain meshes are used to simulate the storm tides owing to Hurricane Floyd in/out of the St. Johns River: the global-scale mesh (WNAT-SJR mesh) and the local-scale mesh (Pseudo-Operational mesh). Each mesh incorporates similar physical features in the St. Johns River; however, the mesh includes different ocean features.

The bathymetric data for these meshes are obtained via the St. Johns River Water Management District (SJRWMD) and the National Geophysical Data Center (NGDC) Coastal Relief CD-ROM, Volume 3. The database consists of 3-arc second digital elevation map (via the United States Geological Survey [USGS]) and hydrographic soundings (via the National Ocean Service [NOS]). For regions extending beyond the St. Johns River, the bathymetry is interpolated from an existing high resolution WNAT model domain mesh (WNAT-333K).

Previous efforts by Hagen et al. (In Press) have resulted in the development of a finite element mesh for tidal computations in the WNAT model domain (WNAT-48K). This grid contains 47,860 computational nodes and 89,212 triangular elements, and was developed using node spacing guidelines generated from a Localized Truncation Error Analysis (LTEA) (Hagen et al. 1998). It provides a detailed description of the physical system with node spacing ranging from 0.5 to 160 km. The new WNAT-SJR mesh is developed by incorporating the St. Johns River
features into the WNAT-48K mesh. The new mesh includes 73,279 nodes and 135,247 elements, and covers a horizontal surface area of approximately $8.346 \times 10^6$ km$^2$. Figures 5.3 through 5.9 present the finite element discretization and bathymetry for the WNAT-SJR mesh with insets of the St. Johns River. The USGS aerial photography and map as supplied by TerraServer-USA are used to extend the boundary in the St. Johns River.

Figure 5.3: Finite element mesh for the WNAT-SJR model.
Figure 5.4: Bathymetry for the WNAT-SJR model.
Figure 5.5: Finite element mesh and bathymetry for St. Johns River (maps and photos from USGS).
Figure 5.6: Finite element mesh and bathymetry for the St. Johns River: inset α.
Figure 5.7: Finite element mesh and bathymetry for the St. Johns River: inset $\beta$.

Figure 5.8: Finite element mesh and bathymetry for the St. Johns River: inset $\gamma$. 
Figure 5.9: Finite element mesh and bathymetry for the St. Johns River: inset $\delta$. 
5.2.2 The Local-Scale ADCIRC Mesh (Pseudo-Operational Mesh)

The Pseudo-Operational mesh incorporates the bathymetric features of the continental shelf. A semi-circular, open ocean boundary is extended from Springmaid Pier, SC to Lake Worth Pier, FL, encompassing the continental shelf. The mesh consists of 26,543 nodes and 47,763 elements, and was generated by the Surface Water Modeling System (SMS). Figures 5.10 and 5.11 show the finite element discretization and bathymetry, respectively for the local-scale domain. The Pseudo-Operational mesh file (fort.14) is shown in Appendix A.

![Figure 5.10: Finite element mesh and bathymetry for the Pseudo-Operational model.](image)
5.3 Finite Difference Grid Development

5.3.1 The Global-Scale WAM Grid

A global-scale finite difference grid with spacing of 0.1° provided by USACE is employed for the WAM computational domain. The domain encompasses a broad range, from 99° W to 50° W in longitude and from 5° N to 53° N in latitude, including the Gulf of Mexico, Caribbean Sea, a large portion of the Northwest Atlantic Ocean. Figure 5.11 shows the computed maximum significant wave height produced by Hurricane Floyd.

Figure 5.11: Wave field of the WAM model and maximum significant wave height generated by Hurricane Floyd (1999).
5.3.2 The Local-Scale SWAN Grid

A local-scale finite difference grid with spacing of 0.005° is employed for the SWAN computational domain (Figure 5.12). Brown cells represent land and blue cells relate to the ocean. The 1.0° × 1.0° area consists of 200 (longitude direction) and 200 (latitude direction) square cells. Both longitudinal and latitudinal distances (approximately 95 km and 110 km, respectively) provide ample fetch to generate the wind-induced waves. The range of bathymetry shown in Figure 5.13 is from 2.0 to -40.0 meters which is sufficient for the wave model.
5.4 Coupling Model Domain

Both local-scale model domains are put on the same coordinate grid in order to interact with each model. Figure 5.14 shows the finite difference grid overlaid on the finite element mesh, to cover the Florida Atlantic Coast including Fernandina Beach, Mayport, and St. Augustine Beach.
Figure 5.14: Overlapped finite element mesh and finite difference grid and NOS tidal gauge stations.
CHAPTER 6
MODEL SETUP

Two different models are used in this research: ADCIRC and SWAN. In order to establish a robust coupling procedure, the ADCIRC and SWAN model parameters must be set according to the domain type and study specification. For instance, parameters that specify the run time, time step, time interval interacts with each model, forcing frequencies, etc. must be included in the model setup. Furthermore, the water level output locations that are used to verify the model results must be specified in the input file for each domain. Presented herein are the model parameters and output locations for the ADCIRC and SWAN models.

6.1 The ADCIRC Model

6.1.1 Astronomical Tides Verification

The model parameters for the astronomical tide verification are set as follows: the coordinate system is set to spherical. Simulations are begun from a cold start. Seven harmonic forcing are applied simultaneously along the open ocean boundary (M_2, K_1, O_1, N_2, K_2, Q_1, and S_2) (See Table 6.1) and are ramped over a 20-day period. The hybrid bottom friction formulation is employed with the following settings: minimum friction coefficient, $C_{f_{min}} = 0.0025$, break depth, $H_{break} = 10$ m, and two dimensionless parameters $\theta = 10$ and $\lambda = 1/3$ (Murray 2003). The
horizontal eddy viscosity coefficient is set at 5 m$^2$/sec, and a time step of 5 seconds is used to ensure model stability.

Table 6.1: Tidal constituents used to force the ADCIRC model.

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Name</th>
<th>Period [hr]</th>
<th>Frequency [rad/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M$_2$</td>
<td>Principal lunar semidiurnal</td>
<td>12.42</td>
<td>0.000140518917083</td>
</tr>
<tr>
<td>K$_1$</td>
<td>Luni-solar diurnal</td>
<td>23.93</td>
<td>0.000072921165921</td>
</tr>
<tr>
<td>O$_1$</td>
<td>Principal lunar diurnal</td>
<td>25.82</td>
<td>0.000067597751162</td>
</tr>
<tr>
<td>N$_2$</td>
<td>Larger lunar elliptic</td>
<td>12.66</td>
<td>0.000137879700000</td>
</tr>
<tr>
<td>K$_2$</td>
<td>Luni-solar semidiurnal</td>
<td>11.97</td>
<td>0.000145842317201</td>
</tr>
<tr>
<td>Q$_1$</td>
<td>Larger lunar elliptic diurnal</td>
<td>26.87</td>
<td>0.000064958541129</td>
</tr>
<tr>
<td>S$_2$</td>
<td>Principal solar semidiurnal</td>
<td>12.00</td>
<td>0.000145444119418</td>
</tr>
</tbody>
</table>

6.1.2 River Inflow Verification

The model parameters for the river inflow verification are set as follows: Total simulation time is 20 days (September 1, 1999 to September 20, 1999), with a 5 seconds time step. ADCIRC is solved in the spherical coordinate system. The simulation is spun up from rest over a 5 day period via a hyperbolic ramp function. A hybrid bottom friction formulation and tidal elevation forcings are employed as the same as the astronomical tide verification. River inflows read into the simulation every 30 minutes.
6.1.3 Hurricane Floyd Wind Forcing Verification

The model parameters for the Hurricane Floyd wind forcing verification are set as follows: Total simulation time is 4.75 days (September 12, 1999, 0:00 GMT to September 16, 18:00 GMT), with a 5 seconds time step. ADCIRC is solved in the spherical coordinate system. The simulation is spun up from rest over a 0.05 day period via a hyperbolic ramp function. A hybrid bottom friction formulation and tidal elevation forcings are employed as the same as the astronomical tide verification. River inflows and meteorological forcings (wind stresses and pressure) are read into the simulation every 30 minutes.

6.1.4 Coupling of the SWAN model for Hurricane Floyd Storm Tide Simulation

The model parameters for the Hurricane Floyd storm tide simulation are set as follows: Total simulation time is 4.75 days (September 12, 1999, 0:00 GMT to September 16, 18:00 GMT), with a 5 seconds time step. ADCIRC is solved in the spherical coordinate system. The simulation is spun up from rest over a 0.05 day period via a hyperbolic ramp function. A hybrid bottom friction formulation is employed with the following specifications: minimum friction coefficient, $C_{f_{\text{min}}} = 0.0025$, break depth, $H_{\text{break}} = 10$ m, and two dimensionless parameters $\theta = 10$ and $\lambda = 1/3$ (Murray 2003). Seven tidal elevation forcings employed with the following specifications: ($M_2$, $K_1$, $O_1$, $N_2$, $K_2$, $Q_1$, and $S_2$) are applied at the open ocean boundaries. River inflows and meteorological forcings (wind stresses and pressure) are read into the simulation every 30 minutes, and wave forcings (wave radiation stresses) from the SWAN model are read
into the simulation every 2 hours. The ADCIRC parameter input file (fort.15) for the Hurricane Floyd storm tide simulation is shown in Appendix B.

6.2 The SWAN Model

The model parameters for the SWAN simulations are as follows: By selecting a stationary mode, SWAN performs 33 times to provide the wave field for ADCIRC every 2 hours. SWAN is solved in the spherical coordinate system. A JONSWAP spectra shape is selected (30 frequencies and 35 directions), and the peak enhancement factor (gamma) is set to 3.3. The offshore boundary conditions associated with Hurricane Floyd are provided from the WAM Model. The significant wave height, wave peak period and wave direction are input at the offshore boundary of SWAN. Water level and currents from the ADCIRC model are read into the simulation. The SWAN parameter input file is shown in Appendix C.

6.3 Model Output Locations

Harmonic tide data and water level data are available from NOAA\NOS at 11 tidal gauge locations in the St. Johns River. Two additional NOAA\NOS tidal gauge locations along the Florida Atlantic Coast are used to further evaluate the model results. These data are tidal amplitude and phases derived from harmonic analyses of observed hourly water levels and time-series of observed water levels at 6 minutes interval. The locations of these stations are shown in Figure 6.1 and listed in Table 6.2.
Figure 6.1: NOAA\NOS tidal gauge locations for the Florida Atlantic Coast and the St. Johns River.
Table 6.2: NOAA\NOS tidal gauge locations shown in Figure 6.1.

