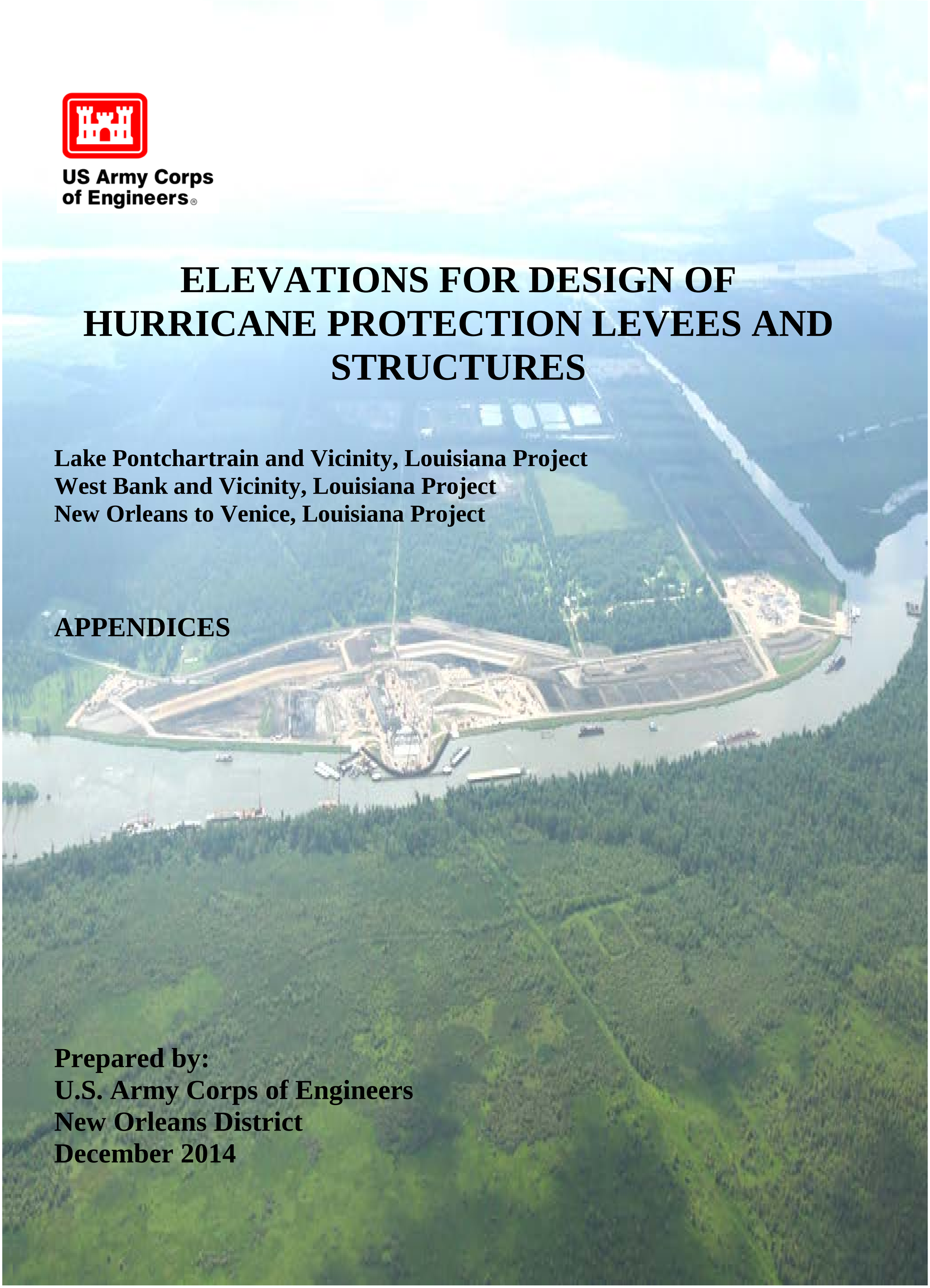

US Army Corps of Engineers®

ELEVATIONS FOR DESIGN OF HURRICANE PROTECTION LEVEES AND STRUCTURES

Lake Pontchartrain and Vicinity, Louisiana Project
West Bank and Vicinity, Louisiana Project
New Orleans to Venice, Louisiana Project

APPENDICES

Prepared by:
U.S. Army Corps of Engineers
New Orleans District
December 2014

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Appendix A

Maps of 1% SURGE levels, Wave Heights and Wave Periods

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Appendix A – Maps of 1% SURGE Levels, Wave Heights, and Wave Periods

This appendix presents the 1% surge level, significant wave heights, and peak periods that have been used for the designs. These numbers are determined with the JPM-OS method. The basis of these numbers is the storm runs with ADCIRC and STWAVE. The results of the storms are processed with a probabilistic model to obtain the 1% numbers. For more information, the reader is referred to Chapter 2.

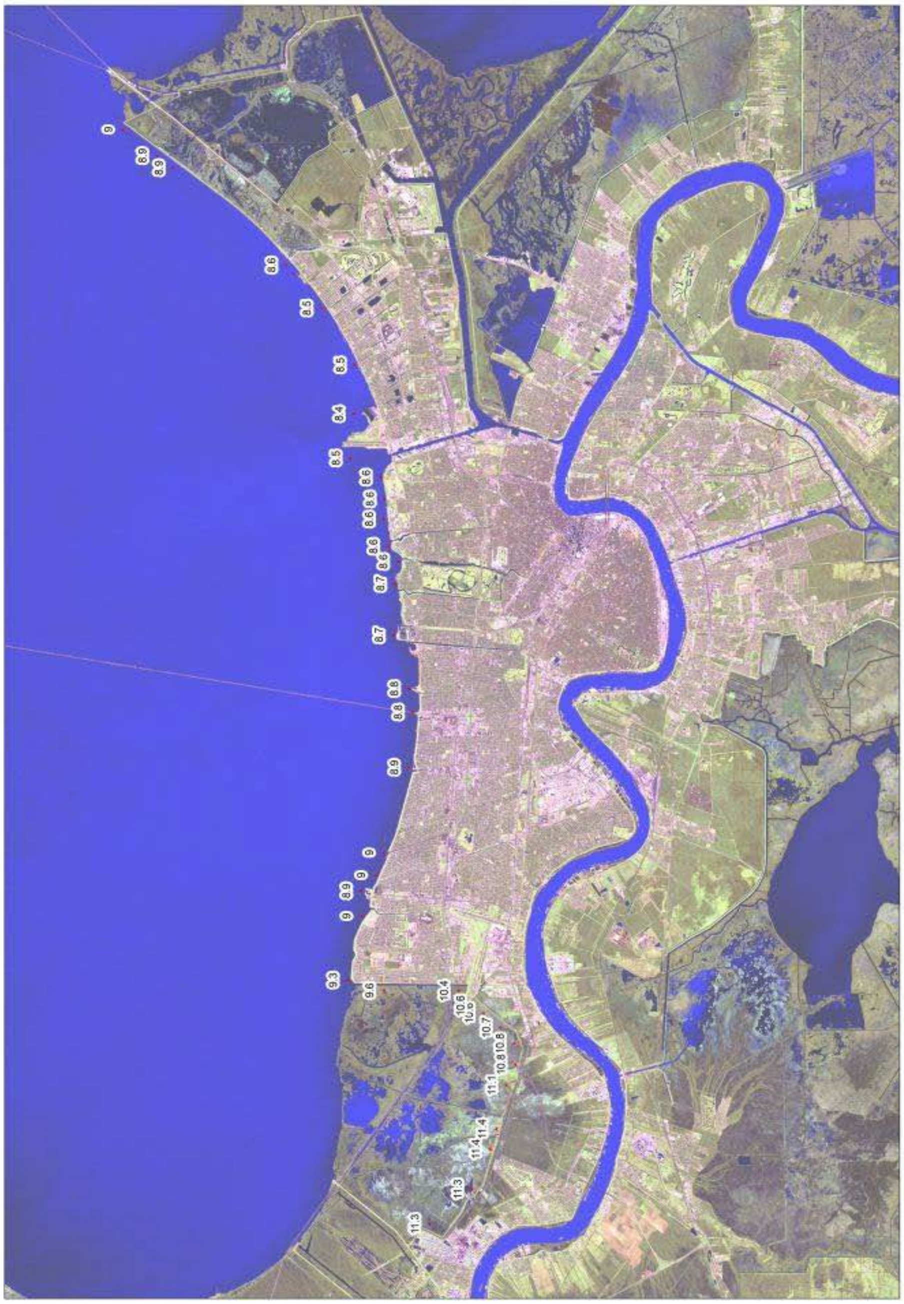

Figure A.1 – 1% Surge Levels at the Lakefront.

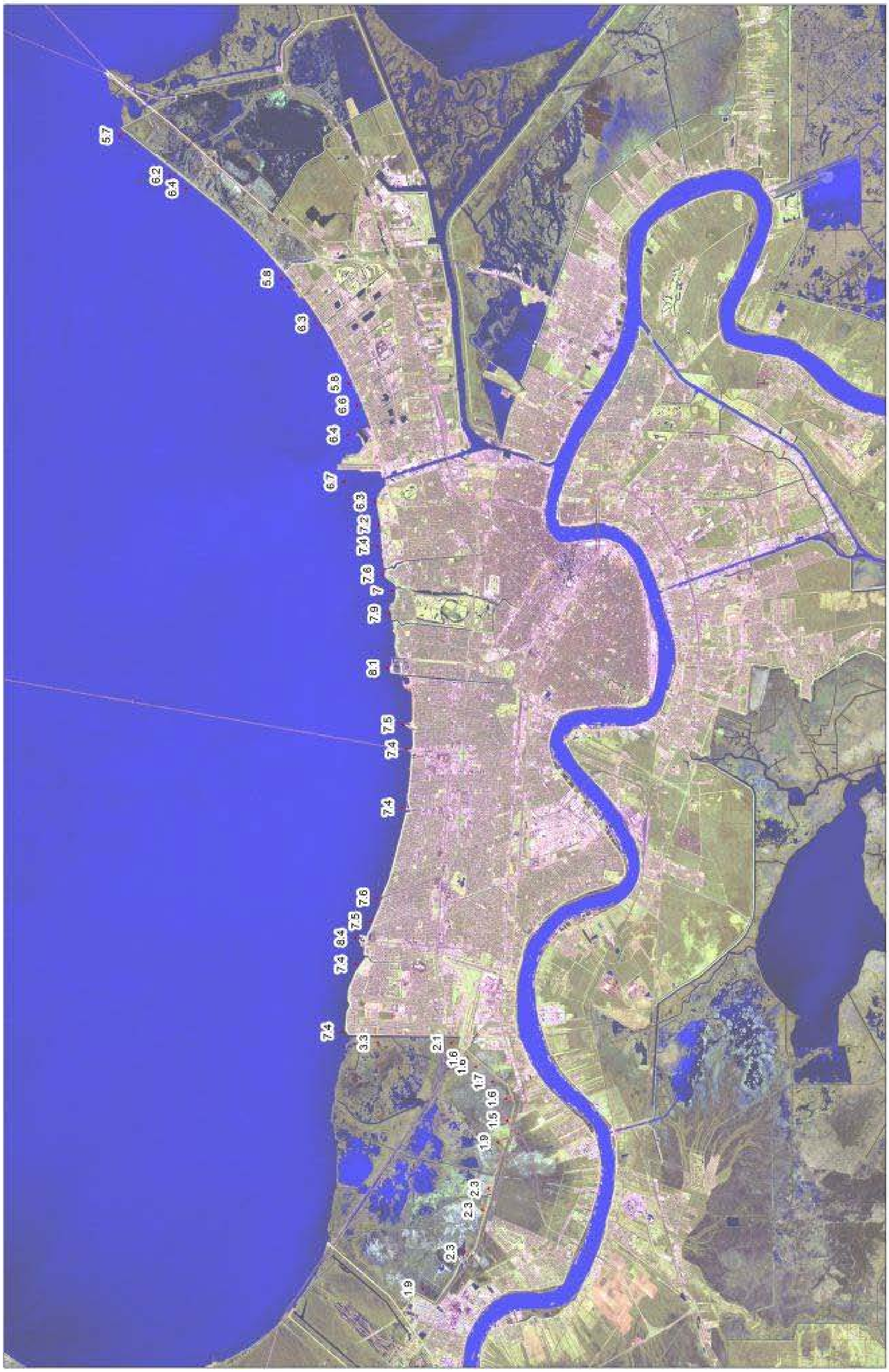

Figure A.2 – 1% Significant Wave Heights at the Lakefront.

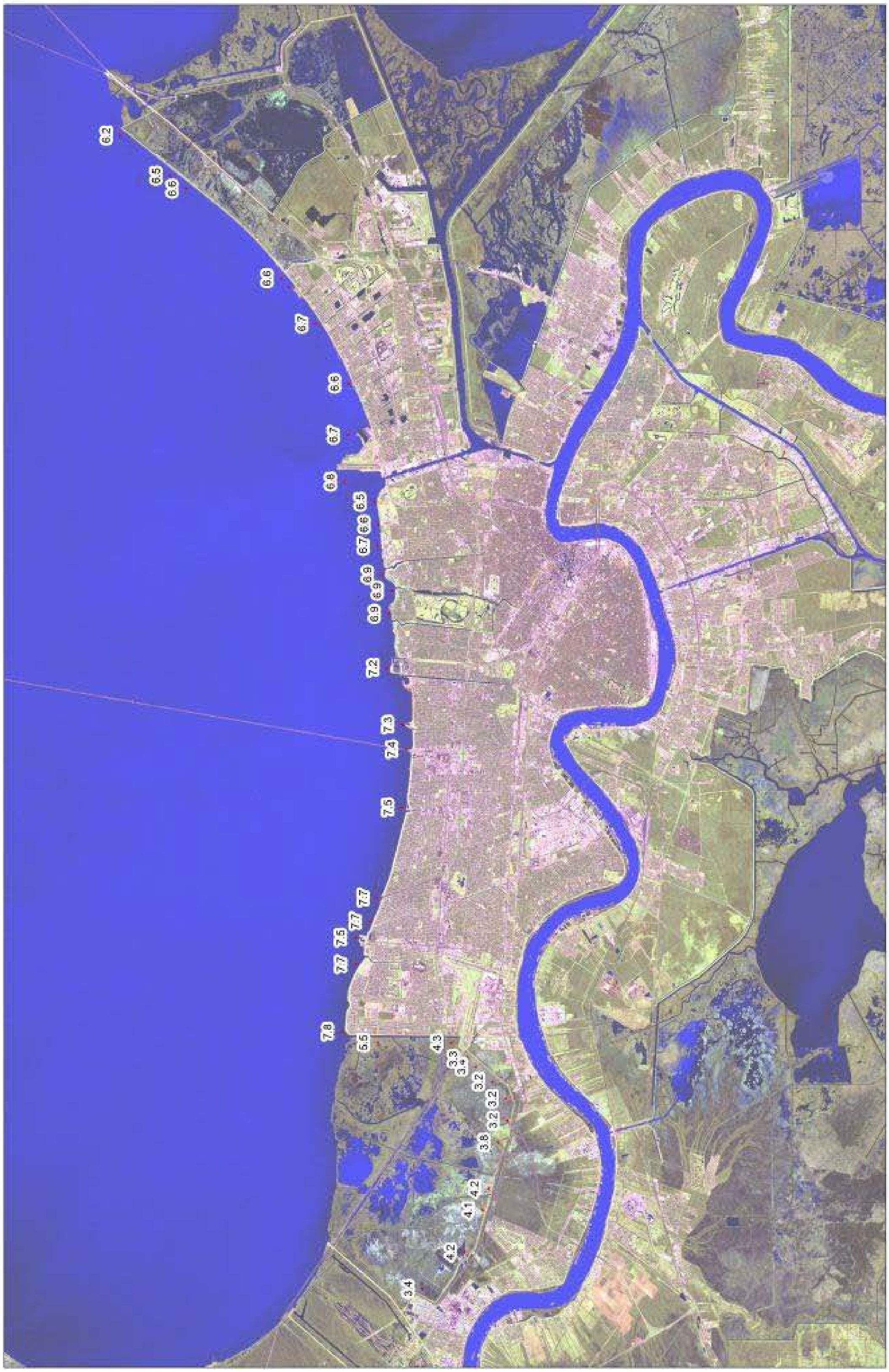

Figure A.3 – 1% Peak Period at the Lakefront.

Figure A.4 – 1% Surge Levels in the New Orleans East Area (without Seabrook).

Figure A.5 – 1% Significant Wave Heights in the New Orleans East Area (without Seabrook).

Figure A.6 – 1% Peak Period in the New Orleans East Area (without Seabrook).

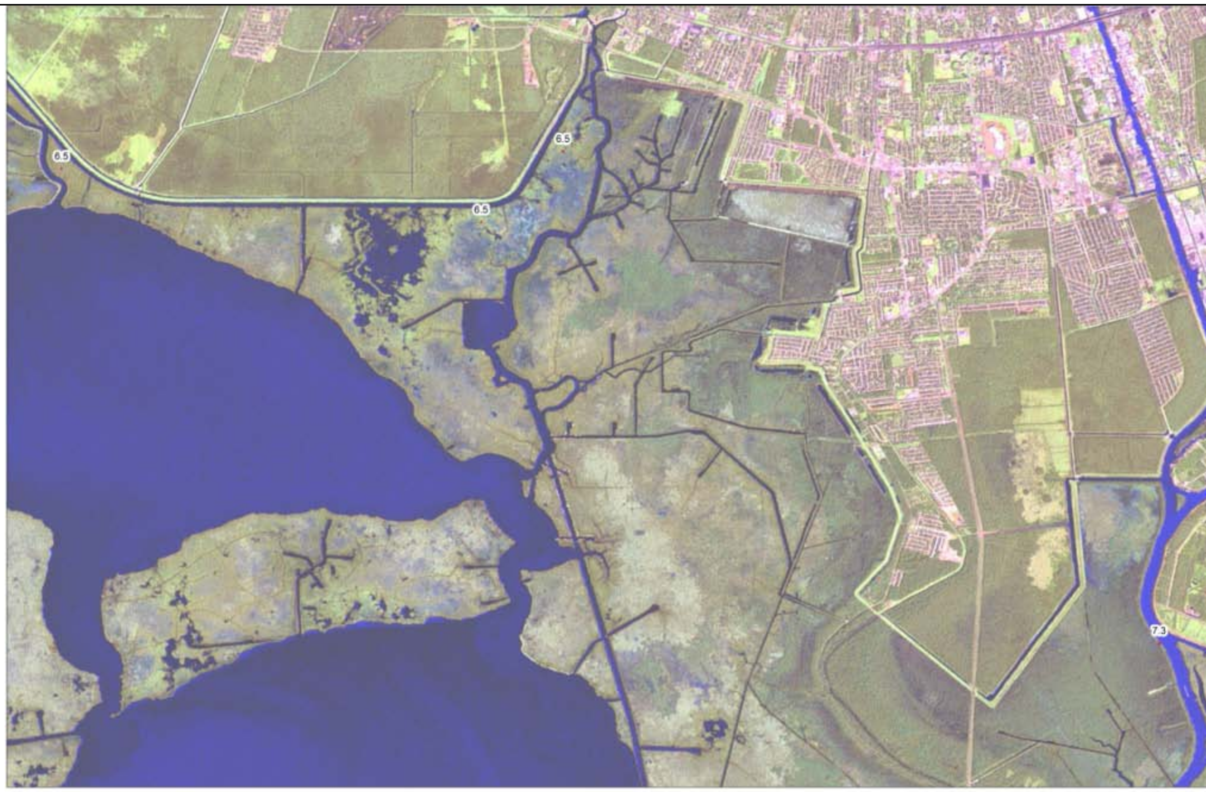

Figure A.7 – 1% Surge Levels at the West Bank.

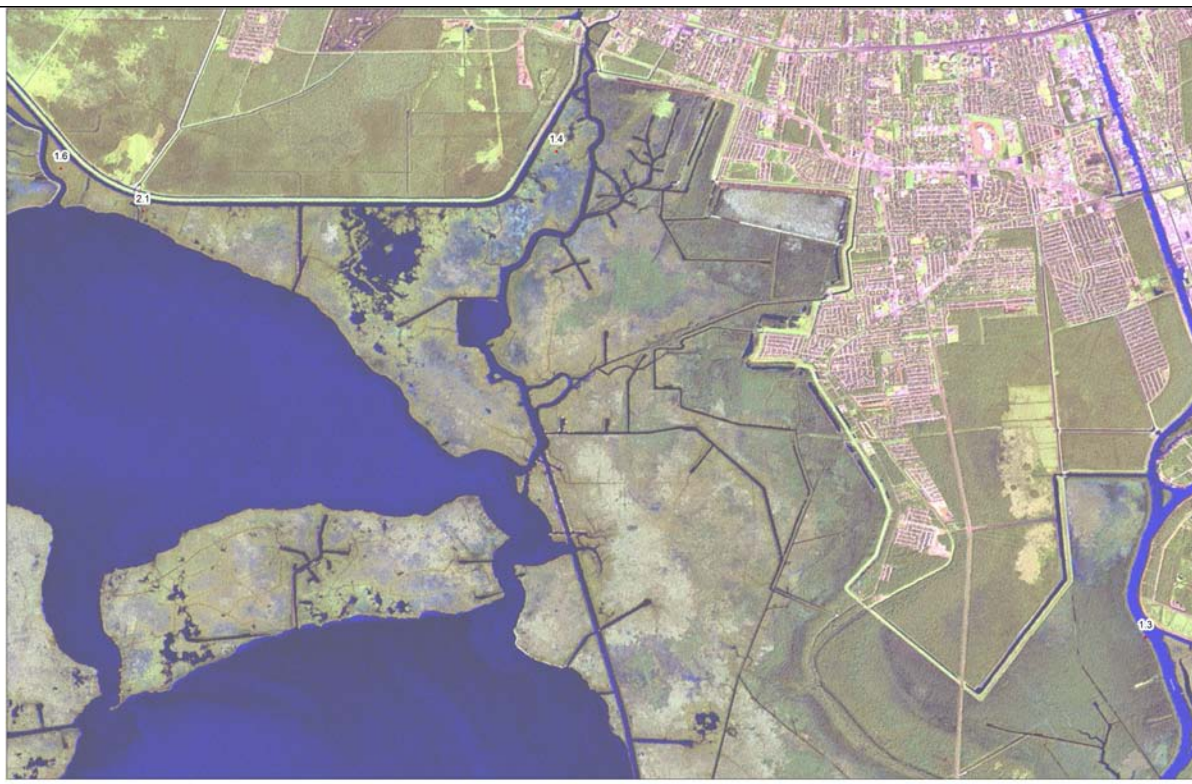

Figure A.8 – 1% Significant Wave Heights at the West Bank.

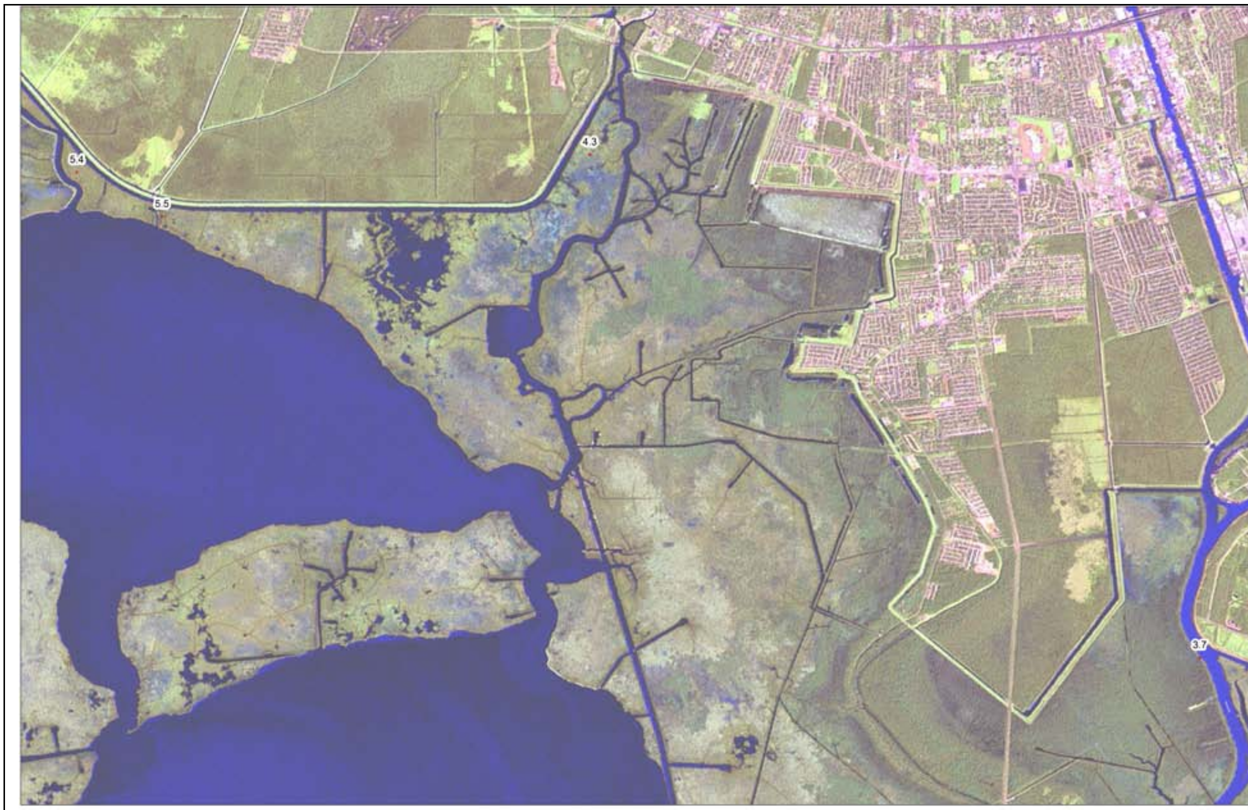

Figure A.9 – 1% Peak Period at the West Bank.

Appendix B

Sample Design Calculations

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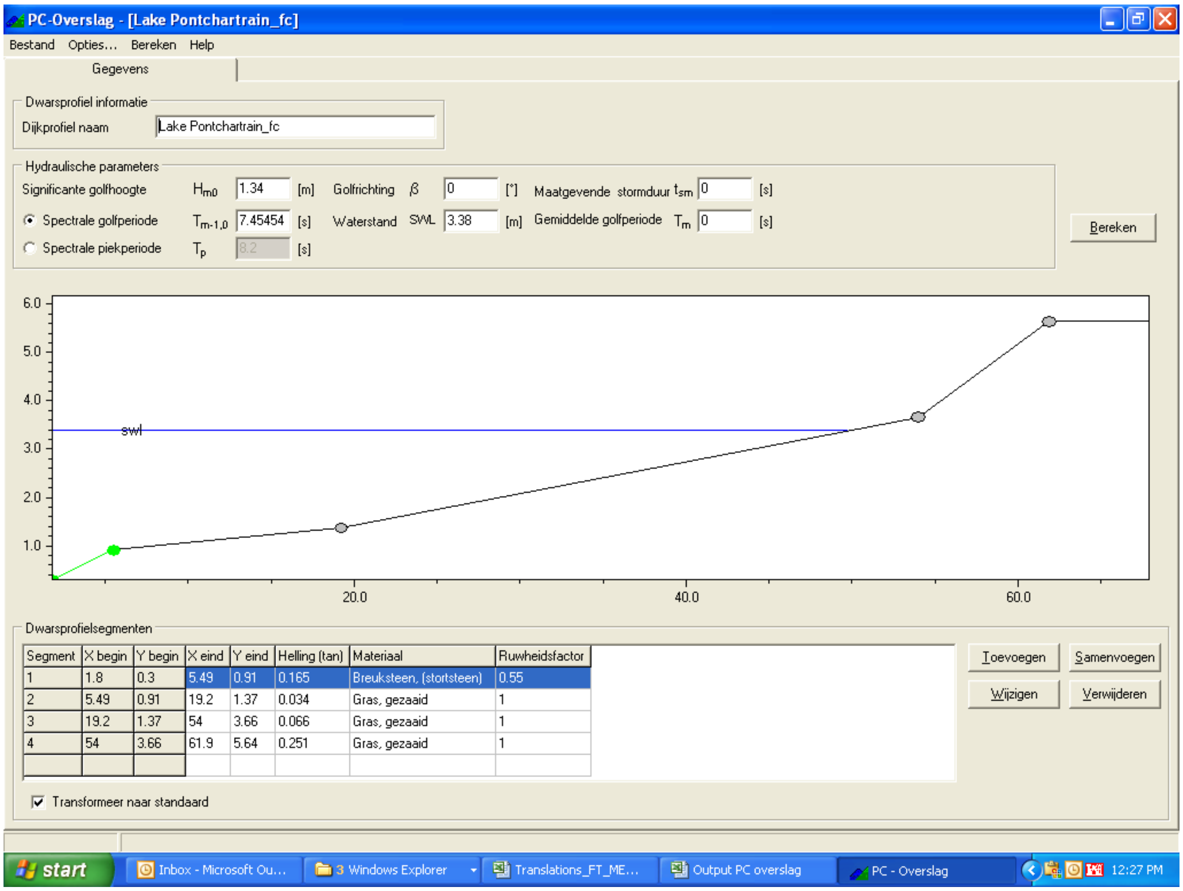
APPENDIX B – SAMPLE DESIGN CALCULATIONS

This appendix shows some examples of the design calculations. The screen dumps below show a typical levee design calculations using the Dutch program PC-Overslag. It presents the various input fields for the design significant wave height, wave period, surge elevation and levee geometry. Note that units are metric and the language is Dutch.