<table>
<thead>
<tr>
<th>NOS ID</th>
<th>Location Name</th>
<th>Longitude (°W)</th>
<th>Latitude (°N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8720030</td>
<td>Fernandina Beach</td>
<td>-81.465</td>
<td>30.672</td>
</tr>
<tr>
<td>8720211</td>
<td>WWTD, Mayport Naval Station</td>
<td>-81.413</td>
<td>30.400</td>
</tr>
<tr>
<td>8720219</td>
<td>Dame Point</td>
<td>-81.558</td>
<td>30.387</td>
</tr>
<tr>
<td>8720220</td>
<td>Mayport</td>
<td>-81.432</td>
<td>30.393</td>
</tr>
<tr>
<td>8720226</td>
<td>Main Street Bridge</td>
<td>-81.658</td>
<td>30.320</td>
</tr>
<tr>
<td>8720242</td>
<td>Longbranch (USE-DDP)</td>
<td>-81.620</td>
<td>30.360</td>
</tr>
<tr>
<td>8720357</td>
<td>I-295 Bridge, West End</td>
<td>-81.692</td>
<td>30.192</td>
</tr>
<tr>
<td>8720503</td>
<td>Red Bay Point</td>
<td>-81.692</td>
<td>29.978</td>
</tr>
<tr>
<td>8720587</td>
<td>St. Augustine Beach</td>
<td>-81.263</td>
<td>29.857</td>
</tr>
<tr>
<td>8720625</td>
<td>Racy Point</td>
<td>-81.548</td>
<td>29.802</td>
</tr>
<tr>
<td>8720767</td>
<td>Buffalo Bluff</td>
<td>-81.682</td>
<td>29.595</td>
</tr>
<tr>
<td>8720774</td>
<td>Palakta</td>
<td>-81.632</td>
<td>29.643</td>
</tr>
<tr>
<td>8720832</td>
<td>Wekala</td>
<td>-81.675</td>
<td>29.477</td>
</tr>
</tbody>
</table>
CHAPTER 7

SIMULATION RESULTS

This chapter is divided into three sections: 1) the ADCIRC-only simulation to verify the St. Johns River hydrodynamic model; 2) the uni-coupling model simulation, which is a one-way communication where the SWAN wave model provides wave radiation stresses to the ADCIRC hydrodynamic model; and 3) the coupled model simulation that permits for the complete interaction between the SWAN wave model and the ADCIRC hydrodynamic model.

7.1 The ADCIRC Model Simulation

First, a comparison between computed and historical tidal signals is shown at the NOS gauge locations to verify that the ADCIRC model is accurately simulating the astronomical tides. Second, river level outputs (with and without river inflows) are evaluated at the NOS gauge locations. The comparison confirms that the river inflows have no significant effect on the computed river levels for the time period when tropical cyclones and hurricanes approach. Third, the water level outputs generated by a 122-day simulation (2005 Hurricane Season spanning June 1 – October 1, 2005) and Hurricane Floyd are compared with the historical NOS data along the Florida Atlantic Coast and inside the river (See Figure 6.1 and Table 6.2). In addition, a sensitivity analysis is presented on a select few parameters and domain specifications. It is noted that many factors can be adjusted in a numerical study, but only a few are presented herein to
examine the sensitivity of model results to the change in certain parameters (e.g. bottom friction and drag coefficients).

7.1.1 Astronomical Tide Verification

The Pseudo-Operational model is first verified through the astronomical tide comparison. A total of 13 NOS tide gauges located along the Florida Atlantic Coast and within the St. Johns River provide historical tidal constituents data (See Figure 6.1 and Table 6.2). A 14-day resynthesis of 23 model constituents derived from the WNAT-SJR model are compared to a 14-day resynthesis of the 37 historical NOS constituents (Hagen et al. In Press).

This verification also explores the effect of the time and advective terms in the GWCE (See equation 4.7). Highly non-linear flows such as shallow converging sections around islands and flood waves propagating onto dry land typically signifies regions where local mass imbalances may occur (Kolar et al. 2000). Thus, it is important to explore whether the time and advective terms in the GWCE play a significant role on the upstream location where highly advective flow dominates. Figures 7.1 through 7.3 show the astronomical tide comparison at Mayport, I-295 Bridge West End, and Wekala, respectively: 1) Historical (black line); 2) No-Time-Adective (red line); and 3) Time-Adective (blue line). Other location results are shown in Appendix D.
Figure 7.1: Astronomical tide comparison at Mayport.

Figure 7.2: Astronomical tide comparison at I-295 Bridge West End.
Figure 7.3: Astronomical tide comparison at Wekala.

The results indicate that both simulations perform reasonably well at all locations; however, the following discrepancies are observed between the model tidal signals presented in Figures 7.1 to 7.3. First, tidal range is over-predicted inside the river when excluding the time and advective terms in the GWCE. Second, a discrepancy in amplitude is recognized in both cases, most notably in the troughs through most of the spring-neap tidal cycle. For the former problem, it is solved because amplitude errors with respect to the historical data are minimized by including the time and advective terms in the GWCE. They play a significant role to minimize the local mass balance errors in the simulation. The latter problem is attributed to the inappropriate bathymetric data or malfunction of the wetting/drying conditions of the ADCIRC model. If the bathymetry is not enough to compute water levels and velocities, the wetting/drying conditions

88
lose an accuracy of computing them. Thus, the ADCIRC model has a tendency to under-estimate the water levels, particularly at the troughs, observed at shallow bathymetric areas when one carries out the harmonic analysis in its simulation. However, overall the model faithfully simulates the tidal dynamics at the entire NOS tide gauge locations employed in this study, certainly sufficiently for it to be used in the uni-coupling and coupling study that follows.

7.1.2 River Inflow Verification

The Pseudo-Operational model is further evaluated through the river level comparison. Five unsteady river inflow events (30 minutes interval) are read into the model to examine the effect of river inflow on the river levels. Figure 7.4 a represents the USGS gauge and river inflow locations in the simulation. Historical USGS streamflow data including precipitation is used to generate the inflow conditions. For example, the inflow conditions at Astor are generated by summing all streamflow data measured at all gauges above Astor (Geneva, Sanford, Wekiva River, and Deland). It should be noted that the travel time of the streamflow between gauges are ignored in the simulation. Figure 7.4 b shows the relationship between precipitation and wind speed at Sanford in September 2005 (See Figure 7.4 a). Since Hurricane Ophelia (inset box) brings some rainfall and high winds to the St. Johns River in this period, it is an ideal case to study the influence of river inflow. Figures 7.5 through 7.7 show the river level comparison at Mayport, I-295 Bridge West End, and Buffalo Bluff (See Figure 6.1): 1) Historical (black line); No-Inflow (red line); and Inflow (blue line). Other location results are shown in the Appendix D.
Figure 7.4: a) USGS gauge and river inflow locations and b) a relationship between precipitation [in] and average wind speed [mph] at Sanford.
Figure 7.5: River level comparison at Mayport.

Figure 7.6: River level comparison at I-295 Bridge West End.
Figures 7.5 to 7.7 display river level comparison corresponding to three tidal gauge locations within the St. Johns River (See Figure 6.1). Some interesting results are drawn from these figures. First, including river flow clearly improves the river levels at all locations. Second, the computed river levels are largely deviated from the historical data. Including the river inflow improves the river levels, increasing water levels an approximately 20 cm higher than no-inflow condition; however, the large deviations are observed between the historical and the computed water levels. The model can not follow water level rising when Hurricane Ophelia approaches with high winds and brings heavy rainfall to the St. Johns River.
Two reasons arise to cause such a huge discrepancy: 1) inappropriate streamflow data and river inflows conditions; 2) the meteorological influences (wind and pressure). With regards to the former, if additional river inflows are provided in the simulation, computed river levels increase and are close to the historical data; however, they can not perfectly much up with the historical data (e.g., the highest peak in the simulation period). Because computed water levels are relatively constant regardless of inflow conditions. In addition, considering the accurate USGS data, it is less likely that inappropriate inflow conditions cause such a huge deviation. It might be apparent that other sources have an affect on the river level rising. Thus, the following chapter will include the wind forcings (wind and pressure) to explore whether the wind forcings play a significant role or not in the St. Johns River.

7.1.3 Wind Forcings Verification and 122-day Simulation

Wind forcings verification in the Pseudo-Operational model is undertaken to investigate how the wind and pressure influence the river level rising. In addition, a long-term simulation is carried out in order to examine the robustness and reliability of the Pseudo-Operational model. The model performs a 122-day simulation (June 1 to October 1, 2005) of the 2005 Atlantic hurricane season, the most active Atlantic hurricane season in recorded history. During the season, 28 storms formed (27 named and one unnamed), surpassing almost all records for storm formation in the Atlantic. More tropical storms, hurricanes, and Category 5 hurricanes formed during the season than in any previously recorded Atlantic season. Figures 7.8 a shows the 2005 Atlantic
storm tracks (inset box represents hurricanes occurring within the simulation period), and the timeline of the 2005 hurricane season.

Hurricane Ophelia, September 6 through 23, 2005, has the most significant impact on the St. Johns River region out of all of the hurricanes included in this time period. Since Hurricane Ophelia lingered along the Florida Atlantic Coast and moved slowly and erratically in a northeasterly direction along the Florida coastline, it caused significant coastal erosion for coastal Florida, Georgia, and South Carolina. Figures 7.8 b presents precipitation and average wind speed at Jacksonville (See Figure 5.6) during the simulation period. Inset boxes in the figure represent the major tropical cyclones and hurricanes that possibly have an affect on the water level changes in the St. Johns River. It is clear from this figure that the magnitudes of wind speeds are related to the period of the tropical cyclones and hurricanes.

It is also noted that there was only one significant rainfall, which was the nearly six inches that fell on June 29 (displayed in Figure 7.8 b) at Jacksonville and clearly did not have a pronounced effect on the water surface elevation as seen in Figure 7.9 and Appendix D. Figure 7.9 displays river level comparison from June 15 to July 15 at Main Street Bridge near Jacksonville: 1) Historical (black line); 2) No-Inflow (red line); and 3) Inflow (blue line).
Figure 7.8: a) The 2005 Atlantic storm tracks and timeline (Wikipedia) and b) precipitation [in] and average wind speed [mph] at Jacksonville during simulation period.
Figure 7.9: River level comparison at Main Street Bridge.

For the long-term simulation, total simulation time is 122 days, with a 5 second time step. The other parameters applied are the same as presented in Chapter 6.1.3 except for the wind forcings where an interval of one hour is used. Figures 7.10 through 7.15 demonstrate the effect of winds on the water levels in September, 2005 at Mayport, I-295 Bridge West End, and Buffalo Bluff, respectively: 1) Historical (black line); 2) No-Wind Forcing (red line); and 3) Wind Forcing (blue line). Other locations and time period results are shown in Appendix D.
Figure 7.10: Water level comparison (September 1 through 15, 2005) at Mayport.

Figure 7.11: Water level comparison (September 16 through 30, 2005) at Mayport.
Figure 7.12: Water level comparison (September 1 through 15, 2005) at I-295 Bridge.

Figure 7.13: Water level comparison (September 16 through 30, 2005) at I-295 Bridge.
Figure 7.14: Water level comparison (September 1 through 15, 2005) at Buffalo Bluff.

Figure 7.15: Water level comparison (September 16 through 30, 2005) at Buffalo Bluff.
The effects of wind forcings are noticeable through water level comparison presented in Figures 7.10 through 7.15. By including wind forcings in the simulation, the model clearly results in a better performance than when wind forcings are neglected, with a more appreciable effect on the timing of the water level rising and at the peak. Slight under-prediction is continued after the peak; however, including wind forcings still produce better results than without wind forcings. Thus, it is determined that the reason of large discrepancy between the historical and the model water levels is attributed to the meteorological influences.