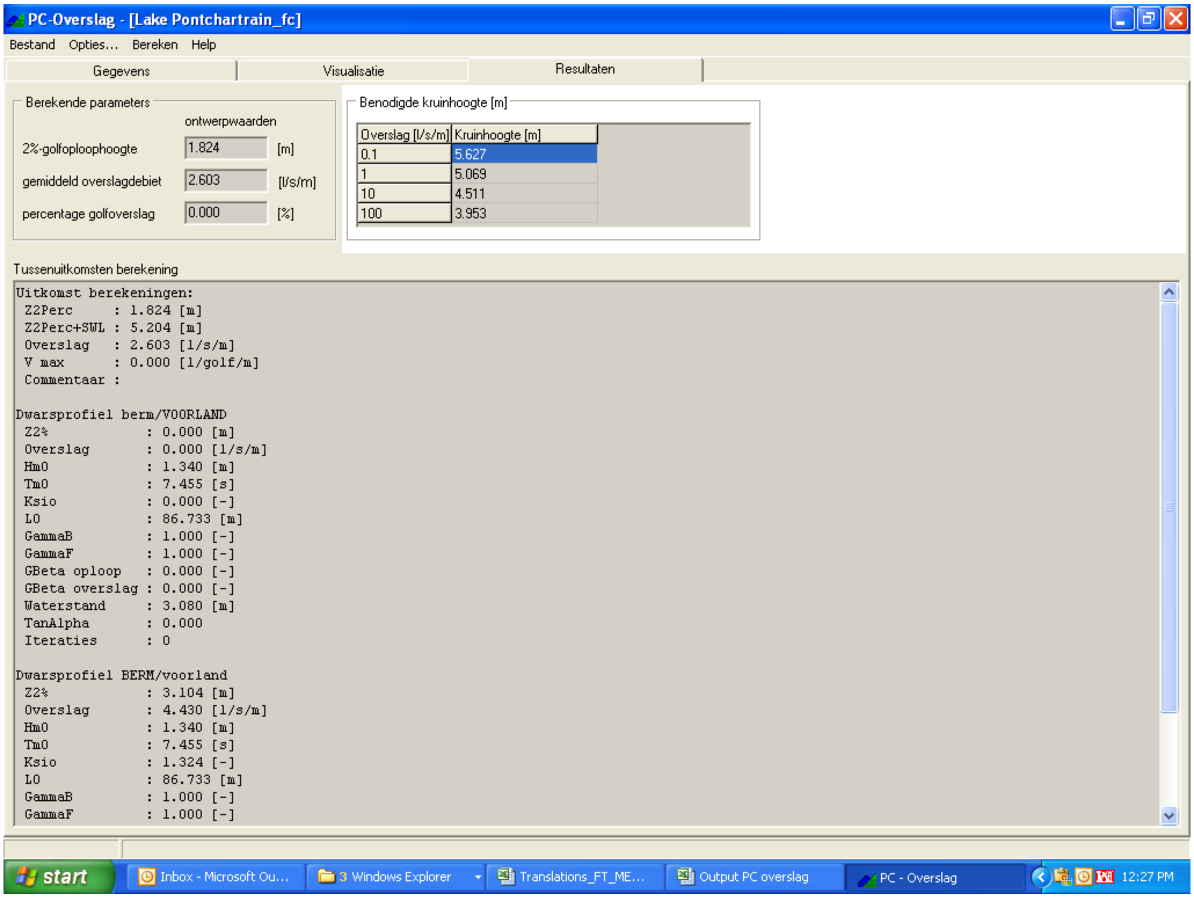


Input parameters are:

H_{m0} – Design Wave Height in meters
$T_{m-1,0}$ – Spectral Wave Period in seconds (T_p = 1.1 $T_{m-1,0}$)
β – Incident angle of wave, (0^o - perpendicular to the line of protection)
SWL – Still Water Level in meters(surge level)
t_{sm} – Storm Duration (Not used to compute peak overtopping discharge)
T_m – Average Wave Period (Not used to compute peak overtopping discharge)

X begin, Y begin – Starting horizontal and vertical points of a segment in meters starting from a fixed reference point of 0,0
X eind, Y eind – Ending horizontal and vertical points of a segment in meters starting from a fixed reference point of 0,0
Helling (tan) – angle of the slope of the segment
Materiaal – Material (ex. gras – grass)
Ruwheidsfactor – Roughness Factor (from TAW Technical Report Wave Run-up and Wave Overtopping at Dikes 2002, Section 2.7, pg.19)

The screen print from PC-Overslag below shows the output from a wave overtopping computations. It gives the overtopping rate (“gemiddeld overslag debiet”) in liters per second per linear meter (L/s/m).



For floodwalls a spreadsheet was developed to perform the wave overtopping computation of Franco & Franco (1999). This spreadsheet is shown below with the various wave input parameters.

REACH JL04 - Lake Pontchartrain Jeff PS 3 Future w/Breakwater
Floodwall Elevations
Eq. Franco and Franco (1999)

g	32.19	cfs2		Note: Add 2 ft to Wall height for
ztop	12.50	ft	Crest height	uncertainties, so Top of Floodwall
SWL	11.00	ft	Still Water Level	Elevation= **14.50**
Hs	2.50	ft	Wave Height	Check with MatLab JP program
B	0.00	ft	Wave Angle (Perpendicular Waves =0)	
Wave Type	1.00		0 for Long Crested, 1 for Short	
gamma_b	0.83	-	(computed)	
gamma_s	1.00	-	See CEM for Different Values	
Rc	1.50	ft	Free Board	
q	**0.21023617**	**cfs/ft**	**Overtopping rate**	(Design target < 0.1)

EM 1110-2-1100 (Part VI)
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Table VI-5-13
Overtopping Formula by Franco and Franco (1999)

Impermeable and permeable vertical walls. Non-breaking, oblique, long- and short-crested waves.

$$\frac{q}{\sqrt{gH_s^3}} = 0.082 \exp\left(-3.0 \frac{R_c}{H_s} \frac{1}{\gamma_\beta \gamma_s}\right) \qquad \text{(VI-5-28)}$$

Uncertainty: Standard deviation of factor 3.0 = 0.26 (see Figure VI-5-16).

Tested range:

$H_s = 12.5 - 14.0$ cm
$s_{op} = 0.04$ (wave steepness)
$\beta = 0^\circ - 60^\circ$ (angle of incidence)
$\sigma =$ app. 22° and app. 28° (directional spreading)
$R_c/H_s = 1.2$ and 1.6
$h_s/H_s =$ app. 4.4
$h_b/h_s = 0.21$

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Appendix C

Boussinesq Modeling (Author: P. Lynett, Texas A&M)

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This appendix describes the Boussinesq model COULWAVE that has been applied in this design report. It gives general background of Boussinesq models, some validation tests with COULWAVE. Finally, the generation of the lookup tables is described. The text below was provided by Pat Lynett from Texas AM (version 07/18/2007).

GENERAL WAVE MODELING BACKGROUND

To estimate wave impact, a model must be constructed. Ideally, a comprehensive effort, involving both physical and numerical modeling, should be undertaken. In this Appendix, the focus will be on describing numerical modeling of the waves. Numerous numerical packages are available, all with varying levels of approximation and computational expense. When attempting to simulate storm conditions, or long time periods in general, it is necessary to include varying water levels due to, for example, storm surges and tides. Typically, water level changes are predicted using long wave models, based on shallow water theory, such as SLOSH (Sea, Lake, and Overland Surges from Hurricanes, *e.g.* Jelesnianski *et al*., 1992) and ADCIRC (Advanced Circulation Model For Oceanic, Coastal And Estuarine Waters, *e.g.,* Kolar *et al.* 1994). These models incorporate topography and coastal barriers, and calculate flooding due to the long waves generated by pressure gradients and wind fields. Wind waves, however, and their impact on nearshore processes such as runup, cannot be directly included due to the theoretical assumptions of the model.

In the open ocean, wind wave generation and propagation is typically described using spectral models. A spectral energy balance is derived, accounting for wave growth, propagation, and dissipation based on some wind energy input. Examples of such models are WISWAVE (Wave Information Study Wave Model, *e.g.* Resio, 1981) and WAM (Wave Model, *e.g.* Komen *et al.* 1994). These models are highly developed for deep, open ocean waves, but do not account completely for coastal effects such as shallow water wave-wave interactions and depth-induced breaking (Wornom *et al*, 2001). They output a directional spectrum, which can then be employed in a coastal zone model to simulate nearshore propagation. For example, WAM could be coupled with SWAN (Simulating Waves Nearshore *e.g.* Booij *et al*, 1999), a coastal spectral model, to estimate the spectral evolution from deep to shallow water (*e.g.* Wornom *et al*., 2001). However, due to the approximations inherent in these models, including phase-averaging, weak nonlinear effects, and no diffraction, they can only crudely approximate dynamic nearshore phenomenon.

Modelers looking to perform phase-resolving simulations of waves from intermediate depths to the shoreline have few options. Well established models such as SHORECIRC (*e.g.* Svendsen & Putrevu, 1994) and SWAN are phase-averaged models and do not directly provide time histories of free surface and velocity fluctuations due to waves. Mild-slope equations models, such as REF/DIF (Refraction/Diffraction Model, *e.g.* Kirby & Dalrymple, 1983), are phase-resolving models and are computationally practical to run in most cases. However, these models have restrictions limiting their use, such as weak diffraction effects, lack of wave reflection, limitation to narrow banded spectrums, and higher-order nonlinearity is generally not captured (see Kirby & Dalrymple, 1994 for a complete discussion). Certainly there is room for improvement, and over the past decade, modeling with Boussinesq equations has begun to occupy this niche of two horizontal dimension (2HD), phase-resolving wave simulation.

Assuming that both nonlinearity and frequency dispersion are weak and are in the same order of magnitude, Peregrine (1967) derived the “standard” Boussinesq equations for variable depth in terms of the depth-averaged velocity and the free surface displacement. Numerical results based on the standard Boussinesq equations or the equivalent formulations have been shown to give predictions that compared quite well with field data (Elgar and Guza 1985) and laboratory data (Goring 1978, Liu *et al*. 1985). Because it is required that both frequency dispersion and nonlinear effects are weak, the standard Boussinesq equations are not applicable to very shallow water depth, where the nonlinearity becomes more important than the frequency dispersion, and to the deep water depth, where the frequency dispersion is of order one. The standard Boussinesq equations break down when the depth is greater than one-fifth of the equivalent deep-water wavelength. For many engineering applications, where the incident wave energy spectrum consists of many frequency components, a lesser depth restriction is desirable. To extend the applications to shorter waves (or deeper water depth) many modified forms of Boussinesq-type equations have been introduced (*e.g.* Madsen *et al.* 1991, Nwogu 1993, Chen and Liu, 1995). Although the methods of derivation are different, the resulting dispersion relations of the linear components of these modified Boussinesq equations are similar, and may be viewed as a slight modification of the (2,2) Pade approximation of the full dispersion relation for linear water waves (Witting 1984). It has been demonstrated that the "modified" Boussinesq equations are able to simulate wave propagation from intermediate water depth (water depth to wavelength ratio is about 0.5) to shallow water including the wave-current interaction (Chen *et al*. 1998).

Despite the success of the modified Boussinesq equations in intermediate water depth, these equations are still restricted to weakly nonlinearity. As waves approach shore, wave height increases due to shoaling until eventually breaking. The wave-height to water depth ratios associated with this physical process violates the weakly nonlinear assumption. This restriction can be readily removed by eliminating the weak nonlinearity assumption (*e.g.* Liu 1994, Wei *et al*. 1995). Numerical implementations of the highly-nonlinear, Boussinesq-type equations include FUNWAVE (Fully Nonlinear Boussinesq Wave Model, *e.g.* Wei *et al*., 1995) and COULWAVE (Cornell University Long and Intermediate Wave Model, *e.g.*, Lynett & Liu, 2002). These models have been applied to a wide variety of topics, including rip and longshore currents (Chen *et al*., 1999; Chen *et al*., 2003), wave runup (Lynett *et al*., 2002), wave-current interaction (Ryu *et al*., 2003), and wave generation by underwater landslides (Lynett & Liu, 2002), among many others. Boussinesq models are steadily becoming a practical engineering tool. Directional, random spectrums can readily be generated by the models, which capture nearshore evolution processes, such as shoaling, diffraction, refraction, and wave-wave interactions, with very high accuracy.

COULWAVE BACKGROUND

COULWAVE (Cornell University Long and Intermediate Wave model) was developed by Patrick Lynett (Texas A&M) and Phil Liu (Cornell) at Cornell during the late 90's. The target applications of the model are nearshore wind wave prediction, landslide-generated waves, and tsunamis, with a particular focus on capturing the movement of the shoreline, i.e. runup and inundation.

COULWAVE has the capability of solving of number of wave propagation models, however the applications for this project use the Boussinesq-type equations. To derive the Boussinesq-type model, one starts with the primitive equations of fluid motion, the Navier-Stokes equations, which govern the conservation of momentum and mass. The fundamental assumption of the Boussinesq is that the wavelength to water depth ratio is large; thus the model is meant to study shallow water waves. This fundamental assumption yields additional physical limitations, such as the vertical variation of the flow must be small, and turbulence must be parameterized – physics such as wave overturning and interaction, and overtopping of vertical structures are, theoretically speaking, beyond the application bounds of the model.

Applications for which COULWAVE has proven very accurate include wave evolution from intermediate depths to the shoreline, including parameterized models for wave breaking and bottom friction. A number of examples model-date comparisons are described now.

WAVE PROPAGATION

COULWAVE is based on the Boussinesq-type equations, which are known to be accurate for inviscid wave propagation from fairly deep water (wavelength/depth ~2) all the way to the shoreline (Wei *et al*, 1995). The equation model consists of a fairy complex set of partial differential equations:

$$\zeta_t + E = 0, \quad \boldsymbol{u}_{\alpha t} + \boldsymbol{F} = 0 \tag{1}$$

where

$$\begin{aligned} E = {} & \nabla \cdot [(h+\zeta)\boldsymbol{u}_\alpha] - \nabla \cdot \Big\{ (h+\zeta) \\ & \times \Big[\Big(\frac{1}{6}(\zeta^2 - \zeta h + h^2) - \frac{1}{2} z_\alpha^2 \Big) \nabla (\nabla \cdot \boldsymbol{u}_\alpha) \\ & + \Big[\frac{1}{2}(\zeta - h) - z_\alpha \Big] \nabla [\nabla \cdot (h\boldsymbol{u}_\alpha)] \Big] \Big\} \end{aligned} \tag{2}$$

$$\begin{aligned} F = {} & \boldsymbol{u}_\alpha \cdot \nabla \boldsymbol{u}_\alpha + g \nabla \zeta \\ & + \Big\{ \frac{1}{2} z_\alpha^2 \nabla (\nabla \cdot \boldsymbol{u}_{\alpha t}) + z_\alpha \nabla [\nabla \cdot (h\boldsymbol{u}_{\alpha t})] \Big\} \\ & + \{ [\nabla \cdot (h\boldsymbol{u}_\alpha)] \nabla [\nabla \cdot (h\boldsymbol{u}_\alpha)] - \nabla [\zeta (\nabla \cdot (h\boldsymbol{u}_{\alpha t})] \\ & + (\boldsymbol{u}_\alpha \cdot \nabla z_\alpha) \nabla [\nabla \cdot (h\boldsymbol{u}_\alpha)] \} \\ & + \Big\{ z_\alpha \nabla [\boldsymbol{u}_\alpha \cdot \nabla (\nabla \cdot (h\boldsymbol{u}_\alpha)] \\ & + z_\alpha (\boldsymbol{u}_\alpha \cdot \nabla z_\alpha) \nabla (\nabla \cdot \boldsymbol{u}_\alpha) + \frac{z_\alpha^2}{2} \nabla [\boldsymbol{u}_\alpha \cdot \nabla (\nabla \cdot \boldsymbol{u}_\alpha)] \Big\} \\ & + \nabla \Big\{ -\frac{\zeta^2}{2} \nabla \cdot \boldsymbol{u}_{\alpha t} - \zeta \boldsymbol{u}_\alpha \cdot \nabla [\nabla \cdot (h\boldsymbol{u}_\alpha) +] \\ & + \zeta [\nabla \cdot (h\boldsymbol{u}_\alpha)] \nabla \cdot \boldsymbol{u}_\alpha \Big\} \\ & + \nabla \Big\{ \frac{\zeta^2}{2} [(\nabla \cdot \boldsymbol{u}_\alpha)^2 - \boldsymbol{u}_\alpha \cdot \nabla (\nabla \cdot \boldsymbol{u}_\alpha)] \Big\} \end{aligned} \tag{3}$$

which are integrated in time to solve for the free surface elevation, ζ, and the horizontal velocity vector, $\boldsymbol{u}_\alpha$. A 4th order Adams-Bashforth-Moulton predictor-corrector time integration scheme is required, and the spatial derivatives are approximated with 4th order, centered finite differences. The high order scheme is required due to the inclusion of first to third order derivatives in the model equations. Waves are generated in the numerical domain with an internal source (Wei *et al*, 1999), which can use as input a wave energy spectrum to create a directional, random wave field. In conjunction with the internal source generator, sponge layers are placed along the outgoing lateral boundaries, and provide excellent wave absorption across a wide range of frequencies and amplitudes. The model simulates moving boundaries in the swash zone using a numerical technique presented in Lynett *et al*. (2002). The moving waterline is modeled by extrapolating the solution from the wet region onto the beach. This linear extrapolation locates the position of the waterline between wet and dry nodes, thereby allowing the real boundary to exist in-between grid points and improving the accuracy of the solution. The numerical results evaluated at the extrapolated waterline are used to update the solution for the next time step. This moving-boundary technique is numerically stable and does not require any artificial dissipation mechanisms.