Owing to the long-term simulation, remarkable results reveal that the wind forcings (wind and pressure) have a strong effect on the water level changes in the St. Johns River. Including the wind forcings provides a more consistent fit with the historical NOS data than for when wind forcings are neglected. These meteorological effects may be further enhanced due to the lack of topographical features surrounding the St. Johns River, which results in more direct influence of the winds on the water levels within the St. Johns River, namely at Red Bay Point and Buffalo Bluff (see Figure 5.1). Another enhancing effect may be the geometry and profile of the St. Johns River, allowing the winds to blow over wide and flat regions of open water. All of these effects lead us to conclude that the St. Johns River is one of the most susceptible rivers to the influence of meteorological events. Next we will apply the hydrograph boundary conditions that incorporate a global-scale model result into a local-scale model as a spatially and temporally variable boundary condition in order to examine whether they improve the Pseudo-Operational model results or not.
This chapter examines that the hydrograph boundary conditions produced by the WNAT-SJR model are properly applied to the Pseudo-Operational model. Previous research (Blain et al. 1994a; Salisbury et al. In Press) has indicated that domain size affects the hurricane storm surge simulation significantly and that incorporating hydrograph boundary conditions (produced by a global-scale domain) into the local-scale domain improves the model results substantially. Because the large-scale domain captures more of the dynamic behavior associated with hurricane storm surge set-up which travels from the deep-ocean toward the shoreline. The results from the WNAT-SJR model, the Pseudo-Operational model, and the Pseudo-Operational model with the hydrograph boundary conditions produced by WNAT-SJR model are compared to explore whether the previous research finding are applicable to the St. Johns River region.

Figures 7.16 to 7.21 shows the water level comparison applying two domain sizes and the hydrograph boundary conditions at Mayport, I-295 Bridge West End, and Buffalo Bluff, respectively: 1) Historical (black line); 2) Pseudo-Operational (red line); 3) WNAT-SJR (blue line); and 4) Pseudo-Operational (Hydro) (green line). It is revealed that the WNAT-SJR model produces better results than the Pseudo-Operational model. Furthermore, applying hydrograph boundary conditions improves the Pseudo-Operational model results, to produce water levels more in line with those generated by the WNAT-SJR model. As the results show, it is seen that the hydrograph boundary conditions are adaptable to the Pseudo-Operational model in the St. Johns River region.
Figure 7.16: Water level comparison (September 1 through 15, 2005) at Mayport.

Figure 7.17: Water level comparison (September 16 through 30, 2005) at Mayport.
Figure 7.18: Water level comparison (September 1 through 15, 2005) at I-295 Bridge.

Figure 7.19: Water level comparison (September 16 through 30, 2005) at I-295 Bridge.
Figure 7.20: Water level comparison (September 1 through 15, 2005) at Buffalo Bluff.

Figure 7.21: Water level comparison (September 16 through 30, 2005) at Buffalo Bluff.
Model reliability and robustness are assessed by performing the 122-day simulation. In addition, some physical features are found by performing sequential simulations: 1) importance of time and advective terms in the GWCE when the ADCIRC simulates narrow regions (e.g., river and channel) where highly advective flows are dominated; 2) inflows from tributaries and precipitation are a secondary factor that increases the river levels in the St. Johns River because fluctuations of water levels is unlikely to relate to the precipitation (See Figure 7.9); 3) The highest priority factor of fluctuations of high water levels in the St. Johns River is meteorological effects, wind and pressure. Furthermore we can confirm that the hydrograph boundary condition can successfully apply to the Pseudo-Operational model. This indicates that we can save computational memory and time without neglecting the large-scale effects of storm surges if we have a large-scale model providing boundary conditions to the shelf-based model.

The sensitivity analysis of drag coefficient can not be carried out for the 122-day simulation because wind data provided by Oceanweather, Inc. have already converted to the wind stresses (using Garratt formulation). Thus, the following chapters we focus on one particular event that is Hurricane Floyd (1999) to undertake some sensitivity analysis by performing the same processes as the 122-day simulation.

7.1.5 Hurricane Floyd Wind Forcings Verification

The Pseudo-Operational model is also verified through the water level comparison including the Hurricane Floyd wind forcings. Since Hurricane Floyd moves along the Florida Atlantic Coast
without making landfall, it’s an ideal case that wind-induced waves have an important role in the storm tides. The historical NOS data is used to evaluate the model results at Fernandina Beach, Mayport, St. Augustine Beach, and Wekala where the data is available in 1999 (See Figure 7.2). Simulation time is 4.75 days (September 12, 1999 at 0:00 through September 16, 1999 at 18:00). Seven tidal constituents derived from WNAT-SJR are provided at the open-ocean boundary. The river inflows and wind forcings are read into the simulation every 30 minutes, and water levels are output every six minutes. Figures 7.22 to 7.25 represent the water level comparison based on the Hurricane Floyd wind forcings: 1) Historical (black line); 2) No-Wind Forcing (red line); and 3) Wind Forcing (blue line).

![Figure 7.22: Water level comparison based on the wind forcings at Fernandina Beach.](image)
Figure 7.23: Water level comparison based on the wind forcings at Mayport.

Figure 7.24: Water level comparison based on the wind forcings at St. Augustine Beach.
Figures 7.22 to 7.25 display the water level comparison regarding to the Hurricane Floyd wind forcings at the NOS tide gauge locations. Other location results are shown in Appendix D. The wind forcings raise the water level at all NOS tide gauge locations. It produces an approximately 50-cm higher peak water level than for when wind forcings are not considered. The peak storm surge level, for when wind forcings are considered, well fits the historical storm surge level at Mayprot and Wekala. However, even when wind forcings are considered, there is a consistent under-prediction of the water level with respect to the historical data.

This under-prediction could be attributed to select inappropriate numerical parameters (e.g., bottom friction coefficient and drag coefficient) and domain sizes or due to the absence of other
physical mechanisms (e.g., short wave set-up and run-up). Next, a sensitivity analysis is undertaken to explore these factors of uncertainty.

### 7.1.6 Sensitivity Analysis

One domain and two parameter specifications are included in the sensitivity analysis: 1) domain size; 2) bottom friction coefficient; and 3) drag coefficient. First, the influence of the domain size is discussed. Again, we examine the hydrograph boundary conditions, where this evaluation of the boundary conditions is performed in the same manner as was done for the previous simulations (See Chapter 7.1.4).

![Figure 7.26: Water level comparison applying two domain sizes and hydrograph boundary conditions at Mayport.](image-url)
Figure 7.26 shows the water level comparison applying two domain sizes and hydrograph boundary conditions at Mayport: 1) Historical (black line); 2) Pseudo-Operational (red line); 3) WNAT-SJR (blue line); and 4) Pseudo-Operational (Hydro) (green line). Other location results are shown in the Appendix D. It is revealed that the WNAT-SJR model produces better results than the Pseudo-Operational model as well as the previous case (See Chapter 7.1.4). Obviously, applying hydrograph boundary conditions improves the Pseudo-Operational model results, to produce water levels more in line with those generated by the WNAT-SJR model. As the results show, it is evident that the hydrograph boundary conditions are adaptable to all cases of simulation.

Second, the influence of bottom friction is investigated, with particular focus placed on the selection of the minimum bottom friction coefficient. When attempting to select a bottom friction value, many authors (see Table 7.1) suggest beginning with a value of 0.0025 for the coefficient in quadratic formulations. Depending on the bottom profile and other contributing physical factors, the bottom friction coefficient can vary from 0.001 to 0.009. Table 7.1 summarizes the bottom friction values used in several tidal studies reported in journal articles (Murray 2003). In this analysis, three minimum friction coefficients (0.001, 0.0025, and 0.004) in the hybrid bottom friction formulation are used to observe the effect of bottom friction on the water levels in the St. Johns River. It is noted that the Pseudo-Operational model is used in the following analysis.
Table 7.1: Bottom friction values from various tidal studies.

<table>
<thead>
<tr>
<th>Investigator (s)</th>
<th>Bottom Friction Coefficient</th>
<th>Location of Model Domain</th>
<th>Model Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Westerink et al. (1992)</td>
<td>0.03</td>
<td>Gulf of Mexico</td>
<td>ADCIRC</td>
</tr>
<tr>
<td>Kolar et al. (1994)</td>
<td>0.003</td>
<td>Western North Atlantic</td>
<td>ADCIRC</td>
</tr>
<tr>
<td>Tomasson &amp; Eliasson (1995)</td>
<td>0.004</td>
<td>Northern Atlantic Ocean</td>
<td>AQUASEA</td>
</tr>
<tr>
<td>Blain (1997)</td>
<td>0.0015</td>
<td>Arabian Gulf</td>
<td>ADCIRC</td>
</tr>
<tr>
<td>Hagen &amp; Bennett (1999)</td>
<td>0.003</td>
<td>South Carolina</td>
<td>ADCIRC</td>
</tr>
<tr>
<td>Wilson et al. (2002)</td>
<td>0.01</td>
<td>Artificial Channel</td>
<td>TELEMAC-2D</td>
</tr>
<tr>
<td>Luettich et al. (2002)</td>
<td>0.0025</td>
<td>Albemarle-Pamlico Estuarine System, N.C.</td>
<td>ADCIRC</td>
</tr>
</tbody>
</table>

Figure 7.27: Water level comparison with various bottom frictions at Mayport.
Figures 7.27 and 7.28 represent the water level comparison with various bottom frictions at Mayport and Fernandina Beach: 1) Historical (black line); 2) BF_0.001 (red line); 3) BF_0.0025 (blue line); and 4) BF_0.004 (green line). Other location results are shown in the Appendix D.

The minimum bottom friction coefficient, 0.001, produces the highest peak storm surge level at all of the NOS tide gauge locations. As the minimum bottom friction coefficient increases, the water level decreases gradually. Selecting lower bottom friction produces an additional water level raising not only the peak storm surge level but also the falling limbs. At Fernandina Beach, results from bottom friction coefficient, 0.001, are an approximately 30-cm higher than others. It is apparent from these results that varying the minimum bottom friction coefficient produces different peak storm surge levels for the hurricane events.
Third, the influence of the drag coefficient, $C_D$, is examined. Previous research (Weaver et al. 2004) has pointed out that the resulting surge prediction can vary as much as $\pm 0.5$ meters depending on the choice of drag coefficient formulation used. Table 7.2 shows the drag coefficient formulation used in a variety of studies (Weaver et al. 2004). The ADCIRC developers recommend using the Garratt’s formulation with putting a cap on the maximum value at $C_D = 0.003$. In this analysis, three drag coefficients (Miller, Garratt, and Yelland & Taylor) are used to assess their contributions in the reproduction of water levels. The parameters are as same as the presented in Chapter 6.1.3.