Fundamentally, the above Boussinesq equations are inviscid. To accommodate frictional effects, viscous submodels are integrated into COULWAVE. Bottom friction is calculated with the quadratic friction equation:

$$R_{BottomFriction} = f\,\frac{\mathbf{u}_b\left|\mathbf{u}_b\right|}{H}$$

where $\mathbf{u}_b$ is the velocity evaluated at the seafloor, and f is a bottom friction coefficient, typically in the range of 0.001 to 0.01. As noted in Lynett *et al* (2002), maximum runup is sensitive to the value of f, particularly for very large, breaking waves: a value of 0.005 is used for all simulations here, which is consistent with the value used in the ADCIRC simulations. To simulate the effects of wave breaking, the eddy viscosity model of Kennedy *et al* (2000) is used here with some modification as given in Lynett (2006b).

WAVE BREAKING

The wave breaking model has received much attention and has undergone numerous validation exercises. The wave breaking model is based on the “eddy-viscosity” scheme, where energy dissipation is added to the momentum equation when the wave slope exceeds some threshold value, and continues to dissipate until the wave slope reaches some minimum value when the dissipation is turned off.

One set of comparisons is shown in Figure 1 for a number of regular waves breaking and running up a slope. As can be seen, COULWAVE captures the mean values of height and water level to a high degree of accuracy. While these comparisons show that the model is capable of capturing a simplified, laboratory setup, it is also necessary to gauge the accuracy against real, field conditions. COULWAVE has been compared with a number of field sites; one such comparison is given in Figure 2. As can be seen, the model captures the spectral transformation of random waves through the surf zone. Note that the breaking model uses a single set of parameters for all trials, so there is no individual case optimization.

The horizontal velocity profile under breaking waves is a necessary component to capture accurately for transport-related physics. Using a process of superposition of velocity profiles (Lynett, 2006), instantaneous and mean profiles under breaking waves in predicted well (see Figure 3.)

Publications which specifically use COULWAVE to simulate wave breaking include Lynett et al (2002), Lynett et al (2003), Basterretxea et al (2004), Lynett & Korycansky (2005), Cheung et al (2005), Lynett (2006a&b), Lynett (2007), and Korycansky et al (2007).

WAVE RUNUP AND INUNDATION

The moving shoreline condition has shown to capture shoreline motion due to a wide range of wave frequencies, wave heights, and beach slopes. The shoreline algorithm was originally developed to simulate the important motion of tsunami runup (Lynett *et al*, 2002), and uses a variation of the so-called “extrapolation” technique. The extrapolation method has its roots in Sielecki and Wurtele (1970), with extensions by Hibberd and Peregrine (1979), Kowalik and Murty (1993), and Lynett et al. (2002). The basic idea behind this method is that the shoreline location can be extrapolated using the nearest wet points, such that its position is not required to be locked onto a fixed grid point; it can move freely to any location. Theoretically, the extrapolation can be of any order; however, from stability constraints a linear extrapolation is generally found. Hidden in the extrapolation, the method is roughly equivalent to the use of low-order, diffusive directional differences taken from the last wet point into the fluid domain (Lynett et al., 2002). Additionally, there are no explicit conservation constraints or physical boundary conditions prescribed at the shoreline, indicating that large local errors may result if the flow in the extrapolated region cannot be approximately as linear in slope. The extrapolation approach can be found in both NLSW and Boussinesq models with finite difference, finite volume, and finite element solution schemes, and has shown to be accurate for a wide range of non-breaking, breaking, two horizontal dimension, and irregular topography problems.

Recently (Korycansky & Lynett, 2005), extensive comparisons have been made with empirical runup laws and existing experimental data for runup due to regular waves. Figure 4 shows how COULWAVE compares with the so-called Irribaren scaling for runup, an established coastal engineering relation based on deep water properties of the waves. Publications which specifically use COULWAVE for runup or the moving shoreline algorithm developed by Lynett include Lynett et al (2002), Lynett et al (2003), Lynett & Korycansky (2005), Cheung et al (2005), Pedrozo-Acuña et al (2006), Lynett (2006a&b), Lynett (2007), and Korycansky et al (2007).

OVERTOPPING OF SLOPING STRUCTURES

Quality, time-dependent data for wave overtopping of levees and dikes is sparse. Thus, as with existing published numerical models (e.g. Dodd, 1998), the large majority of comparisons provided here will use time-averaged experimental data. First, a comparison is made with the data of Saville (1955). This data set is one of the standard comparisons found in the literature (e.g. Kobayashi & Wurjanto, 1989; Dodd, 1998; Hu et al, 2000). An example of the physical setup for these trials is given in Figure 5, a spatial snapshot for a numerical simulation. A range of freeboard and wave conditions were tested. A summary of the comparisons is given in Table 1. Overall, the agreement between the Boussinesq simulations and the experiments is quite good. Where the two diverge, the Boussinesq results tend to agree with the published numerical results of Kobayashi & Wurjanto.

The Boussinesq model results must also exhibit agreement with well established empirical formulas such as those given by Owen (1984) and Van der Meer & Janssen (1995). For these tests, a wide range of wave and levee configurations are tested. Ranges of parameters are: levee slope from 1/3 – 1/8, freeboard from 1’ to 4’, wave height at the structure toe from 2’-8’, and wave period from 8s-16s. The incident wave condition is a shallow water TMA spectrum using a gamma value of 3.0. Approximately 500 Boussinesq simulations were performed, and the comparisons with the formula of van der Meer & Janssen are shown in Figure 6. Agreement is quite good.

A noteworthy result of these comparisons is the conclusion that, when using the wave height and water level at the toe of the last simple slope of the structure, there is no accuracy preference between the empirical formulas and the detailed hydrodynamics (Boussinesq). Thus, for relatively simple setups where the wave height at the structure toe can be estimated with high confidence, the empirical formulas provide the same level of accuracy as the Boussinesq with significantly less computational expense. On the other hand, if the levee is fronted by a series of slopes or an arbitrary shaped protecting structure, some method must be used to provide the wave height at the toe of the last simple slope. For this situation, the Boussinesq can be used to provide this wave height; however the Boussinesq can also provide the overtopping for such a setup and would be the logical choice for estimating overtopping, provided the computational resources and expertise required by the modeling are available. However, it must be noted that while COULWAVE has not specifically been used to model overtopping of a levee with a series of foreshore slopes (in terms of experimental benchmarking) it has been used to model shoaling, breaking, and runup (without overtopping) on numerous irregular beaches, with good accuracy. With the information that the model can simulate overtopping of a simple slope (essentially a validation of the moving shoreline model), and its ability to transform the wave over irregular bathymetry (it can transform the wave to the last slope), it is expected that the model can accurately simulate levee overtopping with irregular foreshore. While there is high confidence that COULWAVE is handling these complicated situations well, there will soon be additional experimental validation of these cases, with data provided by planned ERDC experiments.

DEVELOPMENT OF BOUSSINESQ-BASED OVERTOPPING LOOKUP TABLES

The procedure used to develop the lookup tables is given here. For example, the creation of the lookup table for the New Orleans East Lakefront levee reach, shown in Figure 7, will be described. First, a set of independent parameters and their ranges must be specified. For this example, the reach profile is constant, and the independent parameters are incident wave height, peak wave period, and surge water elevation. All of these parameters are specified at 600' from the levee toe, and represent information provided from STWAVE and ADCIRC runs. For each independent parameter, a range and increment are given to create a bin:

wave height = [2' 5' 7' 9' 11']
peak wave period = [6s 8s 10s 12s 15s 18s]
surge water elevation = [8' 11' 14' 17' 20' 24']

For each parameter combination, a Boussinesq simulation is run. Thus, for this New Orleans East Lakefront location, there are a total of 5 x 6 x 6 = 180 simulations that are used to create the lookup table. Figure 8 gives an example snapshot of the wave surface from a Boussinesq simulation. For each simulation, time series of free surface elevation, depth-averaged velocity, and mass flux are recorded throughout the reach length. Each of these time series is distilled to a significant wave height, a mean water level, and a mean flux. Note that mean flux, when measured on the crest of a levee, is identical to the overtopping rate in units of water volume/time per unit length of crest. Using the interpolation routines of MATLAB, a simple program was created to provide wave height, wave setup, and overtopping values for any combination of input conditions bracketed by the independent parameter ranges shown above. The use of this function is simple:

```
function lookup(location, water_level, wave_period, wave_height)
% This Matlab function will use built-in 3-dimensional linear interpolation to do
% a lookup.  Inputs are in English units.  "location" corresponds to the site examined:
%               1 = Lakefront_Airport_Floodwall
%               2 = Citrus_Lakefront_Floodwall_Levee
%               3 = NO_East_Lakefront_Levee
%               4 = Jefferson_Parish_Lakefront_Levee
%               5 = Lakefront_Levee_short
%               6 = Lakefront_Levee_long
```

For example, to estimate wave heights and overtopping for New Orleans East Lakefront, for an incoming wave height of 8', wave period of 14 sec, and water level of +15', you would run:

```
lookup(3, 15, 14, 8)
```

and the MATLAB lookup function provides the following information:

****
Simulation Predictions for NO_East_Lakefront_Levee
Water Level Relative to MWL (ft): 15
Significant Wave height (ft) at STWAVE handoff: 8
Peak Wave Period (s): 14

Predicted H_{mo} at structure toe (ft) = 3.3299
Predicted wave setup at structure toe (ft) = 0.51698
Predicted water level (plus wave setup) at toe (ft) = 15.517
Total water depth at structure toe (ft) = 1.517
Levee crest elevation (ft) = 18
Levee toe elevation (ft) = 14
Levee freeboard, including wave setup effect on mean water level (ft) = 2.483
Levee overtopping rate given by Boussinesq simulation (ft^3/s/ft):0.37727
Levee overtopping rate given by TAW formula (ft^3/s/ft):0.66254

NOTE: Empirical prediction based on wave height at toe from Boussinesq simulation
This is not consistent with the formula - TAW wave height should not include any
reflected energy. It does here, and so formula predictions should be larger,
and this could be a substantial difference.

Levee overtopping rate given by TAW formula with R=0.4 (ft^3/s/ft):0.12753

The script displays a number of important values. The script provides the wave setup at the structure toe, the wave height at the toe, and the overtopping rate predicted by the Boussinesq model. The script also provides the overtopping rates as predicted by the empirical TAW guidance. However, this TAW prediction must be used with caution within this script. The TAW equations are driven by the wave height at the toe of the structure, without the structure in place. More specifically, the laboratory data on which the formulations are built use a side channel, with no structure, to measure the incident wave height. In the Boussinesq simulation, the structure is there, and so the wave height at the toe includes the reflected wave component. Therefore, in general, the Boussinesq prediction will be lower than the TAW prediction based on the Boussinesq toe wave height. To provide a range of numbers, the TAW prediction assuming a reflection coefficient of 0.4 is also provided. Essentially, this second TAW prediction uses 0.6*wave height at toe to drive the formula. The 0.4 value is expected to be near the largest possible value for the reflection coefficient; a value near 0.2 is more common.

Note that while the discussion above has focused only on the New Orleans East Lakefront, lookup tables for five other characteristic reaches are included with this tool. These other locations are noted in the "function lookup" description given above. One additional example for a different reach is given now, for the New Orleans Lakefront typical section shown in Figure 9; the largest predicted wave setup will be sought for this reach. Note, however, that the largest wave setups do not generally occur when there is significant overtopping. Usually, these large setups (approaching 1.5') occur when there is a wide, shallow surf zone which dissipates nearly all of the wave energy. This implies a low surge level (and a large freeboard). For the New Orleans Lakefront with hydrodynamic conditions:

Surge Water Level relative to datum (ft): 8

Significant Wave height (ft) at STWAVE handoff: 11
Peak Wave Period (s): 12

The wave setup = 1.3' (freeboard of 9.2'), but there is no overtopping. With a higher surge:

Surge Water Level relative to datum (ft): 12
Significant Wave height (ft) at STWAVE handoff: 11
Peak Wave Period (s): 12

The wave setup is reduced to 0.8' (freeboard of 5.7'), but now there is overtopping of 0.033 $ft^3/s/ft$.

For the reaches that have a floodwall, the Boussinesq provides the wave height and water level at the toe of the floodwall, and the empirical equations of Franco & Franco in the Coastal Engineering Manual are used to provide overtopping rates for a range of floodwall elevations. The Boussinesq model cannot easily model the overtopping of a vertical wall, and thus the hybrid Boussinesq-empirical approach is used for reaches with floodwalls.

While the lookup tool described above, for the six specific reaches, is useful to estimate the overtopping for a known reach profile, it does not provide design flexibility. For example, if the levee crest elevation of the New Orleans East Lakefront levee was changed from 18' to 20', or if the foreshore protection elevation was changed from 7' to 12', the existing lookup will no longer be as useful for providing overtopping information. To accommodate this design flexibility, a second lookup table was generated. For this lookup, the physical properties of the reach are no longer held constant. Here, the levee elevation, levee slope, and properties of the foreshore protection are allowed to vary. Figure 10 gives a graphical description of the independent parameters. Following this figure, the parameters and their ranges are:

wave height = [2' 5' 8' 11']
wave period = [6s 10s 14s 18s]
surge water elevation = [8' 12' 16' 20' 24']
crest elevation of levee = [1' 6' 12' 18' 24']
levee slope = [1/4 1/8]
crest elevation of foreshore protection = [1' 5' 10' 15']
distance between foreshore protection crest and levee toe = [100' 225' 350']

Now there are 7 independent parameters, and a Boussinesq simulation is run for each parameter combination. For this generic lookup table, the total number of simulations required to create the lookup table is 4 x 4 x 5 x 5 x 2 x 4 x 3 = 9600. As with the specific-reach lookup described previously, a MATLAB program is created to perform the seven-dimensional interpolation required. The use of this function is:

```
function lookup(water_level, wave_period, wave_height, levee_elevation, levee_slope,
breakwater_location, breakwater_elevation, wall_or_levee)
% This Matlab function will predict overtopping rates, based on approximated 10,000
% Boussinesq simulations.  For levees (with no floodwall), the provided overtopping
% rate is directly from the Boussinesq simulations.  For reaches with floodwalls,
% either stand-alone or crowning a levee, the overtopping rate is from the empirical
% relation of Franco & Franco (CEM), using the Boussinesq-predicted wave height and
% water level at the toe of the wall.  All inputs are in English units.
% water_level = surge elevation in ft
% wave_period = peak wave period at STWAVE handoff in sec
% wave_height = H_mo at STWAVE handoff in ft
```

% levee_elevation = % levee_slope = side slope of levee
% levee_toe_elevation = elevation of levee toe in ft
% breakwater_location = distance from levee toe to crest of breakwater (foreshore protection) in ft, must be >100'
% breakwater_elevation = crest elevation of foreshore protection in ft
% wall_or_levee = a boolean which tells if there is a floodwall or not.
% If =1, this means there exists a floodwall with toe elevation = levee_elevation, and the floodwall height will be varied to provide the critical height.
% If =0, this means there is only a levee with toe elevation = levee_toe_elevation, and the levee crest will be varied to provide the critical height.