<table>
<thead>
<tr>
<th>Investigator (s)</th>
<th>$C_D \times 10^{-3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Miller (1964)</td>
<td>4.0</td>
</tr>
<tr>
<td>Garratt (1977)</td>
<td>$0.75 + 0.067</td>
</tr>
<tr>
<td>Smith (1980)</td>
<td>$0.61 + 0.063</td>
</tr>
<tr>
<td>Large &amp; Pond (1981)</td>
<td>$0.44 + 0.063</td>
</tr>
<tr>
<td>Klapstov (1983)</td>
<td>$0.49 + 0.07</td>
</tr>
<tr>
<td>Geernaert (1987)</td>
<td>$0.58 + 0.085</td>
</tr>
<tr>
<td>Yelland &amp; Taylor (1996)</td>
<td>$0.29 + \frac{3.1}{</td>
</tr>
</tbody>
</table>

1. $|W|$ is the magnitude of wind velocity at 10 m height above the mean sea level.
Figure 7.29: Water level comparison with several drag coefficients at Mayport.

Figure 7.30: Water level comparison with several drag coefficients at Wekala.
Figure 7.29 and 7.30 show the water level comparison with several drag coefficients at Mayport and Wekala: 1) Historical (black line); 2) Miller (red line); 3) Garratt (blue line); and 4) Yelland & Taylor (green line). Other location results are shown in the Appendix D. It is apparent that Miller formulation, the oldest formulation and independent of the wind velocity, provides a better level of fit relative to the other two formulations, for the tide gauge locations found along the Florida Atlantic Coast. On the other hand, Miller formulation results in a large over-prediction at Wekala and other tide gauge locations found inside the St. Johns River (See Appendix D).

Leading up to the highest peak, the results associated with the use of Miller formulation well captures the peaks and troughs of the water levels relative to the historical NOS data; however, the falling limbs are under estimated when compared to the applications of the other two formulations. This implies that the drag coefficient has a strong influence on the peaks and troughs (prior to the highest peak) reproduced by the model. Yet, Miller formulation results in a significant deviation from the historical NOS data inside St. Johns River (See Appendix D). It is evident from these results that the drag coefficient formulation used has a great impact on the water level.

Several remarkable results are obtained through the sensitivity analysis. First, the WNAT-SJR model performs better than the Pseudo-Operational model for the hurricane storm surge simulation; however, applying the hydrograph boundary conditions can make up for this deficiency in the Pseudo-Operational model. Second, the peak storm surge level is dependent on
the bottom friction formulation that is used in the simulations. As the minimum bottom friction coefficient decreases, the peak storm surge level increases. Third, the drag coefficient has an effect on the peaks and troughs generated prior to the highest peak level. Wind forcings that consider the wind velocity in the drag coefficient formulation slightly underestimate the storm surge level.

Consequently, it is verified that the model results are subject to the model domain and parameters used and may be readily fitted (to the historical data) through adjustments of the domain size and model parameters. However, some other physical phenomena associated with the hurricane storm tides (i.e., wind-induced waves) are not described in the simulations presented thus far. Thus, the Pseudo-Operational model simulations that follow will employ the fixed parameters, 0.0025 for minimum bottom friction coefficient, and Garratt formulation for the drag coefficient, as previously mentioned in Chapter 6. It should be emphasized that the following simulation results might be changed by applying the different parameters.
7.2 The Uni-Coupling Model Simulation

This section examines the contributions of the wind-induced waves (wave radiation stresses) on the storm tides through a uni-coupling of the ADCIRC and SWAN models. First, a uni-coupling procedure is described as the SWAN model providing wave radiation stresses to the ADCIRC model. Second, the uni-coupling model results are compared with the non-coupling model results along Florida Atlantic Coast (Fernandina Beach, Mayport, and St. Augustine Beach). Finally, a brief sensitivity analysis, testing different boundary condition and choosing different modes in the SWAN, is carried out.

7.2.1 The Uni-Coupling Procedure

Coupling of tides, surges and waves is incorporated into the simulation by the uni-coupling of SWAN with ADCIRC. Figure 7.31 shows the diagram of the uni-coupling procedure. First, the SWAN model computes the significant wave height, peak wave period, and wave direction with Hurricane Floyd wind and boundary condition provided by the global WAM model. Second, the wave radiation stresses, based on linear wave theory as described in Chapter 4, are calculated with computed significant wave height, peak wave period, and wave direction and then interpolated onto the computational nodes of the ADCIRC mesh (fort.23) by applying a linear interpolation technique at the SWAN-ADCIRC interface. It is noted that if the ADCIRC domain is off the SWAN domain, the wave radiation stresses produced by the global WAM model are interpolated onto the computational nodes of the ADCIRC mesh. Finally, the ADCIRC model
computes the surface water levels and currents with the following forcings: 1) astronomical tides; 2) river inflows; 3) wind forcing (wind and pressure); and 4) wave radiation stresses. There is no feedback from ADCIRC to SWAN, leading to a one-way coupling of the two models. The wave-current interaction will be described by a two-way coupling procedure in the following section.

Figure 7.31: A diagram of uni-coupling SWAN and ADCIRC models.
7.2.2 Wind-Induced Wave Verification

The uni-coupling model is verified through the water level comparison: 1) the historical NOS data; 2) No-Coupling; and 3) Uni-Coupling. The historical NOS data is used to evaluate the model results at Fernandina Beach, Mayport, and St. Augustine Beach. Simulation time is 4.75 days (September 12, 1999 at 0:00 through September 16, 1999 at 18:00). Seven tidal constituents and the significant wave height, peak wave period, and wave direction provided from global WAM model are provided at the open-ocean boundary for ADCIRC and SWAN, respectively. The wave radiation stresses are read into simulation every 2 hours. Although it is possible to provide the wave radiation stresses at every time step, it is computationally too expensive to do so at the present time, justifying our approach to read in these wave forcings every 2 hours.

![Figure 7.32: Water level comparison in non- and uni-couplings at Fernandina Beach.](image)

Figure 7.32: Water level comparison in non- and uni-couplings at Fernandina Beach.
Figure 7.33: Water level comparison in non- and uni-couplings at Mayport.

Figure 7.34: Water level comparison in non- and uni-couplings at St. Augustine Beach.
Figures 7.32 through 7.34 show the water level comparison between non- and uni-couplings at the NOS tide gauge locations. Other location results are shown in Appendix E. The wave radiation stresses produce an additional rise (approximately 10-15% increase) in the peak water level at all locations. The peak storm tide level more closely matches the historical storm tide level at Mayport, Fernandina Beach, and St. Augustine Beach when uni-coupling is employed. In addition, wave radiation stresses improve the falling limbs after the peak. Consequently, it is apparent that the wind-induced waves contribute to the rise in water level for the Hurricane Floyd storm tide simulation.

7.2.3 Sensitivity Analysis

Additional verifications are performed through a sensitivity analysis, validations of the boundary conditions obtained from the WAM model and of the different modes used in SWAN for the uni-coupling model. First, a different boundary condition obtained from the nesting SWAN simulation is used in the uni-coupling simulation for the sake of checking the accuracy of the boundary conditions obtained from the global WAM model. Figure 7.34 represents the domain used for the nested SWAN simulation. The meso-scale finite difference grid with spacing of 0.02°, from 79° W to 81.5° W in longitude and from 29° N to 31.5° N in latitude, is employed to produce the different boundary condition. A 0.1 m wave height is supplied as the boundary condition for the meso-scale simulation. The same Hurricane Floyd winds are used in this simulation. It is noted that although SWAN is inappropriate for computing in the deep ocean, it is deemed acceptable for a simulation of this kind and for the purpose of comparison.
Figure 7.35: Nested SWAN domain.

Figure 7.36: Water level comparison using different boundary conditions at Mayport.
Figure 7.36 shows the water level comparison using different boundary conditions at Mayport: 1) Historical (black line); No-BC (no boundary condition in SWAN, red line); Global WAM (blue line); and Nesting-SWAN (green line). Other location results are shown in Appendix E. The absence of boundary conditions causes an under-prediction of the peak storm tide level at all locations. Slight changes are recognized at the highest peak and lowest trough after measuring the highest peak; however, it is apparently similar at all locations. Thus, it is necessary to provide the boundary condition for the local wave model and of adequate accuracy to employ the boundary condition from the global WAM model to the coupling model.

Second, the different modes in SWAN are employed to examine how the performance of SWAN has an affect on the water level changes in the ADCIRC simulation. SWAN has two types of simulation modes; one is a stationary mode, the other is a non-stationary mode. The stationary mode uses no time marching and iterative procedures while the non-stationary model employs time marching and no iterative procedures. The stationary mode is used for waves with a relatively short residence time in the computational area (wave boundary conditions and storm surge). The non-stationary mode is also used for storm surge and seasonal wave simulation (Holthuijsen et al. 2004). Thus, these modes are applicable to couple with the ADCIRC model to evaluate whether the SWAN simulation changes the water level rising in the uni-coupling model.
Figure 7.37: Water level comparison applying the different modes in SWAN at Mayport.

Figure 7.37 shows the water level comparison by applying different modes in SWAN at Mayport: 1) Historical (black line); 2) Stationary (red line); and 3) Non-Stationary (blue line). Other location results are shown in Appendix E. Simulation conditions of the non-stationary mode are the same as presented in Chapter 6.2 except for time step, where a 5 second time step is applied.

Although applying different modes in SWAN makes slight difference at peaks and troughs, water levels are largely similar regardless of the stationary and non-stationary modes (See Appendix E). But the most different thing is the total computational time in the SWAN simulation. The total computational time of the non-stationary simulation with 5 second time
step takes five times more than ones of the stationary simulation. It is possible to extend the time step as large as possible; however, the time step in SWAN depends on the grid resolution. The time step in SWAN should be small enough to resolve the time variations of the computed wave field itself. Therefore, from the aspect of the computational time, applying the stationary mode is preferable to employing the non-stationary mode for the uni- and coupling model.

Consequently, the contributions of the wind-induced waves (wave radiation stresses) in the St. Johns River are revealed through the uni-coupling of the SWAN and ADCIRC models presented herein. Although these wind-induced waves contribute only 10 – 15% to the generation of the peak storm tides, computed peak storm tide levels coincide well with the historical NOS data for when these wind-induced waves are considered. It is evident that the influence of the wind-induced waves can not be ignored in the simulation of the hurricane storm tides. In addition, the influences of the SWAN simulation on the uni-coupling model are verified through the sensitivity analysis. We can confirm the importance of the boundary condition and difference of the simulation modes in SWAN. Next, we will explore the more complicated behavior of the storm tides by describing the complete wave-current interaction through the full coupling of the SWAN and ADCIRC models.
7.3 The Coupling Model Simulation

This final section analyzes the complex mechanisms of wave-current interaction in the hurricane storm tides through a full coupling of the ADCIRC and SWAN models. First, a coupling procedure that allows for interaction between the SWAN and ADCIRC models is explained. Second, the coupling model results are compared with the non-coupling and uni-coupling model results along Florida Atlantic Coast (Fernandina Beach, Mayport, and St. Augustine Beach). Finally, a brief sensitivity analysis, examining exchange times and hydrograph boundary condition implementations, is conducted.