For example, if the user wanted to estimate the overtopping rate due a surge level of 12', a wave period of 9s, and a wave height of 8' on a levee with crest elevation of 18' and a side slope of 1/5 with a foreshore breakwater at a seaward distance from the levee of 300' and a crest elevation of 9', the function call would be:

lookup(12,9,8,18,1/5,300,9,0)

and the lookup output is:

```
**********************************************************************
Simulation Predictions for Generic Profile with Foreshore Protection
Water Level Relative to MWL (ft): 12
Significant Wave height (ft) at STWAVE handoff: 8
Peak Wave Period (s): 9
Levee Elevation (ft): 18
Levee Slope: 1/5
Foreshore Protection Location (ft), distance seaward of levee toe: 300
Foreshore Protection Elevation (ft): 9

Predicted H_{mo} at structure toe (ft) = 3.8201
Predicted wave setup at structure toe (ft) = 0.97742
Predicted water level (plus wave setup) at toe (ft) = 12.9774
Total water depth at structure toe (ft) = 11.9774
Levee crest elevation (ft) = 18
Levee toe elevation (ft) = 1
Levee freeboard, including wave setup effect on mean water level (ft) = 5.0226
Levee overtopping rate given by Boussinesq simulation (ft^3/s/ft):0.12919
Levee overtopping rate given by TAW formula (ft^3/s/ft):0.27757

NOTE: Empirical prediction based on wave height at toe from Boussinesq simulation
This is not consistent with the formula - TAW wave height should not include any
reflected energy.  It does here, and so formula predictions "should" be larger,
and this could be a substantial difference.

Levee overtopping rate given by TAW formula with R=0.4 (ft^3/s/ft):0.013209
```

As with the specific reach lookup, TAW formula predictions are provided. Also, the MATLAB program outputs a plot of the bottom profile and the wave height and wave setup. The plot corresponding to the above lookup call is given as Figure 11. Floodwall overtopping is included in the hybrid Boussinesq-empirical manner described for the specific reach cases.

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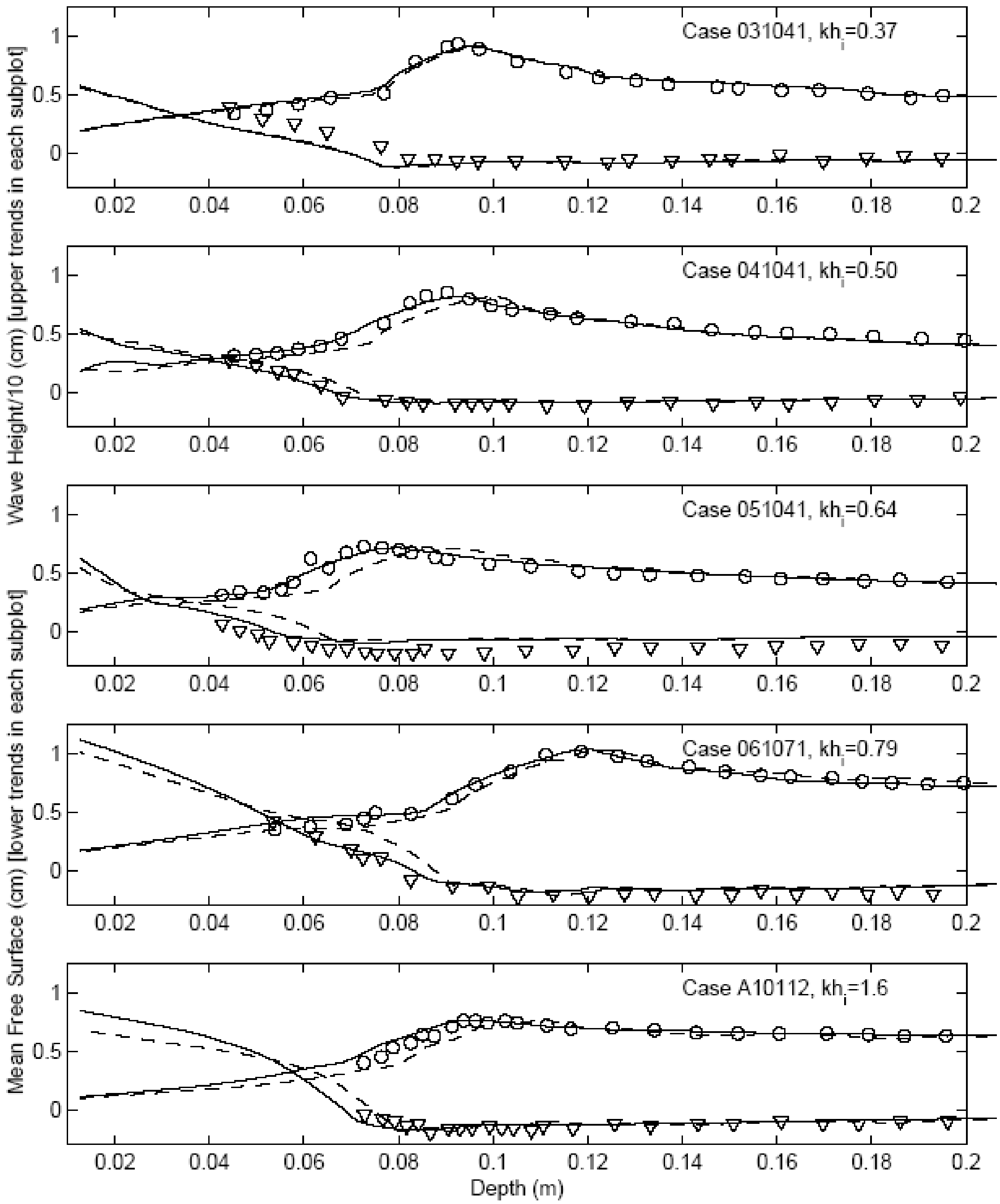

Figure 1. Wave height and mean free surface measurements from the experiments of Hansen and Svenson (1978) (symbols), from the traditional Boussinesq model (dashed-line), and from COULWAVE (solid line). Trials are for monochromatic waves breaking on a planar 1/20 slope.

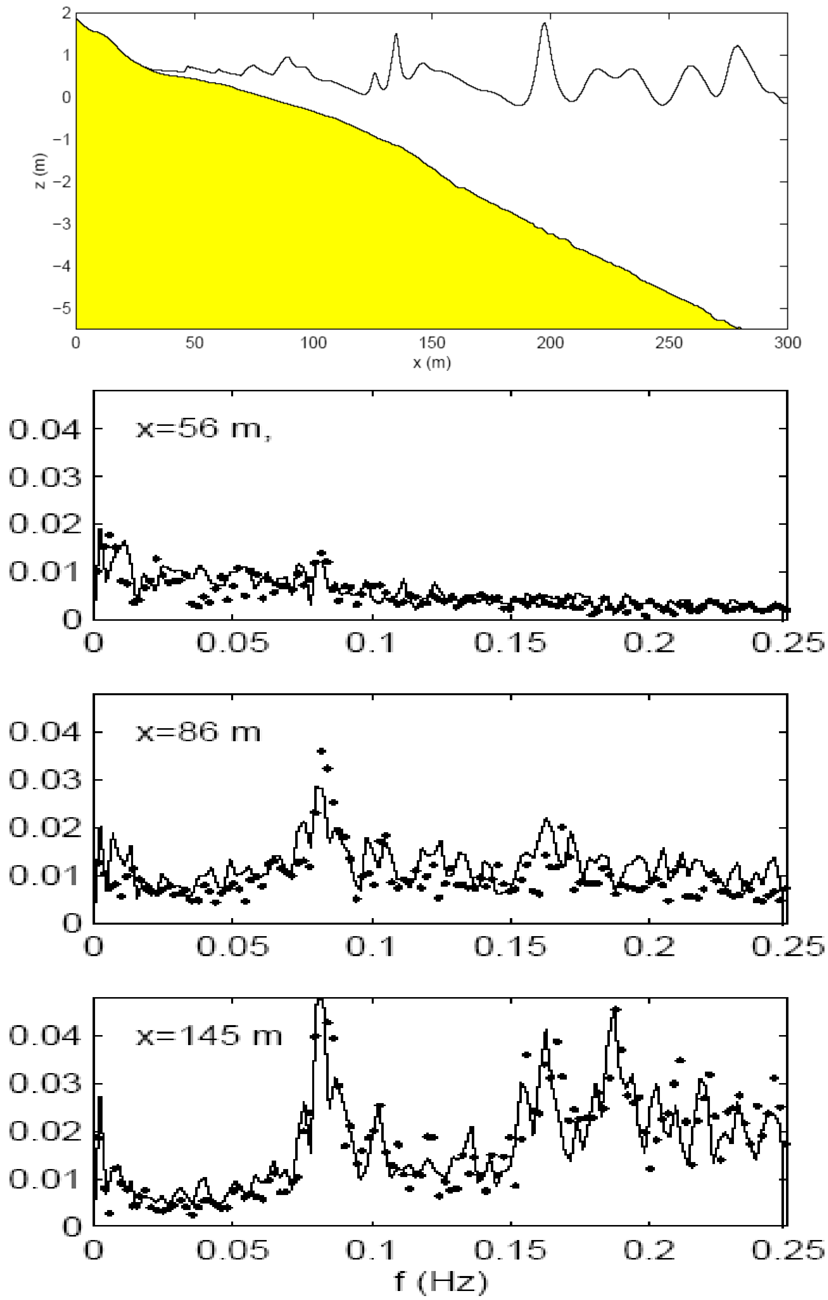

Figure 2. COULWAVE random wave comparison with field data. The lower subplots show the spectrum comparisons at three different locations, where the dots are the field data from Raubenhiemer (2002), and the solid lines are the COULWAVE results.

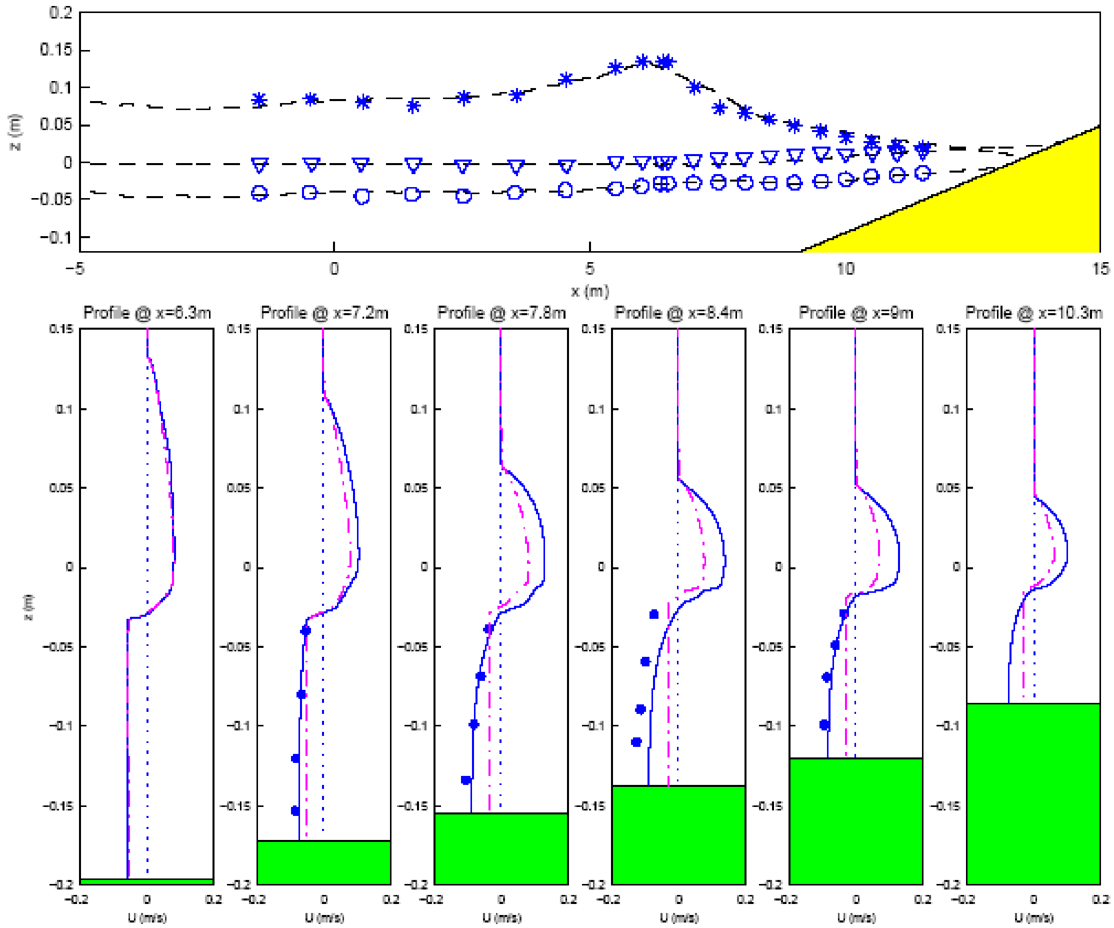

Figure 3. Comparison with the data of Ting and Kirby (1995) spiller. The top plot shows the mean crest level (stars), mean water level (triangles), and mean trough level (circles) for the experiment as well as the numerical simulation. The lower subplots are the time-averaged horizontal velocities, where the experimental values are shown with the dots, COULWAVE results by the solid line, and the standard Boussinesq results by the dashed-dotted line.

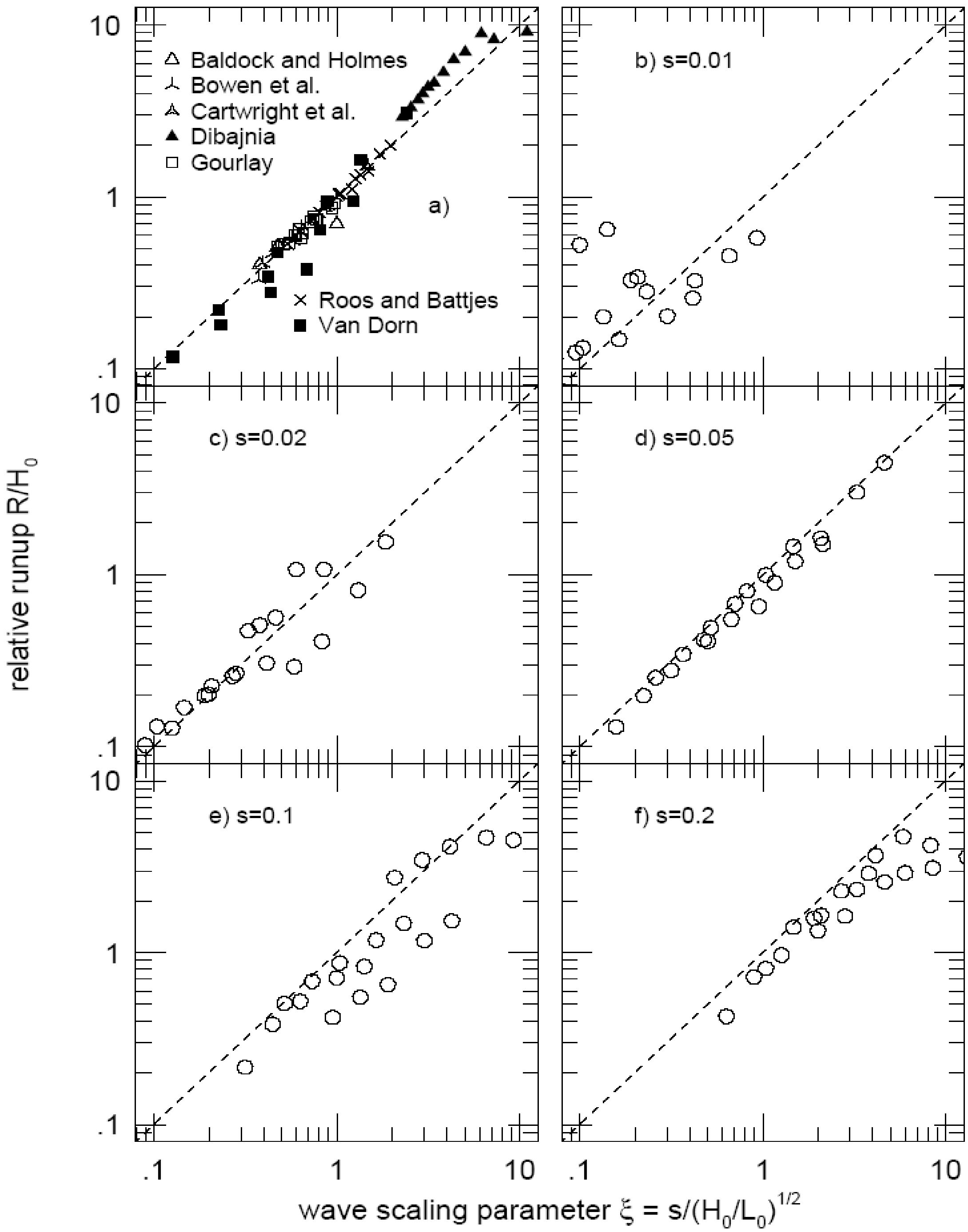

Figure 4. Wavetank experimental measurements of runup from the literature (Bowen *et al.*, 1968; Roos and Battjes, 1976; Van Dorn, 1976, 1978; Gourlay, 1992; Baldock and Holmes, 1999; Gourlay, 1992; Dijabnia, 2002) and COULWAVE runup results (open circles). The relative runup R/H_0 is plotted vs. the wave scaling parameter $\xi = s(H_0/L_0)^{1/2}$. Panel a) Experiments; b) COULWAVE runs with s=0.01 c) COULWAVE runs with s=0.02 d) COULWAVE runs with s=0.05 e) COULWAVE runs with s=0.1 f) COULWAVE runs with s=0.2.