7.3.1 The Coupling Procedure

Interactions of tides, surges and waves are incorporated into the simulation by the coupling of SWAN and ADCIRC. Figure 7.38 describes the methodology of the coupling. First, the SWAN model computes the significant wave height, peak wave period, and wave direction with winds from Hurricane Floyd and boundary condition produced by the global WAM model. Second, the wave radiation stresses computed with these values are interpolated onto the computational nodes of the ADCIRC mesh at the SWAN-ADCIRC interface. It is noted that if the ADCIRC domain is off the SWAN domain, the wave radiation stresses produced by the global WAM model are interpolated onto the computational nodes of the ADCIRC mesh. Third, the ADCIRC model computes the surface water levels and currents with the following forcings: 1) astronomical tides; 2) river inflows; 3) wind forcing (wind and pressure); and 4) wave radiation
stresses. After two hours of execution, the ADCIRC model provides surface water levels and currents to the computational nodes of the SWAN grid by applying an inverse weighted interpolation method at the SWAN-ADCIRC interface (Zundel 2005). Fourth, incorporating the surface water levels and currents from the ADCIRC model, the SWAN model re-computes the significant wave height, peak wave period, and wave direction again. These same routines (1-4) are repeated until the end of the ADCIRC model simulation. The wave-current interactions are captured by exchanging the wave radiation stresses for the water levels and currents.

Figure 7.38: The methodology of the coupling of SWAN and ADCIRC models.
7.3.2 Wave-Current Interaction Verification

The coupling model is verified through the water level comparison: 1) the historical NOS data; 2) Non-Coupling; and 3) Uni-Coupling; and 4) Coupling. The historical NOS data is used to evaluate the model results at Fernandina Beach, Mayport, and St. Augustine Beach. Simulation time of the ADCIRC model is 4.75 days (September 12, 1999 at 0:00 through September 16, 1999 at 18:00). The SWAN model performs at every exchange time. Seven tidal constituents and the significant wave height, wave period, and wave direction provided from global WAM model are provided at the open-ocean boundary for ADCIRC and SWAN, respectively. The exchange time is set to 2 hours.

![Figure 7.39: Water level comparison among three models at Fernandina Beach.](image)
Figure 7.40: Water level comparison among three models at Mayport.

Figure 7.41: Water level comparison among three models at St. Augustine Beach.
Figures 7.39 through 7.41 show the water level comparison among the three models at the NOS tide gauge locations: 1) Historical (black line); 2) Non-Couling (red line); 3) Uni-Coupling (blue line); and 4) Coupling (green line). Other location results are shown in Appendix E. Three intriguing results are recognized from these figures. First, the coupling model produces lower peak storm tide level than the uni-coupling model at all locations. Second, the coupling model produces higher ebb tides than the other two models. Third, the coupling model produces slight oscillations in the hydrograph.

A considerable reason that the coupling model produces lower peak storm tide level than the uni-coupling model is that the output of the significant wave height, peak wave period, and wave direction in the SWAN simulation is changed by including tidal currents and surface water levels, and as the result, the wave radiation stresses are also changed. The magnitude of the wave radiation stresses at the peak storm tide level is weakened as tidal currents and surface water levels have a reducing effect on the significant wave height, which also results in the production of higher ebb tides. On the contrary to the peaks, wave radiation stresses increase at ebb tides owing to the tidal currents and surface water levels. The wave-current interactions demonstrate these phenomena through the coupling of the SWAN and ADCIRC models.

The oscillations in the coupling model are due to the implementation of hot starts when the coupling simulations are performed by restarting the ADCIRC model for each iteration. Thus, the hot starts are responsible for the slight difference that can be observed between the uni-coupling and coupling models and do not reflect a physical influence.
7.2.3 Sensitivity Analysis

A sensitivity analysis is conducted in order to explore other influences acting on the wave-current interaction: the exchange time that communicates with each model and validation of the hydrograph boundary conditions in the coupling model. First, three exchange times (of one, two, and four hours) are employed to investigate the effect of exchange time on the model results. It should be noted that it is possible to interact the SWAN model with the ADCIRC model at every time step, but it is too computationally expensive to do so. Figure 7.42 shows the water level comparison at Mayport for the three exchange times used: 1) Historical (black line); 2) One-Hour (red line); 3) Two-Hour (blue line); and 4) Four-Hour (green line). Other location results are shown in Appendix E.

Figure 7.42: Water level comparison used several exchange times at Mayport.
Slight amplitude swinging can be recognized in the three model curves shown in Figure 7.42, where these slight swings arise from the restarting of ADCIRC at each loop. However it is apparent that all of three results are similar regardless of the exchanging time. Consequently, the exchange time is relatively insignificant to the wave-current interaction for the hurricane storm tides; however, an appropriate value for the exchange time should be based on the total simulation time and user demand.

In addition, it is noted that the coupling model can incorporate the hydrograph boundary conditions in the hurricane storm tide simulation, where this evaluation of the boundary conditions is performed in the same manner as was done for the previous simulations (see Chapter 7.1.4). Figure 7.43 presents the water level comparison by applying the hydrograph boundary conditions in the uni-coupling and coupling model at Mayport: 1) Historical (black line); 2) Coupling; 3) Uni-Coupling (Hydrograph BC); and 4) Coupling (Hydrograph BC). Other location results are shown in Appendix E.
The results indicate that applying the hydrograph boundary conditions improves the peak storm tide level, to provide a level of fit that is more consistent with the historical NOS data for the gauge stations located at Mayport and St. Augustine Beach. Furthermore, the model results fit the historical NOS data reasonably well at the trough after the highest peak. As the results show, the uni-coupling and coupling models can properly incorporate the hydrograph boundary conditions in the Hurricane Floyd storm tide simulation.

In sum, the wave-current interaction is more completely captured through the full coupling of the SWAN and ADCIRC models, to arrive at three major findings: 1) decreasing peaks and increasing troughs in the storm tide hydrograph result from the wave-current interaction; 2) the
exchange time of interaction of waves and currents is rather insignificant in describing the coupled process; and 3) the hydrograph boundary conditions are applicable to the uni-coupling and coupling models. As the results show, the coupling, including the uni-coupling, of the ADCIRC and SWAN models results in a better performance than the non-coupling model in the Hurricane Floyd storm tide simulation. Furthermore the uni-coupling and the coupling models produce relatively similar results.

Figure 7.44 represents the maximum storm tide contours with the coupling model around Mayport. Approximately 1.5 m maximum storm tide levels are computed along the coastline between Mayport and St. Augustine Beach. It is apparent that results reflect the historical NOS data observed at St. Augustine Beach (See Figure 7.41). The coupling model also provides good agreement with the historical NOS data, approximately 1.2 m maximum storm tide levels, at Mayport (See Figure 7.40). The coupling model successfully captures, not only the temporal variation, but also the spatial variation in the storm tides near and within the St. Johns River.
7.4 Quantitative Analysis

All of the results presented thus far are statistically analyzed in order to provide quantitative analysis of the simulation output. Statistical methods are used to assist with the process of calibration largely because they help quantify the goodness-of-fit between observed and simulated data (Reckhow et al. 1990). The following two statistical methods are used herein: 1) calculation of median relative error and 2) calculation of root mean square error.
Given a time-series of observed values, \( O = o_1, o_2, \cdots, o_n \), and simulated values, \( S = s_1, s_2, \cdots, s_n \), the relative error defines a new time series with the \( ith \) term defined as \( |o_i - s_i| / o_i \). The median value of the time-series of relative errors defines the median relative error between observed and simulated values. This statistic is multiplied by 100 to express the error measure as a percent.

Root mean square error is defined as: \( RMS = \sqrt{\frac{\sum (O_i - S_i)^2}{N}} \), in which \( N \) is the number of samples (in this analysis \( N = 660 \)), \( O_i \) and \( S_i \) are observed and simulated values, respectively. All model results (non-coupling with uni-coupling, non-coupling with hydrograph boundary condition, and uni-coupling with hydrograph boundary condition) are summarized in Table 7.3.

Several observations are noted from the quantitative analysis presented in Table 7.3. First, both uni-coupling and coupling models produce results more in line with the historical data than the non-coupling model, for the gauge stations located along the Florida Atlantic Coast. This indicates that the contribution of the wind-induced wave is significant to include in the storm tide simulations on an RMS- and median relative-error basis. Second, the hydrograph boundary conditions improve all results except for Wekala, where 5 – 20% of the median relative errors are improved without the hydrograph boundary conditions. However, applying the hydrograph boundary conditions increases the level of error associated with the results at Wekala. This could be attributed to delay wave propagations or caused by phase errors. Finally, it is suggested that results from the uni-coupling with the hydrograph boundary conditions are most desired along the coastline.
Table 7.3: Qualitative analysis comparison.

<table>
<thead>
<tr>
<th>Station</th>
<th>Conditions</th>
<th>RMS Error [m]</th>
<th>Median Relative Error [%]</th>
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<tr>
<td>Fernandina Beach</td>
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<td>0.481</td>
<td>39.2</td>
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<td></td>
<td>Uni-Coupling</td>
<td>0.446</td>
<td>34.8</td>
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<tr>
<td></td>
<td>Coupling</td>
<td>0.479</td>
<td>38.4</td>
</tr>
<tr>
<td></td>
<td>Non-Coupling w/ Hydro</td>
<td>0.480</td>
<td>35.8</td>
</tr>
<tr>
<td></td>
<td>Uni-Coupling w/ Hydro</td>
<td>0.430</td>
<td>28.4</td>
</tr>
<tr>
<td></td>
<td>Coupling w/ Hydro</td>
<td>0.476</td>
<td>33.1</td>
</tr>
<tr>
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<td>Non-Coupling</td>
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<td>52.5</td>
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<tr>
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<td>42.5</td>
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<tr>
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<td></td>
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<td>34.2</td>
</tr>
<tr>
<td></td>
<td>Coupling w/ Hydro</td>
<td>0.331</td>
<td>36.6</td>
</tr>
<tr>
<td>St. Augustine Beach</td>
<td>Non-Coupling</td>
<td>0.713</td>
<td>57.1</td>
</tr>
<tr>
<td></td>
<td>Uni-Coupling</td>
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<tr>
<td></td>
<td>Coupling</td>
<td>0.709</td>
<td>52.3</td>
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<tr>
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<td>42.8</td>
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<tr>
<td></td>
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<td>32.0</td>
</tr>
<tr>
<td></td>
<td>Coupling w/ Hydro</td>
<td>0.468</td>
<td>34.1</td>
</tr>
<tr>
<td>Wekala</td>
<td>Non-Coupling</td>
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<td>Coupling w/ Hydro</td>
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7.5 Creation of the Best Hydrograph

To conclude this chapter, an attempt is made to create the most qualitatively accurate hydrograph by properly adjusting the parameters based on the previous verification and calibration. This approach attempts to create the best hydrograph along the Florida Atlantic Coast; thus, we ignore effects inside the river. The uni-coupling with hydrograph boundary conditions is applied because of the best performance in the quantitative analysis. Furthermore, 0.0025 for minimum bottom friction coefficient and Miller formulation (See Table 7.2) for drag formulation are employed. The historical NOS data are used to evaluate three hydrographs: 1) the uni-coupling using Garratt formulation with hydrograph B.C.; 2) the non-coupling using Miller formulation; and 3) the uni-coupling using Miller formulation with hydrograph B.C..

![Figure 7.45: Water level comparison in three hydrographs at Fernandina Beach.](image)

Figure 7.45: Water level comparison in three hydrographs at Fernandina Beach.
Figure 7.46: Water level comparison in three hydrographs at Mayport.

Figure 7.47: Water level comparison in three hydrographs at St. Augustine Beach.
Figures 7.45 to 7.47 display water level comparison in three hydrographs corresponding to three tidal gauge locations along the Florida Atlantic Coast: 1) Historical (black line); 2) Uni-Coupling (Garratt, Hydro); 3) Non-Coupling (Miller); and 4) Uni-Coupling (Miller, Hydro). The hydrographs presented in Figures 7.45 to 7.47 clearly indicate that the uni-coupling using Miller formulation with the hydrograph boundary conditions performs the best level of fit relative to the historical NOS data unto and including the peak; however, the Garratt formulation fits better after the peak is achieved. The Miller formulation results in more consistent tidal signal, peak and trough (prior to the highest peak) with the historical NOS data. However, the Miller formulation still results in under-estimation at the trough after the highest peak. This implies that the formulation of the drag coefficient has a strong influence.

As a result of this approach, we rediscover two facts. First, the wind-induced waves increase the peak storm tide level and the trough after the highest peak. Second, the drag coefficient formulation in the wind stress formulation has an affect on the peak and troughs generated before and after the highest peak. In addition, since large discrepancies occurred inside the river, we surmise that the drag coefficient would be spatially variable in the domain regardless of the wind speed. This is one of a speculation; therefore, further research is needed for determining the influence of the drag coefficient. Overall, the best level of fit for the hydrograph is created by localizing the target area, including a uni-coupling of waves with hydrodynamics, and adjusting the drag coefficient and its formulation, which indicates that on the larger scale the physics are not being represented in their entirety.
CHAPTER 8
CONCLUSION AND FUTURE WORK

Presented in this dissertation is a numerical simulation focusing on the development of a model for the St. Johns River in order to predict flows, tides (astronomical and meteorological) and waves and the investigation of the contribution of the wind-induced waves on the hurricane storm tides. Initially, the ADCIRC model alone is applied in order to assess the validity of using ADCIRC to simulate long-wave phenomena related to hurricane events. Furthermore, the contributions of the wind-induced waves to the storm tides are examined by a coupling of the hydrodynamic and wave models in a one-way direction (uni-coupling). Finally, the coupling of the hydrodynamic and wave models investigates the wave-current interaction in the storm tides. The conclusions from this study have implications toward the refinement of storm tide modeling.

8.1 Conclusions

Research performed in this dissertation accomplished the development of a coupling model interface, which permitted not only the wave-current interaction but also the non-linear relationship among tides, wind surges and wind-induced waves. Although existing ADCIRC-2DDI and SWAN models were used, the coupling interface was a new contribution as well as a code to compute wave radiation stresses. In addition, the coupling interface incorporates linear and inverse distance weighted interpolation schemes for wave to hydrodynamic and hydrodynamic to wave, respectively.
Five primary conclusions are drawn from the numerical simulations presented in this dissertation: 1) the importance of the time and advective terms in the GWCE; 2) the relative insignificance of the river inflows in the St. Johns River; 3) the significance of local and global meteorological effects on the water level changes in the St. Johns River; 4) spatial and temporal variations of bottom coefficient and drag coefficient; and 5) contributions of the wind-induced wave on the Hurricane Floyd storm tides.

Including the time and advective terms in the GWCE into the simulation plays a significant role on the tidal resynthesis. The regions of narrow river width (e.g., Palakta and Wekala; See Figure 6.1) where highly non-linear flows are dominate could cause local mass imbalances that have a tendency to cause high water level. However, mass balance errors that cause the discrepancies between the historical data and the model results can be minimized by including the time and advective terms in the GWCE into the simulation. Thus, it is important that the time and advective terms in the GWCE are employed to the local-scale simulation.

The river inflows from tributaries are relatively insignificant to the fluctuation of river levels in the St. Johns River. Two considerable reasons arise: one is the St. Johns River is mostly surrounded by large marsh areas; therefore precipitation quickly permeates to ground and becomes the ground water; other is an inappropriate streamflow data and inflow condition; however, presented in Chapter 7.1.2 and 7.1.3 confirms that the effect of the inflows are minimal on the river level changes. Thus, it would be concluded that the river inflow is a secondary factor of the fluctuation of river levels in the St. Johns River.
The effect of local and global meteorological forcings is significant for the St. Johns River to predict the water level changes. Comparing model output for when wind forcings are considered and for when wind forcings are neglected in the short- and long-term simulations indicates that including the wind forcings provides a model response that is more consistent with the historical NOS data than for when the wind forcings are neglected. This is a significant finding because it is shown that river inflows, which were once regarded as the important factors of river-stage variations, contribute relatively little to water level changes in the St. Johns River during high water levels. Taking this into account, it is necessary for a future operational St. Johns River model to incorporate meteorological forcings in order to forecast its river levels more accurately.

It is clear from the sensitivity analysis presented in Chapter 7.1.6 that the bottom friction and drag coefficients should be varied, not only spatially, but also temporally. Peak and trough levels prior to the highest peak level are strongly related to the drag coefficients used for the regions found along the Florida Atlantic Coast. Furthermore, the peak storm surge level also varied with choosing different bottom friction coefficients. Particularly, the discrepancy of the peak levels prior to the highest peak could be attributed to the absence of some physical mechanisms that convert wind and pressure to wind forcing in ADCIRC. Consequently, these coefficients should be varied spatially and temporally in order to enhance future simulations.

Another significant finding in this study is the contribution of the wind-induced waves on the hurricane storm tides. Depending on the coupling procedure used, the peak storm tide level and trough (after the peak storm tide level) is improved, 10 – 15% higher than the non-coupling
model, to provide a better level fit with the historical NOS data. Although the wind-induced waves compose a relatively small percentage of the storm tide generation, it should not be ignored because the wind-induced waves may be the cause for levees and riverbank failures. Thus, regardless of the coupling procedure used, the effect of the wind-induced waves should be included in future hurricane storm tide simulations.

Lastly, the discussion as previously mentioned in the Chapter 3 warrants consideration. First, a slight difference can be recognized between the uni-coupling and coupling models at the troughs and peak storm tide levels due to wave-current interactions; however, overall, both models result in a better performance than for when coupling is not considered. The uni-coupling with the hydrograph boundary conditions results in the most accurate performance among these models. From an operational (i.e., getting a solution quickly) point of view, this indicates that the uni-coupling is suitable to coupling the hydrodynamic model with the wave model.

Second, in the absence of same output values used as a reference value in both models, it is impossible to build a convergent criterion to make a fully coupling model. SWAN does not calculate wave-induced currents and have output of water levels; therefore there is no value to use a reference value. Third, depending on the simulation time and user demand, a two-hour exchange time is appropriate in order to provide a sufficient interaction between the two models. Fourth, the linear interpolation method used for the interpolation of data between SWAN and ADCIRC, and the inverse weighted distance method applied for the interpolation of data between ADCIRC and SWAN are shown to be sufficient for the transfer of data in the coupling
procedure. Significant interpolation errors can not be confirmed in the coupling model results. Finally, as previously reported in the sensitivity analysis (see Chapter 7), there is the requirement to pursue sustained efforts towards optimizing parameter selections. This issue will be discussed more heavily in the future work.

8.2 Future Work

Future work will need to focus on a few key areas. A spatially and temporally variable parameterization of bottom friction and drag effect, along with the determination of a new wind stress formulation, may be necessary in order to simulate the hurricane storm tides more accurately. According to the results presented in the sensitivity analysis (see Chapter 7.1.6), varying the bottom friction coefficient produces a noticeable spatial variance in the model results outside/inside St. Johns River; thus, it is necessary to determine a spatially variable bottom friction coefficient in an optimal manner. In addition, it is necessary to determine a temporally variable bottom friction coefficient for applications dealing with coastal erosion. Varying the drag coefficient also results in a remarkable variance in the model results outside/inside St. Johns River; furthermore, the deficiency in the peak water levels prior to the highest peak is closely related to, not only a drag coefficient formulation, but also a wind stress formulation. ADCIRC should consider the physics of the air-sea interaction, including the effects sea and land roughness, similar as to that done by the WAM and SWAM models (see Chapter 4.3.2) in order to develop an optimal wind stress formulation.
APPENDIX A

ADCIRC-2DDI INPUT FILE: MESH DESCRIPTION
The Pseudo-Operational Model Finite Element Mesh

47763  26543
  1  -80.0681729447  26.935906083  13.7944021372
  2  -80.0028147807  26.9591125243  53.1514246596
  3  -79.9402844035  26.9815787738  142.2879586344

\[
\begin{array}{ccc}
\text{This portion of the input has been eliminated} \\
\end{array}
\]

\[
\begin{array}{ccc}
\text{This portion of the input has been eliminated} \\
\end{array}
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\begin{array}{ccc}
\text{This portion of the input has been eliminated} \\
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\begin{array}{ccc}
\text{This portion of the input has been eliminated} \\
\end{array}
\]

1 = Number of open boundaries
53 = Total number of open boundary nodes
53 = Number of nodes for open boundary 1
1
2
3

13549
13593
APPENDIX B

ADCIRC-2DDI INPUT FILE: MODEL PARAMETER
Astronomical Tide Verification 32 CHARACTER ALPHANUMERIC RUN DESCRIPTI
Pseudo-Operational 24 CHARACTER ALPANUMERIC RUN IDENTIFICA
1999091200 – 199909161800 32 CHARACTER ALPHANUMERIC RUN DESCRIPTI
Pseudo-Operational 24 CHARACTER ALPANUMERIC RUN IDENTIFICA