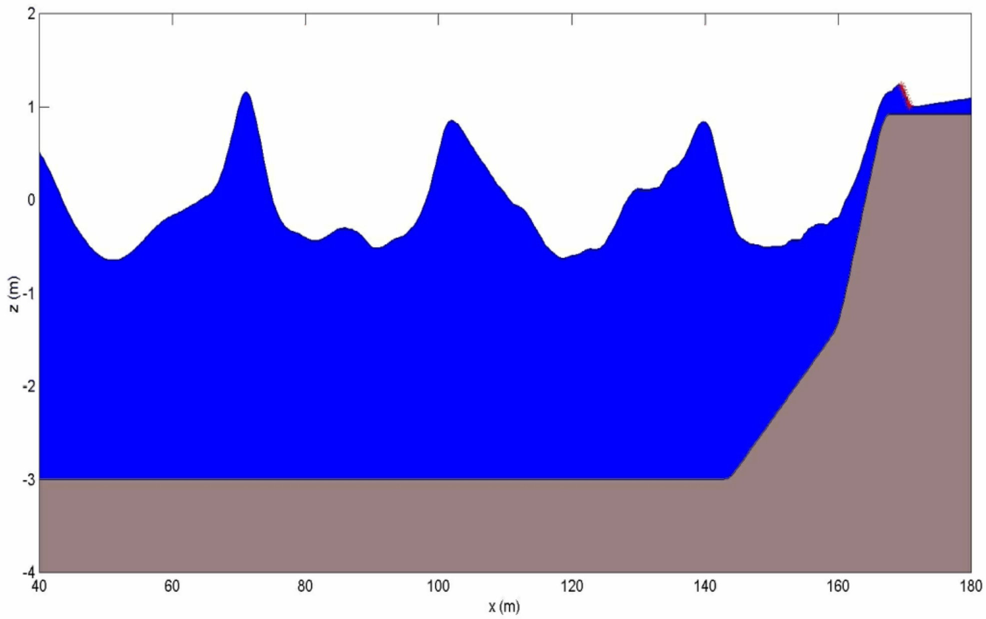

Figure 5. COULWAVE snapshot from a recreation of the Saville (1955) experiments. The general setup is a wavemaker depth ~3m, a flat portion leading up to a 1/10 slope, which connects to the "structure." In these experiments, the structure has either a 1/3 or 1/1.5 slope.

Table 1. Numerical comparisons with data from the Saville (1955) experiments. In the table, Ho is the wave height at the wavemaker, T is the wave period, Htoe is the wave height at the toe of the structure, R is the distance between the structure crest and the surge level, d_toe is the is the water depth at the toe, slope is the 1/slope of the structure, Q_meas is the measured overtopping flux, Q_K&W is the simulated overtopping by Kobayashi & Wurjanto (1989), and Q_Bous is the COULWAVE simulated flux.

Run	Ho (m)	T (s)	Htoe (m)	R (m)	d_toe (m)	slope	Q'_meas (m2/s)	Q'_K&W (m2/s)	Q'_Bous (m2/s)
1	1.83	6.39	1.74	0.91	1.37	3	0.51	0.21	0.35
2	1.83	6.39	1.74	1.83	1.37	3	0.32	0.02	0.21
3	1.83	6.39	1.74	0.91	2.74	3	0.50	0.41	0.49
4	1.83	6.39	1.74	1.83	2.74	3	0.28	0.11	0.16
5	1.37	7.67	1.36	0.92	2.74	3	0.45	0.41	0.44
6	1.83	10.8	1.94	0.91	1.37	3	0.47	0.42	0.42
7	1.83	10.8	1.9	1.83	1.37	3	0.13	0.12	0.12
8	1.83	10.8	1.94	2.74	1.37	3	0.31	0.02	0.04
9	1.83	10.8	1.94	0.91	2.74	3	0.73	0.71	0.68
10	1.83	10.8	1.94	1.83	2.74	3	0.31	0.35	0.35
11	1.83	10.8	1.94	2.74	2.74	3	0.06	0.12	0.11
12	1.37	14.97	1.62	0.92	1.37	3	0.46	0.49	0.46
13	1.37	14.97	1.62	0.92	2.74	3	0.65	0.57	0.63
14	1.37	14.97	1.62	1.82	2.74	3	0.39	0.26	0.33
15	1.37	14.97	1.62	2.74	2.74	3	0.13	0.08	0.09
16	1.37	14.97	1.62	3.66	2.74	3	0.06	0.08	0.03
17	1.83	10.8	1.88	0.91	1.37	3	0.38	0.51	0.44
18	1.83	10.8	1.88	2.74	1.37	1.5	0.10	0.06	0.09
19	1.83	10.8	1.88	0.91	0	1.5	0.30	0.31	0.31
20	1.83	10.8	1.88	1.83	0	1.5	0.16	0.05	0.09

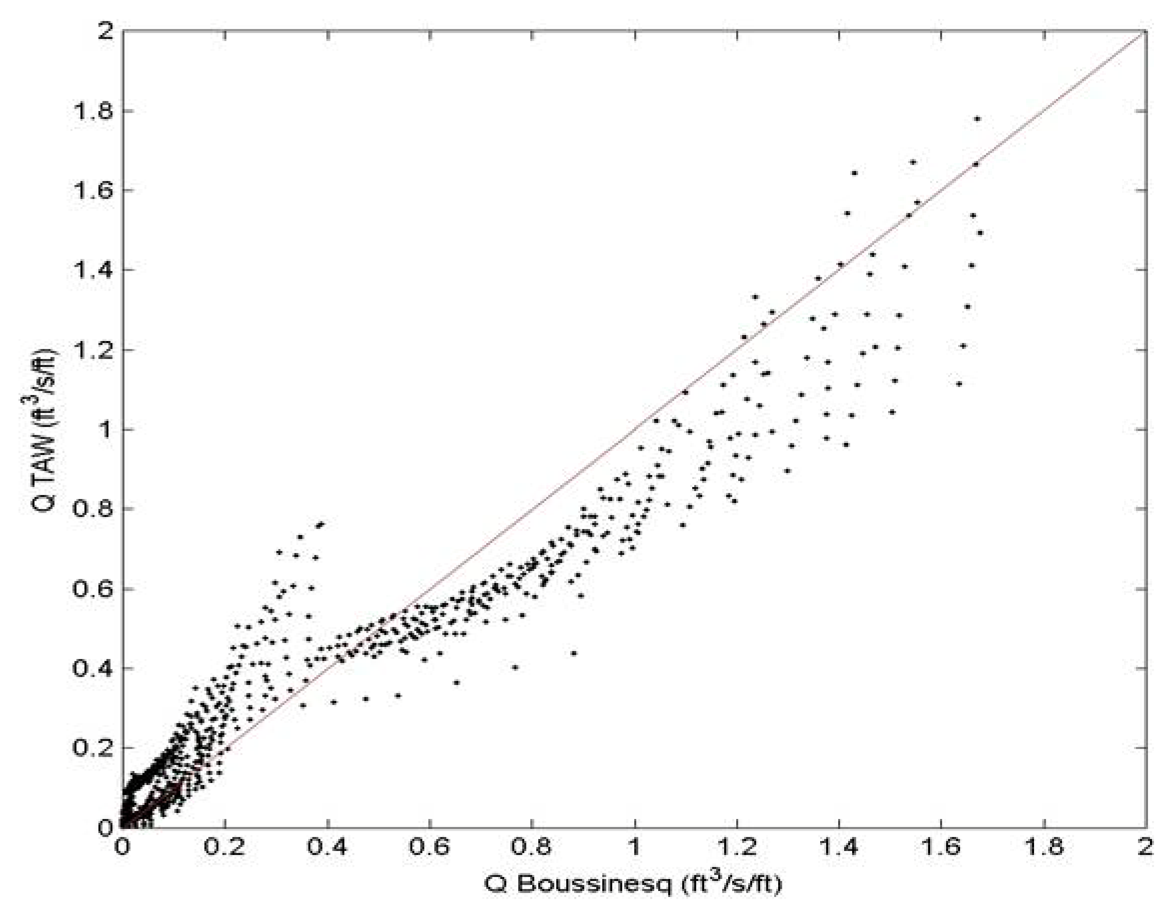

Figure 6. Comparison of Boussinesq overtopping rates with the formula given in the TAW design guidance.

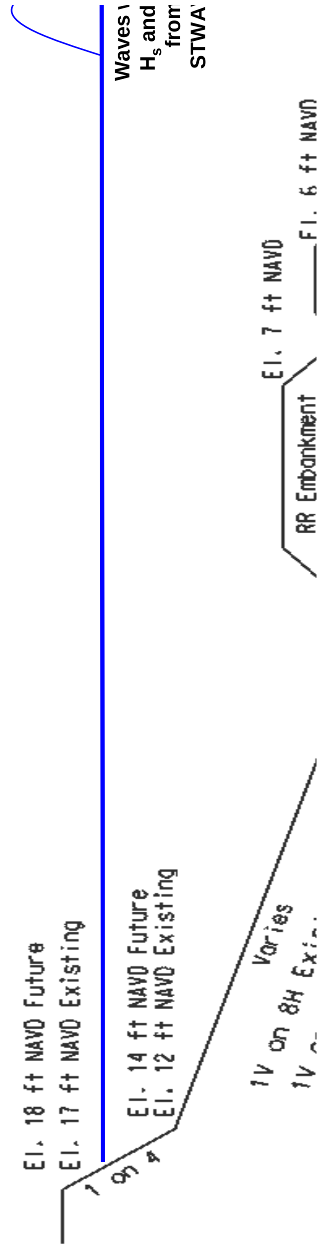

Figure 7. New Orleans East Lakefront Levee typical section

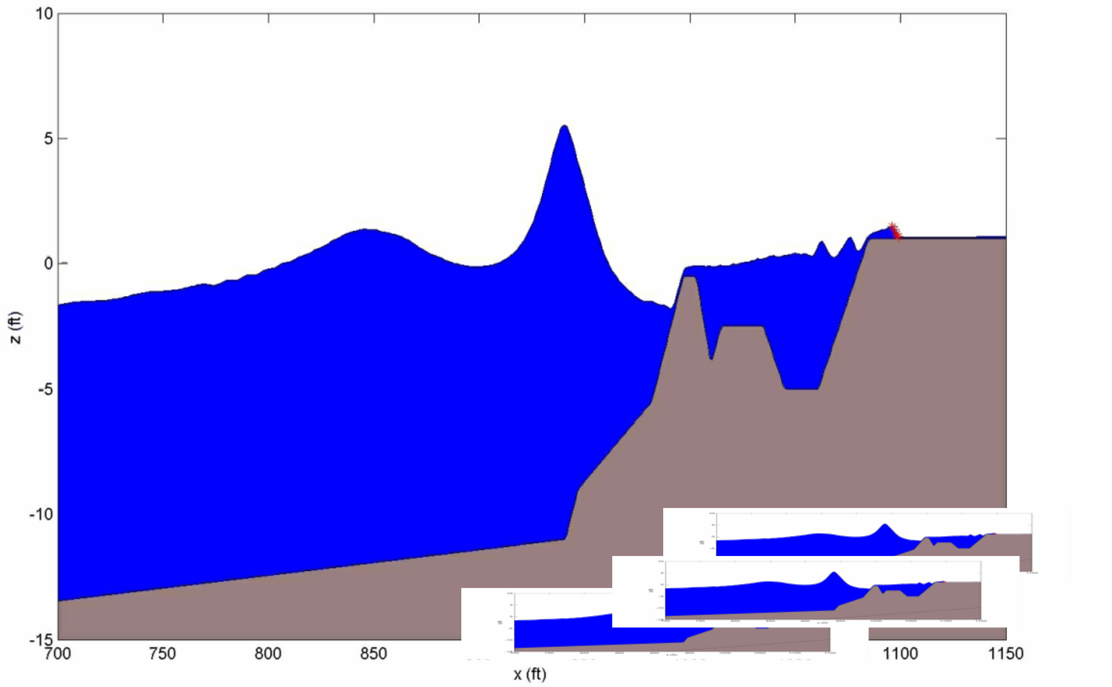

Figure 8. Snapshot from Boussinesq simulation of waves propagating across a reach with foreshore protection.