1 ! NFOVER - NONFATAL ERROR OVERRIDE OPTION
1 ! NABOUT - ABREVIATED OUTPUT OPTION PARAMETER
1 ! NSCREEN - OUTPUT TO UNIT 6 PARAMETER
0 ! IHOT - HOT START OPTION PARAMETER
2 ! ICS - COORDINATE SYSTEM OPTION PARAMETER
0 ! IM - MODEL RUN TYPE: 0=2DDI, 1=3DL(VS), 2=3DL(DSS)
2 ! NOLIF - NONLINEAR BOTTOM FRICITION OPTION
2 ! NOLIFA - OPTION TO INCLUDE FINITE AMPLITUDE TERMS
1 ! NOLICA - OPTION TO INCLUDE CONVECTIVE ACCELERATION TERMS
1 ! NOLICAT - OPTION TO CONSIDER TIME DERIVATIVE OF CONV ACC TERMS
0 ! NWP - VARIABLE BOTTOM FRICITION AND LATERAL VISCOSITY OPTION
1 ! NCOR - VARIABLE CORIOLIS IN SPACE OPTION PARAMETER
0 ! NTIP - TIDAL POTENTIAL OPTION PARAMETER
102 ! NWS - WIND STRESS AND BAROMETRIC PRESSURE OPTION PARAMETER
1 ! NRAMP - RAMP FUNCTION OPTION
9.81 ! G - ACCELERATION DUE TO GRAVITY - DETERMINES UNITS
0.006 ! TAU0 - WEIGHTING FACTOR IN GWCE
5.0 ! DT - TIME STEP (IN SECONDS)
0.00 ! STATIM - STARTING SIMULATION TIME IN DAYS
0.00 ! REFTIME REFERENCE TIME FOR NODAL FACTORS AND EQUILIBRIUM
1800 7200 ! WTIMINC - METEOROLOGICAL WIND TIME INTERVAL
4.75000 ! RNDAY - TOTAL LENGTH OF SIMULATION (IN DAYS)
0.05000 ! DRAMP - DURATION OF RAMP FUNCTION (IN DAYS)
0.35 0.30 0.35 ! TIME WEIGHTING FACTORS FOR THE GWCE EQUATION
0.01 2 1 0.05 ! H0, NODEDRYMIN, NODEWETMIN, VELMIN - MINIMUM WATER
DEPTH AND DRYING/WETTING OPTIONS
-79.5 30.2 ! THE CPP COORDINATE PROJECTION IS CENTERED
0.0025 1.0 10.0 0.33 ! FFACCTORMIN, HBREAK, FTHETA, FGAMMA
0.00 ! EVM - SPATIALLY CONSTANT HORIZONTAL EDY VISCOSITY
0.00 ! CORI - CONSTANT CORIOLIS COEFFICIENT
7 ! NTIF - NUMBER OF TIDAL POTENTIAL CONSTITUENTS
K1 ! ALPHANUMERIC DESCRIPTION OF TIDAL POTENTIAL FORCING DA
0.141565 0.000072921158358 0.736 0.932 253.157
O1 ! ALPHANUMERIC DESCRIPTION OF TIDAL POTENTIAL FORCING DA
0.100514 0.000067597744151 0.695 0.890 71.814
M2 ! ALPHANUMERIC DESCRIPTION OF TIDAL POTENTIAL FORCING DA
0.242334 0.000140518902509 0.693 1.025 321.188
S2 ! ALPHANUMERIC DESCRIPTION OF TIDAL POTENTIAL FORCING DA
0.112841 0.000145444104333 0.693 1.000 0.000
N2 ! ALPHANUMERIC DESCRIPTION OF TIDAL POTENTIAL FORCING DA

149
0.046398 0.000137879699487 0.693 1.025 202.964
K2  ! ALPHANUMERIC DESCRIPTION OF TIDAL POTENTIAL FORCING DATA
0.030704 0.000145842317201 0.693 0.834 327.071
Q1  ! ALPHANUMERIC DESCRIPTION OF TIDAL POTENTIAL FORCING DATA
0.019256 0.00064958541129 0.695 0.890 313.590
7  ! NBFR - TOTAL NUMBER OF FORCING FREQUENCIES ON OPEN BOU
K1  ! ALPHANUMERIC DATA FOR OPEN OCEAN FORCING
0.000072921158358 0.932 253.157
O1  ! ALPHANUMERIC DATA FOR OPEN OCEAN FORCING
0.000067597744151 0.890 71.814
M2  ! ALPHANUMERIC DESCRIPTION OF OPEN BOUNDARY FORCING DATA
0.000140518902509 1.025 321.188
S2  ! ALPHANUMERIC DESCRIPTION OF OPEN BOUNDARY FORCING DATA
0.000145444104333 1.000 0.000
N2  ! ALPHANUMERIC DESCRIPTION OF OPEN BOUNDARY FORCING DATA
0.000137879699487 1.025 202.964
K2  ! ALPHANUMERIC DESCRIPTION OF OPEN BOUNDARY FORCING DATA
0.000145842317201 0.834 327.071
Q1  ! ALPHANUMERIC DESCRIPTION OF OPEN BOUNDARY FORCING DATA
0.000064958541129 0.890 313.590
K1  ! ALPHANUMERIC DESCRIPTION OF ELEVATION BOUNDARY FORCIN
  0.0736267 206.6267
  0.0731114 205.6133
::
This portion of the input has been eliminated
::
  0.1023096 185.4990
  0.1028647 185.6091
O1
  0.0540994 224.2146
  0.0534128 223.0488
::
This portion of the input has been eliminated
::
  0.0748745 197.8069
  0.0751726 197.9097
M2
  0.4234556 7.7306
  0.4225926 7.5396
::
This portion of the input has been eliminated
::
  0.6686705 355.0236

150
0.6768420 355.3221
S2
0.0739617 28.3916
0.0737267 28.2895

This portion of the input has been eliminated

0.1181744 13.2754
0.1198141 13.6223
N2
0.0974802 351.5178
0.0973079 351.3138

This portion of the input has been eliminated

0.1532670 342.2512
0.1552358 342.5835
K2
0.0166260 35.8229
0.0165660 35.8063

This portion of the input has been eliminated

0.0266029 19.6527
0.0269709 19.9537
Q1
0.0094799 215.0585
0.0093081 213.8019

This portion of the input has been eliminated

0.0136708 185.4010
0.0137406 185.4913

45  ! ANGINN - MINIMUM ANGLE FOR TANGENTIAL FLOW
0  ! NFFR - NUMBER OF FREQUENCIES IN THE SPECIFIED NORMAL FLOW
1  0.000000 4.750000 72  ! NOUTE, TOUTSE, TOUTFE, NSPOOLE - FORT 61 OPTIONS
18  ! NSTAE - NUMBER OF ELEVATION RECORDING STATIONS, FOLLOWED BY
LOCATIONS ON PROCEEDING LINES
-81.4650 30.6717  ! 8720030 FERNANDINA BEACH, AMELIA RIVER, FL
-81.4133 30.4000  ! 8720211 WWTD, MAYPORT NAVAL STA., ST JOHNS
RIVER, FL

This portion of the input has been eliminated
-81.6750  29.4767  No.219  ! 8720832 WELAKA, ST. JOHNS RIVER, FL
-81.2633  29.8567  No.77  ! 8720587 ST. AUGUSTINE BEACH, ATLANTIC OCEAN, FL
0 0.000000 0.000000 0  ! NOUTV, TOUTSV, TOUTFV, NSPOOLV - FORT 62
0  ! NSTAV - NUMBER OF ELEVATION RECORDING STATIONS
0 0.000000 0.000000 0  ! NOUTM, TOUTSM, TOUTFM, NSPOOLM
0  ! NSTAM - NUMBER OF ELEVATION RECORDING STATIONS
1 0.000000 4.750000 1440  ! NOUTGE, TOUTSGE, TOUTFGE, NSPOOLGE
1 0.000000 4.750000 1440  ! NOUTGV, TOUTSGV, TOUTFGV, NSPOOLGV
0 0.000000 0.000000 0  ! NOUTGM, TOUTSGM, TOUTFGM, NSPOOLGM
0  ! NHARF - NUMBER OF FREQUENCIES IN HARMONIC ANALYSIS
0.0 0.0 0.0 0.0  ! THAS, THAF, NHAINC, FMV - HARMONIC ANALYSIS PARAMETERS
0 0 0 0  ! NHASE, NHASV, NHAGE, NHAGV - CONTROL HARMONIC ANALYSIS
0 0  ! NHSTAR, NHSINC - HOT START FILE GENERATION PARAMETERS
1 0 0.000030 25  ! ITITER, ISLDIA, CONVCR, ITMAX
APPENDIX C

SWAN INPUT FILE: MODEL PARAMETER
$*************HEADING***************************************************************************$

$ PROJ 'ST-TI' 'GWAM'$
$ $
$ MIDDLE DOMAIN LOG -81.5 to -80.5, LAT 29.8 to 31.8, DX = 0.005 deg DY = 0.005 deg$
$ Time of simulation: 9909120000 - 9909161800
$ Time of simulation: 0:00 GMT 9/12/1999 - 18:00 GMT 9/16/1999
$ $
$**********MODEL INPUT****************************************************************************$
$ SET RHO = 1030.
COORD SPHE CCM
$ $ CGRID REG -81.5 29.8 0 1.0 1.0 199 199 CIRCLE 72 0.04 0.33 30
$ $ INPGRID BOTTOM -81.5 29.8 0 200 200 0.005 0.005
READINP BOTTOM 1. 'G500.bot' 1 0 FREE
$ $ INPGRID WIND -81.6 29.8 0. 6 5 0.2 0.2
READINP WIND 1. './WIND/002' 1 0 FREE
$ $ INPGRID WLEVEL -81.5 29.8 0. 199 199 0.005 0.005
READINP WLEVEL 1. './WLEVEL/001' 1 0 FREE
$ $ INPGRID CURRENT -81.5 29.8 0. 199 199 0.005 0.005
READINP CURRENT 1. './CURRENT/001' 1 0 FREE
$ $ BOUN SHAPE JONSWAP 3.3 PEAK DSPR POW
BOUN SIDE E CCW CON PAR 1.284 5.210 239.470 4
$ $********** OUTPUT REQUESTS ********************************************************************************$
$ $ FRAME 'FLOYD' -81.5 29.8 0 1.0 1.0 199 199
OUTPUT BLOCK 6
BLOCK 'FLOYD' XP YP DEPTH HS DIR RTP FORCE
BLOCK 'FLOYD' NOHEAD 'FLOYD' XP YP DEPTH HS DIR RTP FORCE
$ COMPUTE
STOP
$
APPENDIX D