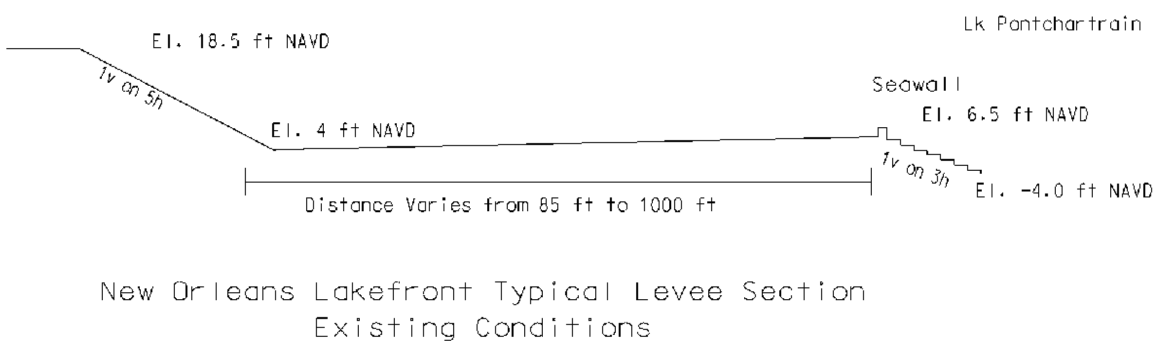

Figure 9. New Orleans Lakefront Levee typical section

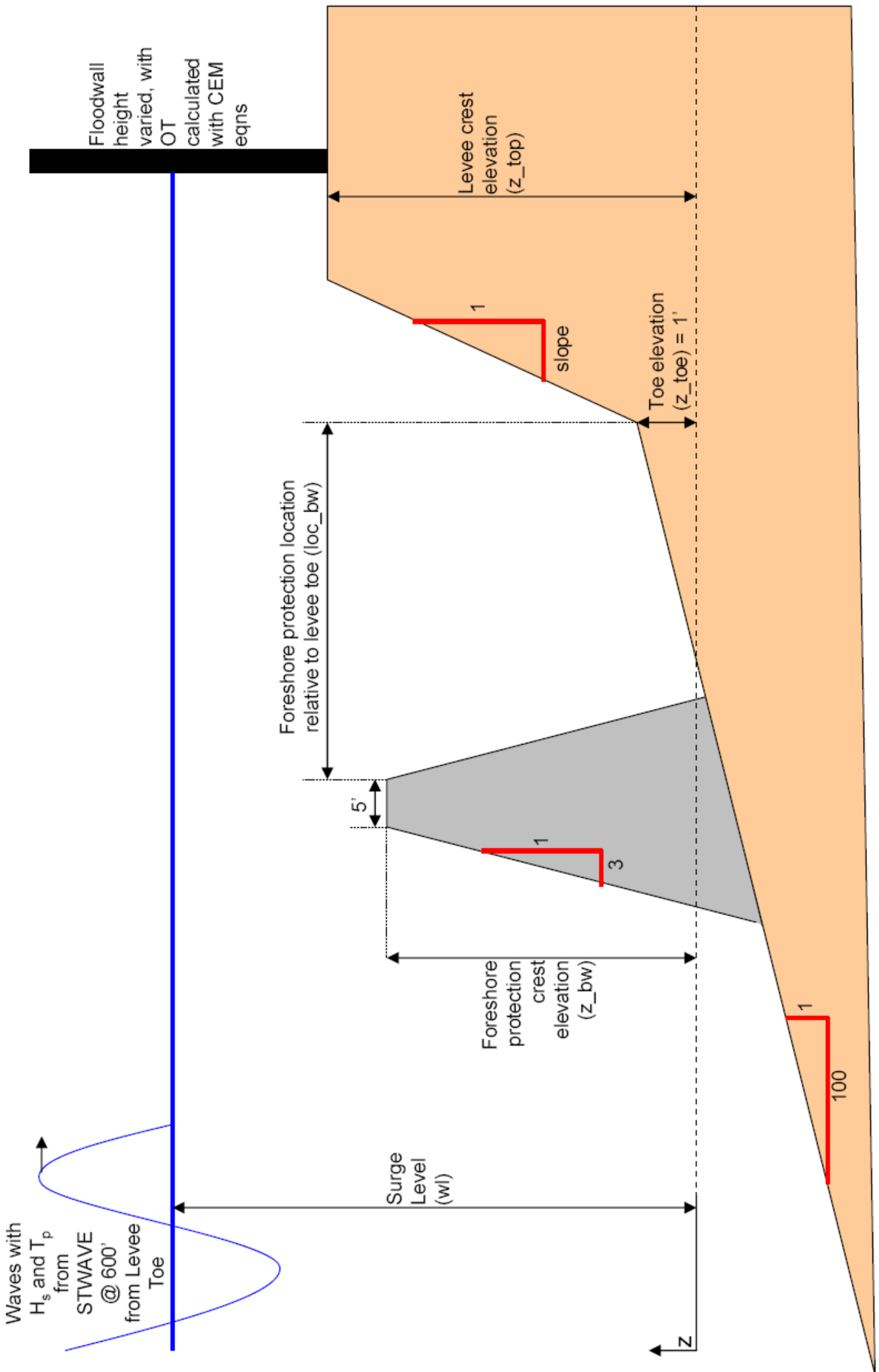

Figure 10. Schematic for generic reach lookup.

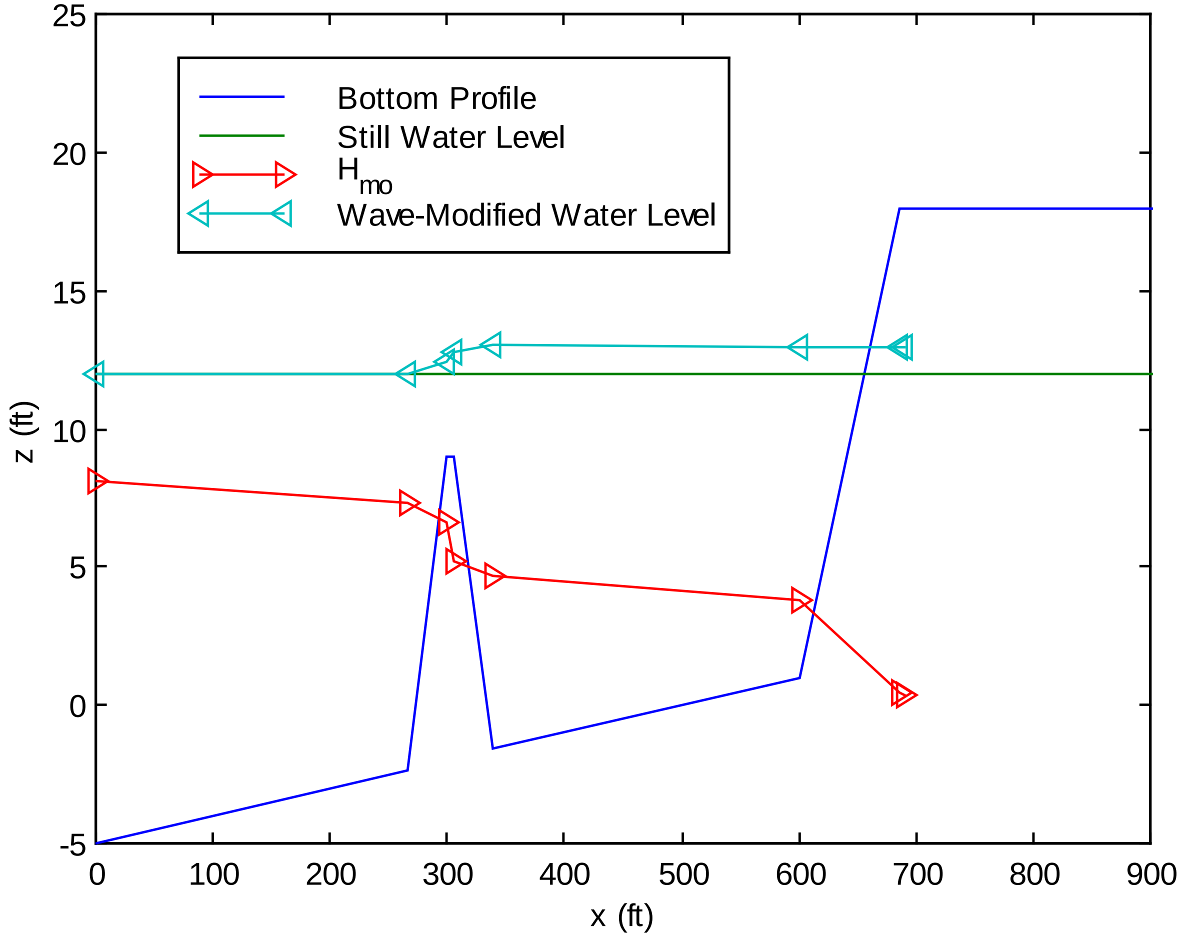

Figure 11. Example Matlab output plot from the generic lookup tool.

Appendix D

Comparison Between Empirical and Boussinesq Approach

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APPENDIX D – COMPARISON BETWEEN EMPIRICAL AND BOUSSINESQ APPROACH

General

In the design approach empirical formulations have been used to evaluate the overtopping rate for the levee designs. This appendix discusses a comparison between using Boussinesq results and empirical formulations in the design approach. A comparison is necessary to test if both approaches result in (more or less) the same results. The benefit of the Boussinesq model is to evaluate more complicated geometries. Hence, several sections were evaluated with a Boussinesq model and a lookup table was created. A lookup table was provided for the following sections:

1 = Lakefront Airport Floodwall
2 = Citrus Lakefront Floodwall Levee
3 = New Orleans East Lakefront Levee
4 = Jefferson Parish Lakefront Levee
5 = Lakefront Levee short
6 = Lakefront Levee long

The overtopping rate can be evaluated quickly from the lookup table if the water level, the wave height and the wave period at 600 ft in front of the structure are known. Note that the geometry itself is fixed for the six cases. The reader is referred to Appendix C for a description of the Boussinesq model and a complete overview of the Boussinesq runs.

Here, we present a comparison between the empirical approach and the Boussinesq results for Case 1, 3, 4, and 5. Case 2 is not evaluated because this levee/floodwall combination cannot be evaluated with the present TAW formulations in a straightforward way. If an empirical approach is used in this case, much expert judgment has to be included to present an answer. Note that the results in the Boussinesq lookup table also include empirical information (i.e. empirical formulation of Franco & Franco, 1999) because the Boussinesq model cannot handle vertical walls and a full Navier-Stokes model is needed for this case. The advantage of the Boussinesq model in this case is to have an approximation of the wave height just in front of the vertical wall. Case 6 is very similar to Case 5 and is therefore not evaluated herein.

A number of Monte Carlo Simulations (MCS) shows that the empirical and the Boussinesq approach come up with the same order of magnitude if the overtopping rate is in the range of 0.001 – 0.1 cfs/ft. Disagreement outside this range between both approaches seems obvious if the background of both approaches is considered. The empirical formulations were fitted against laboratory data and the given range is more or less equivalent with the test range of the experiments. The lower limit of the Boussinesq results is assumed to be 0.001 – 0.005 cfs/ft. Below this value the water layer becomes very thin at the sloping structure and the Boussinesq results are inaccurate (Lynett, pers. comm.). Because the design approach uses a criterion of 0.1 cfs/ft, we will focus our comparison on the range 0.01 – 0.1 cfs/ft.

In this Appendix we show the results of MCS (10,000 runs) using the empirical and the Boussinesq approach. To make a fair comparison three remarks are made:

- We only vary the hydraulic conditions (surge level, wave height, and wave period) in the MCS. The coefficients in the empirical formulations are kept constant and we use the mean values for these parameters. The reason for this is that we are not able to vary similar parameters in the Boussinesq lookup table. The results from the Boussinesq runs have been made with the "best estimate" values as well (e.g. roughness, eddy viscosity, etc.).

- We use for both approaches the same surge level as hydraulic boundary condition. The Boussinesq model computes the local wave set-up near the structure due to wave breaking and therefore the local water level just in front of the structure will be a bit higher. One may wonder if this local wave set-up should be included in the water level for the empirical approach. The TAW manual does not give a clear answer, but suggests using the water level at the toe of the structure. At that point, the effect of the wave setup appears to be minimal according to the Boussinesq results. Hence, we use the same values for both approaches.
- For Case 3 (New Orleans Lakefront Levee) it appears that the overtopping rate is far below the range of 0.01 – 0.1 cfs/ft using the 1% numbers. The Boussinesq runs have been made for a fixed geometry. Therefore, the 1% design values have been adjusted for this case to give results in 0.01 – 0.1 cfs/ft range.

The results of the comparison for case 1, 3, 4, and 5 are discussed subsequently in the next sections H .2 to H.5. This appendix closes with a discussion of these results in Section H.6.

All elevations described herein are in North American Vertical Datum 1988 (2004.65).

Case 1: Lakefront Airport Floodwall

The geometry of the Lakefront Airport Floodwall is shown in Figure 1. Note that the overtopping rate in the Boussinesq lookup table is computed for different wall heights using the empirical equation of Franco & Franco (1999). In the empirical approach, a vertical wall is assumed with an average bottom level of 4.0 ft in front of the structure. The 1% design values (mean values / standard deviation) that are applied for this case are summarized in Table 1. Because it is a wall, we evaluate the future conditions for this case (2057). The results of the MCS are presented in Figure 2 for both approaches.

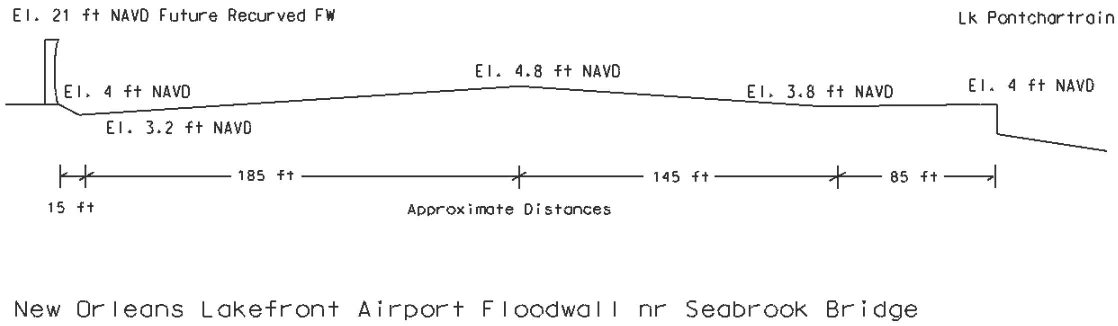


Figure 1 Cross-section Lakefront Airport Floodwall.

	Empirical Approach	**Boussinesq Run**
Surge Level	10.4 / 0.8 ft	10.4 / 0.8 ft
Significant Wave Height	2.6 / 0.3 ft (depth-limited)	7.5 / 0.8 ft
Peak Period	7.8 / 1.5 s	7.8 / 1.4 s
Levee Height	14 ft	See (floodwall 14ft)
Composite Slope	-	
Berm Coefficient	-	

Table 1 – 1% Design Values Lakefront Airport Floodwall (mean values/standard deviation)

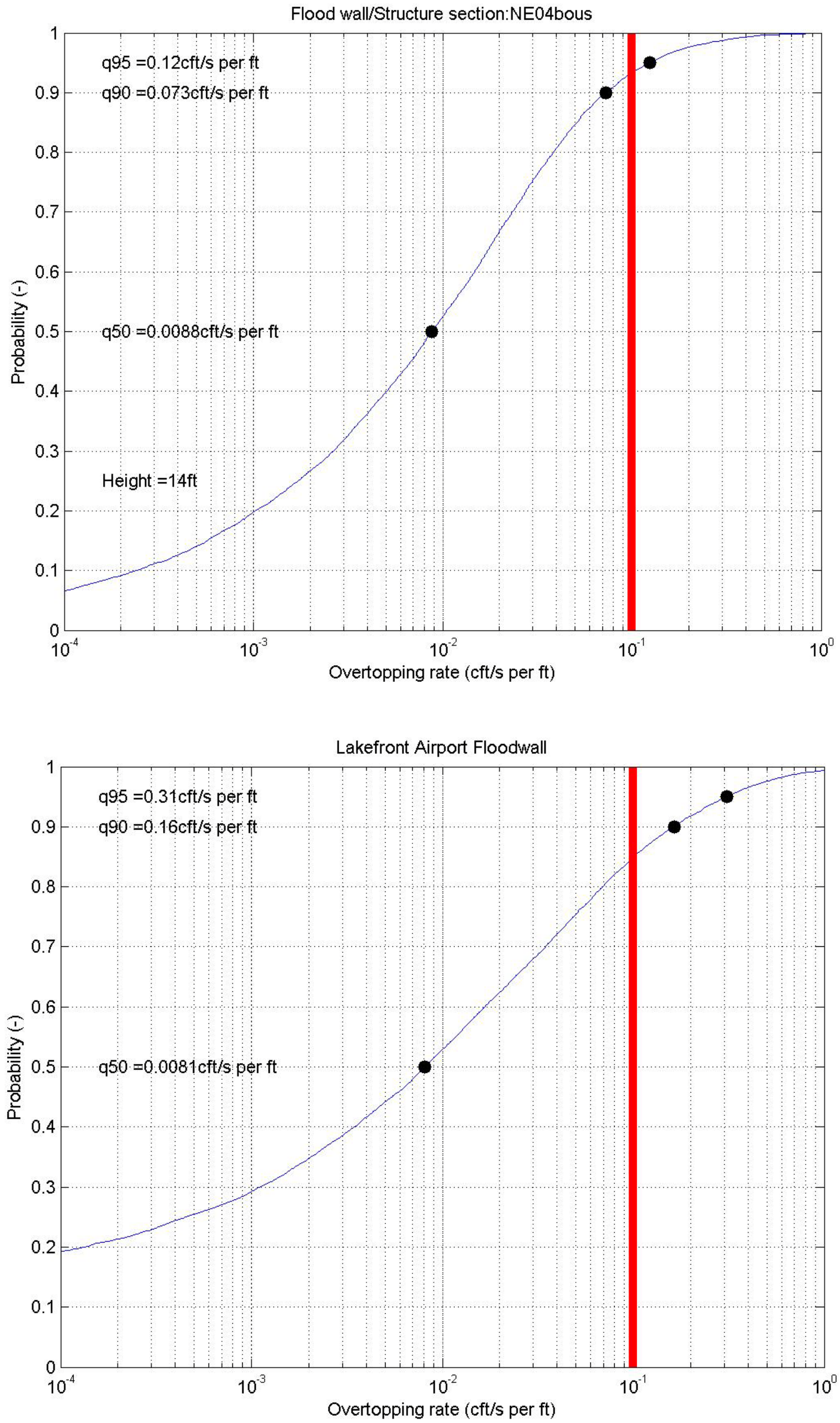


Figure 2 – Result from MCS using the empirical formulations from the TAW manual (upper panel) and using the Boussinesq results (lower panel) for Lakefront Airport Floodwall.