NUMERICAL SIMULATION RESULTS: THE ADCIRC RESULTS
Presented in this appendix shows the numerical results of the ADCIRC simulation at the all NOS gauge locations (See Figure 6.1). The results represents as the following order: 1) astronomical tides comparison; 2) river level comparison; 3) water level comparison depending on the wind forcings; 4) water level compassion applying to two domain sizes and the hydrograph boundary conditions; 5) water level comparison based on the Hurricane Floyd wind forcings; 6) water level comparison applying to two domain sizes and the hydrograph boundary conditions; 7) water level comparison with various bottom friction coefficient; 8) water level comparison with several drag coefficient formulations. It is noted that Figures 3 and 4 are not shown at Fernandina Beach and St. Augustine Beach.
Figure D.1.1: Simulation results (1 – 3, 1-4) at WWTD Mayport Naval Station.
Figure D.1.2: Simulation results (3, 4, 3-6) at WWTD Mayport Naval Station.
Figure D.1.3: Simulation results (3, 4, 7-8 - 8) at WWTD Mayport Naval Station.
Figure D.2.1: Simulation results (1 – 3, 1-4) at Mayport.
Figure D.2.2: Simulation results (3, 4, 3-6) at Mayport.
Figure D.2.3: Simulation results (3, 4, 7-8 - 8) at Mayport.
Figure D.3.1: Simulation results (1 – 3, 1-4) at Dame Point.
Figure D.3.2: Simulation results (3, 4, 3-6) at Dame Point.
Figure D.3.3: Simulation results (3, 4, 7-8 - 8) at Dame Point.
Figure D.4.1: Simulation results (1 – 3, 1-4) at Longbranch (USE-DDP).
Figure D.4.2: Simulation results (3, 4, 3-6) at Longbranch (USE-DDP).
Figure D.4.3: Simulation results (3, 4, 7-8 - 8) at Longbranch (USE-DDP).
Figure D.5.1: Simulation results (1 – 3, 1-4) at Main Street Bridge.
Figure D.5.2: Simulation results (3, 4, 3-6) at Main Street Bridge.
Figure D.5.3: Simulation results (3, 4, 7-8) at Main Street Bridge.
Figure D.6.1: Simulation results (1 – 4, 1-2) at I-295 Bridge, West End.
Figure D.6.2: Simulation results (3, 4, 3-6) at I-295 Bridge, West End.
Figure D.6.3: Simulation results (3, 4, 7-8 - 8) at I-295 Bridge, West End.
Figure D.7.1: Simulation results (1 – 4, 1-2) at Red Bay Point.
Figure D.7.2: Simulation results (3, 4, 3-6) at Red Bay Point.
Figure D.7.3: Simulation results (3, 4, 7-8 - 8) at Red Bay Point.
Figure D.8.1: Simulation results (1 – 4, 1-2) at Racy Point.
Figure D.8.2: Simulation results (3, 4, 3-6) at Racy Point.
Figure D.8.3: Simulation results (3, 4, 7-8 - 8) at Racy Point.
Figure D.9.1: Simulation results (1 – 4, 1-2) at Palakta.
Figure D.9.2: Simulation results (3, 4, 3-6) at Palakta.
Figure D.9.3: Simulation results (3, 4, 7-8 - 8) at Palakta.
Figure D.10.1: Simulation results (1 – 4, 1-2) at Buffalo Bluff.
Figure D.10.2: Simulation results (3, 4, 3-6) at Buffalo Bluff.
Figure D10.3: Simulation results (3, 4, 7-8 - 8) at Buffalo Bluff.
Figure D.11.1: Simulation results (1 – 4, 1-2) at Wekala
Figure D.11.2: Simulation results (3, 4, 3-6) at Wekala.
Figure D11.3: Simulation results (3, 4, 7-8 - 8) at Wekala.
Figure D.12.1: Simulation results (1 – 8) at Fernandina Beach.
Figure D.13.1: Simulation results (1 – 8) at St. Augustine Beach.
APPENDIX E

NUMERICAL SIMULATION RESULTS: THE UNI-COUPLING AND COUPLING RESULTS
Presented in this appendix shows the numerical results of the uni-coupling and coupling simulation at the all NOS gauge locations (See Figure 6.1). The results represents as the following order: 1) water level comparison between non- and uni-coupling; 2) water level comparison using different boundary conditions; 3) water level comparison applying the different modes; 4) water level comparison among three models; 5) Water level comparison used several exchange times; 6) water level comparison by applying the hydrograph BC.
Figure E.1: Simulation results (1 - 6) at WWTD Mayport Naval Station.
Figure E.2: Simulation results (1 – 6) at Mayport.
Figure E.3: Simulation results (1 – 6,) at Dame Point.
Figure E.4: Simulation results (1 – 6) at Longbranch (USE-DDP).
Figure E.5: Simulation results (1 – 6) at Main Street Bridge.
Figure E.6: Simulation results (1 – 6) at I-295 Bridge, West End.
Figure E.7: Simulation results (1 – 6) at Red Bay Point.
Figure E.8: Simulation results (1–6) at Racy Point.
Figure E.9: Simulation results (1 – 6) at Palakta.
Figure E.10: Simulation results (1 – 6) at Buffalo Bluff.
Figure E.11: Simulation results (1 – 6) at Wekala
Figure E.12: Simulation results (1 – 6) at Fernandina Beach.
Figure E.13: Simulation results (1 – 6) at St. Augustine Beach.
LIST OF REFERENCES


Barber, N.F. and F. Ursell (1948)
“The generation and propagation of ocean waves and swell.” *Phil. Trans. R. Soc. Lond.*, A240, 527-609.


Bowden, K.F. (1983)
“Physical oceanography of coastal waters.” *Ellis Horwood Ltd.*

Bryan, K., S. Manabe, and R. C. Pacanowski, (1975)

Cardone, V.J., W.J. Pierson and E.G. Ward (1976)

Cartwright, D.E. (1958)


Charnock, H. (1955)


“Coupled storm surge and wave simulation for neighboring seas of Korean peninsula.” Workshop on Wave, Tide Observations and Modelings in the Asian-Pacific Region.


“Operational systems for the prediction of tropical cyclone generated winds and waves.” Proceeding of 6th International Workshop on Wave Hindcasting and Forecasting. Monterey, California.

“20 years of operational forecasting at Oceanweather.” Proceeding of 7th International Workshop on Wave Hindcasting and Forecasting. Banff, Alberta, Canada.

“Water wave mechanics for engineers and scientists.” *World Scientific Publishing Co. Pte. Ltd.*.


“Storm tide simulations for Hurricane Hugo (1989): On the significance of including inland flooding areas.” *J. of Waterway, Port, Coastal and Ocean Engineering, ASCE.*

Doyle, J.D. (2002)

Fabrice A. (2005)

“A numerical model investigation of tides and diurnal-period continental shelf waves along Vancouver Island.” *J. of Physical Oceanography*, 18, 115-139.


Garrett, J.R. (1977)


“Coastal forecasts and storm surge predictions for tropical cyclones: A timely partnership program.” *Oceanography* Vol. 19, 130-141.

“Finite element grids based on a localized truncation error analysis.” Ph.D. Dissertation Department of Civil Engineering and Geological Science, University of Notre Dame, Indiana.


Hendershott, M.C. (1981)  


Kitaigorodskii, S.A. (1973)  


“Dynamics and modelling of ocean waves.” Cambridge University Press.

Komen, G.J. (2004)
“The wave modeling (WAM) group, a historical perspective.” The 16th BMRC Modelling Workshop.

Le Provost, C. and P. Vincent (1986)


Longuet-Higgins, M.S. (1952)

“Changes in the form of short gravity waves on long waves and tidal currents.” J. Fluid Mech., 8, 565-583.

Longuet-Higgins, M.S. and R.W. Stewart (1964)
”Radiation stresses in water waves; a physical discussion, with applications.” Deep-Sea Research, 11, 529-562, 1964.

“ADCIRC: An advanced three-dimensional circulation model for shelves, coasts, and estuaries, report No. 1,” Theory and methodology of ADCIRC-2DDI and ADCIRC-3DL, technical report DRP-992-6, November 1992, U.S. Army Engineer Waterways Experiment Station, Vicksburg MS.
“Ocean surface wave: Their physics and prediction.” *World Scientific Publishing Co. Pte. Ltd.*.


Miles, J.W. (1957)

Miles, J.W. (1960)


Moon, I.J. (2005)

“A sensitivity analysis for a tidally-influenced riverine system.” Master’s Thesis, Department of Civil and Environmental Engineering, University of Central Florida Orlando, Florida.

National Oceanic and Atmospheric Administration, Atlantic Oceanographic and Meteorological Laboratory (NOAA/AOML) (1999)
“Hurricane research division, Hurricane Floyd.”

Open University (2000)


Panchang, V. B. Xu, and Z. Demirbilek (1999)
“Chapter 4; wave prediction models for coastal engineering applications.” *Developments in Offshore Engineering*, J. B. Herbich, Editor, Gulf Pub., Houston, 163-194.
“Coupled 2D hydrodynamic-sediment transport and wave models, study case for a
hurricane event in Matagorda Bay of Texas.” The Fifth International Symposium on
Ocean Wave Measurement and Analysis, Spain.

Perrie, W. and Y. Zhan (2001)
“Coupling and feedbacks of the atmosphere and ocean surface: Synoptic and monthly
time scales.” WCRP/SCOR Workshop on Intercomparison and Validation of Ocean-
Atmosphere Flux Fields, Bolger Center, Potomac, MD.

“Atmosphere-ocean interactions volume 1.” WITpress.


“Physical geography.net” Retrieved from September 5, 2006 from
http://www.physicalgeography.net/fundamentals/8r.html.

Pierson, W.J., and L. Moskowitz (1964)
"A proposed spectral form for fully developed wind seas based on the similarity theory of


Powell, M.D., S.H. Houston, L.R. Amat, and N. Morisseau-Leroy (1998)
"The HRD real-time hurricane wind analysis system.” J. of Wind Engineering and
Industrial Aerodynamics: Vols. 77 & 78, 53-64.

Prandle, D., J.C. Hargreaves, J.P. McManus, A.R. Campbell, K. Duwe, A. Lane, P. Mahnke, S.
Shimwell and J. Wolf (2000)
"Tide, wave and suspended sediment modelling on an open coast - Holderness." Coastal

“Statistical evaluation of mechanistic water-quality models.” J. of Env. Eng., 116(2), 250-
267.
“SMS steering module for coupling waves and currents, 2: M2D and STWAVE.” Coastal and Hydraulics Engineering Technical Note ERDC/CHL CHETN-IV-60, U. S. Army Engineer Research and Development Center, Vicksburg, MS.

Reid R.O. (1990)


“The Effect of Tidal Inlets on Open Coast Storm Surge Hydrographs,” Coastal Engineering.

“On charting global ocean tides.” Reviews in Geophysics and Space Physics, 18, 243-68.

“Hurricane: Coping with disaster.” American Geophysical Union, Washington, DC.

“STWAVE – STEady-State Spectral Wave Model. report 1: user’s manual for STWAVE version 2.0.”

Smith, S.D. and E.G. Banke (1975)


Sorensen, R.M. (1993)
“Basic wave mechanics for coastal and ocean engineers” A Wiley-Interscience Publication.

“Introduction to physical oceanography.” Retrieved from June 20, 2005 from http://oceanworld.tamu.edu/resources/ocng_textbook/contents.html

“Numerical model for wind-wave-current coupled fields induced by typhoon.” Workshop on Wave, Tide Observations and Modelings in the Asian-Pacific Region.

“High resolution simulation of Hurricane Floyd using MM5 coupled with a wave model.”  
PSU/NCAR Mesoscale Modeling System Users' Workshop.

Thompson, E.F. and V.J. Cardone (1996)  
"Practical modeling of hurricane surface wind fields. ASCE J. of Waterway, Port, Coastal, and Ocean Engineering: Vol. 122, No. 4, 195-205.

Tolman, H.L. (1992)  
“Effects of numerics on the physics in a third-generation wind-wave model.” J. of Physical Oceanography, 22, 10, 1095-1111.

Tolman, H.L. (2002)  
“User manual and system documentation of WAVEWATCH-II version 2.22.”

Toth, D.J. (1993).  


WAMDI Group. (1988)  

Weaver, R.J. and D. Slinn (2004)  

Willebrand, J. (1975)  
“A feasibility study for the development of a joint surge and wave model.” Proudman Oceanographic Laboratory, Report, No 1, 109.


“Coupling waves and currents in POLCOMS with measurements from the coastal observatory.” Waves in operational oceanography, June 2003, Brest, France, Brest.

Wu, J. (1982)

Zhang, M.Y., and Y.S. Li (1996)

“SMS steering module for coupling waves and currents, 1: ADCIRC and STWAVE.” Coastal and Hydraulics Engineering Technical Note ERDC/CHL CHETN-IV-41, U. S. Army Engineer Research and Development Center, Vicksburg, MS.