Case 3: New Orleans East Lakefront Levee

The geometry of the New Orleans East Lakefront Levee is shown in Figure 3. The 1% design values for the existing conditions (2007) are not directly used because these values result in very low overtopping values using both approaches (<< 0.01 cfs/ft). Hence, the water level has been increased in the MCS for both approaches with 5.0 ft. The new values used are summarized in Table 2. The results of the MCS are presented in Figure 4.

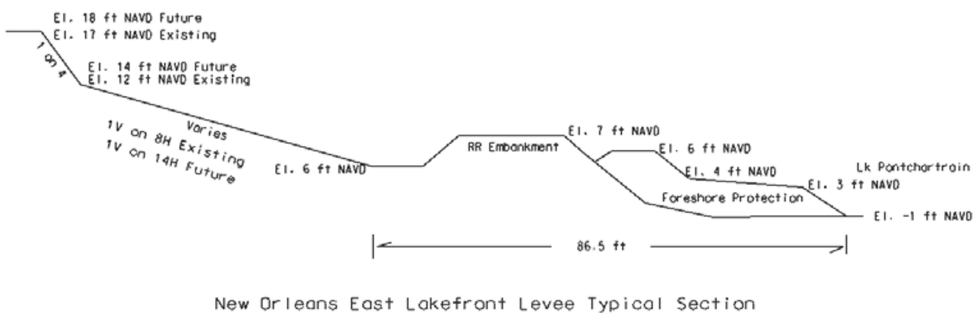
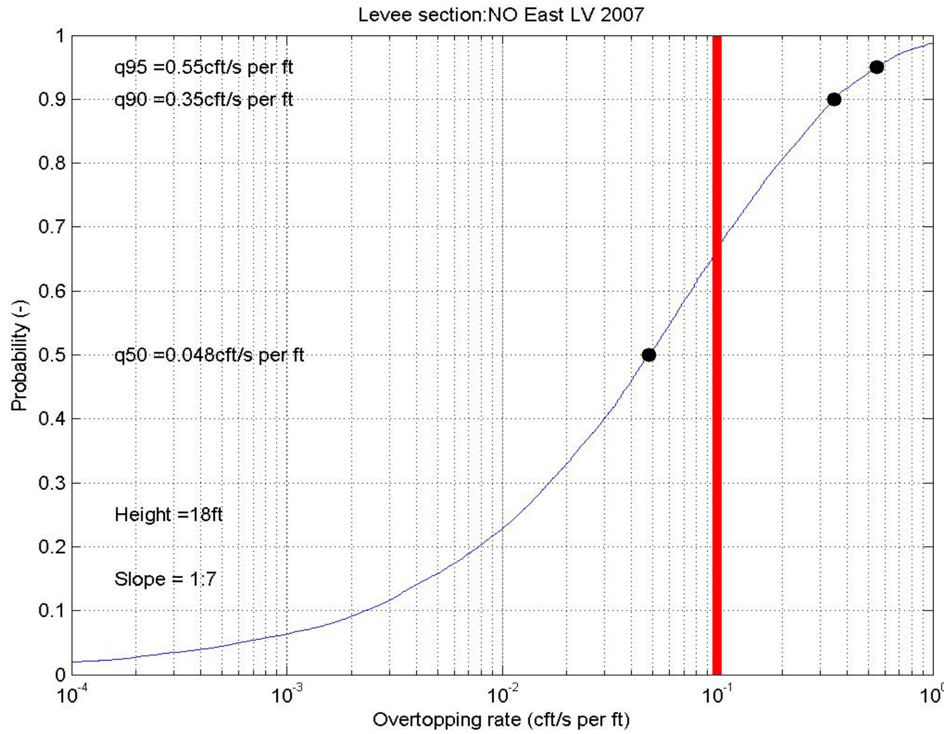


Figure 3 – Cross-section New Orleans East Lakefront Levee

	Empirical Approach	**Boussinesq Run**
Surge Level	13.9 (increase 5ft) / 0.8 ft	13.9 (increase 5ft) / 0.8 ft
Significant Wave Height	6.1 / 0.6 ft (depth-limited)	6.6 / 0.66 ft
Peak Period	6.7 / 1.34 s	6.7 / 1.34 s
Levee Height	18.0 ft	See Figure (future conditions)
Composite Slope	1:7	
Berm Coefficient	0.7	

Table 2 – 1% Design Values New Orleans East Lakefront Levee



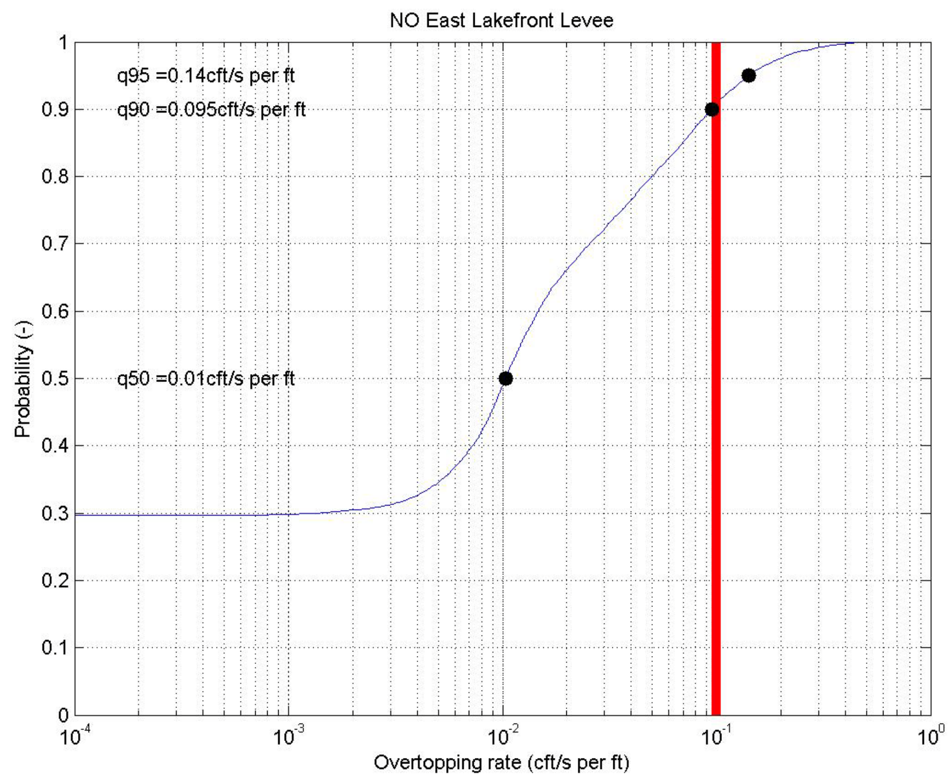


Figure 4 – Result from MCS using the empirical formulations from the TAW manual (upper panel) and using the Boussinesq results (lower panel) for New Orleans East Lakefront Levee.

Case 4: Jefferson Parish Lakefront Levee

The Jefferson Lakefront Levee is shown in Figure 5. The 1% design values are applied without adaptation and summarized in Table 3. The results of the MCS are presented in Figure 6.

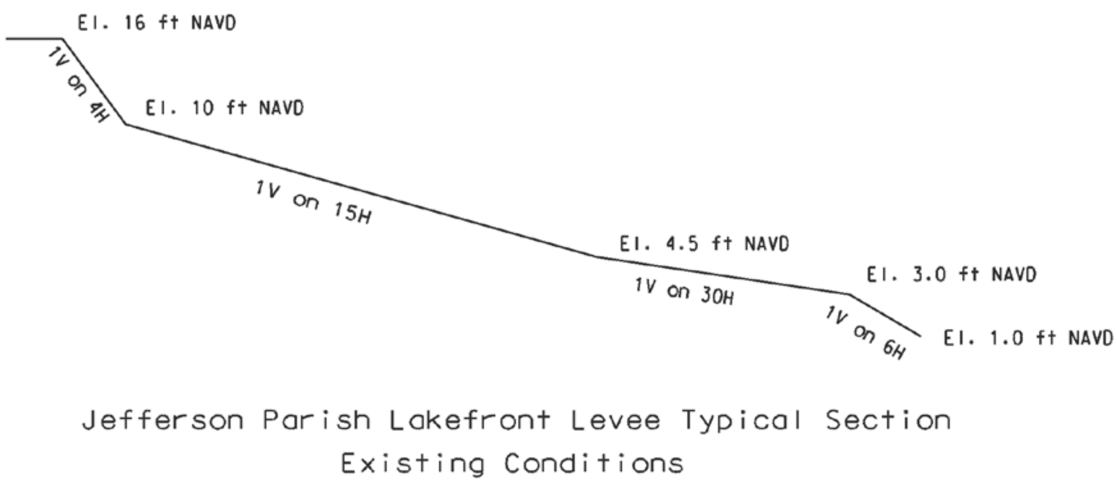


Figure 5 – Cross-section Jefferson Parish Lakefront

	Empirical Approach	**Boussinesq Run**
Surge Level	9.9 / 0.8 ft	9.9 / 0.8 ft
Significant Wave Height	4.0 / 0.4 ft (depth-limited)	7.4 / 0.74 ft
Peak Period	7.8 / 1.56 s	7.8 / 1.56 s
Levee Height	16 ft	See Figure 5
Composite Slope	1:4	
Berm Coefficient	0.65	

Table 3 – 1% Design Values Jefferson Parish Lakefront

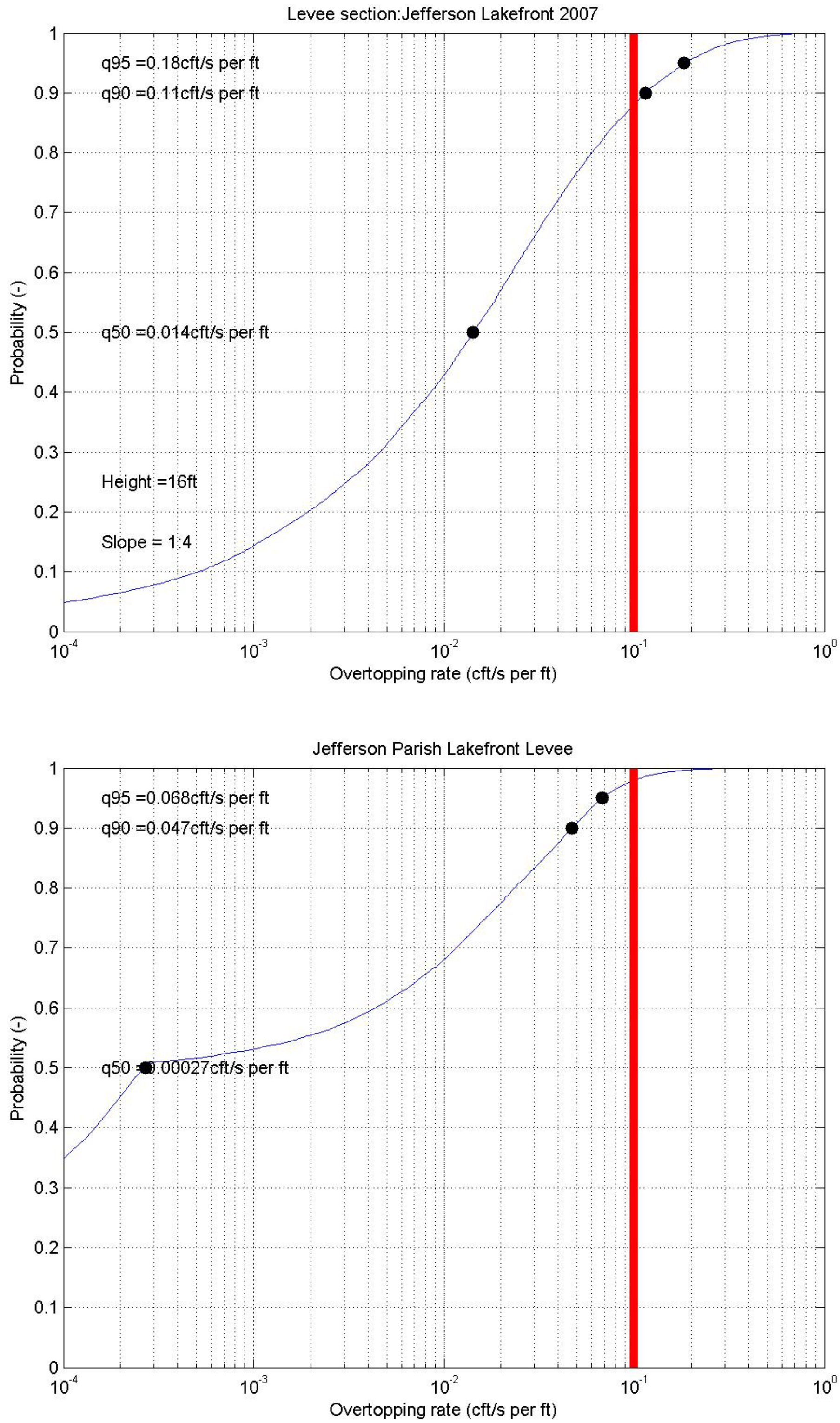


Figure 6 – Result from MCS using the empirical formulations from the TAW manual (upper panel) and using the Boussinesq results (lower panel).

Case 5: New Orleans Lakefront Levee

The geometry of the New Orleans Lakefront Levee is shown in Figure 7. In this case the berm length is 85 ft. The 1% design values for the existing conditions (2007) are directly applied except for the surge level (Table 4). The surge level has been increased with 1ft to make sure that the 90% -overtopping rate is within the 0.01 – 0.1 cfs/ft range. The results of the MCS are presented in Figure 8.

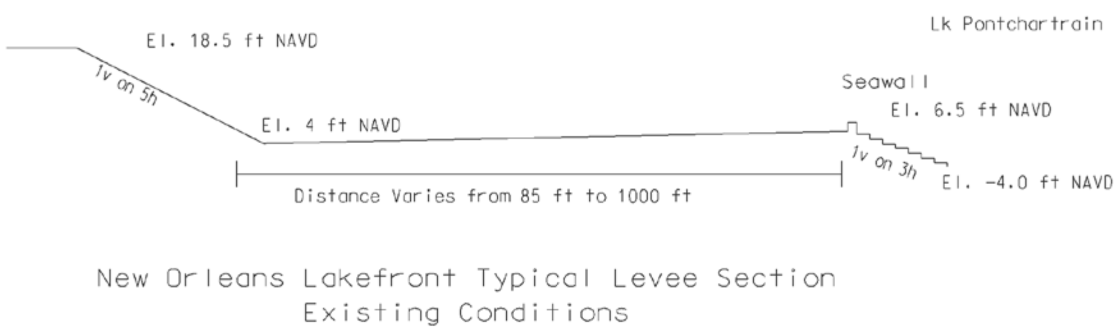
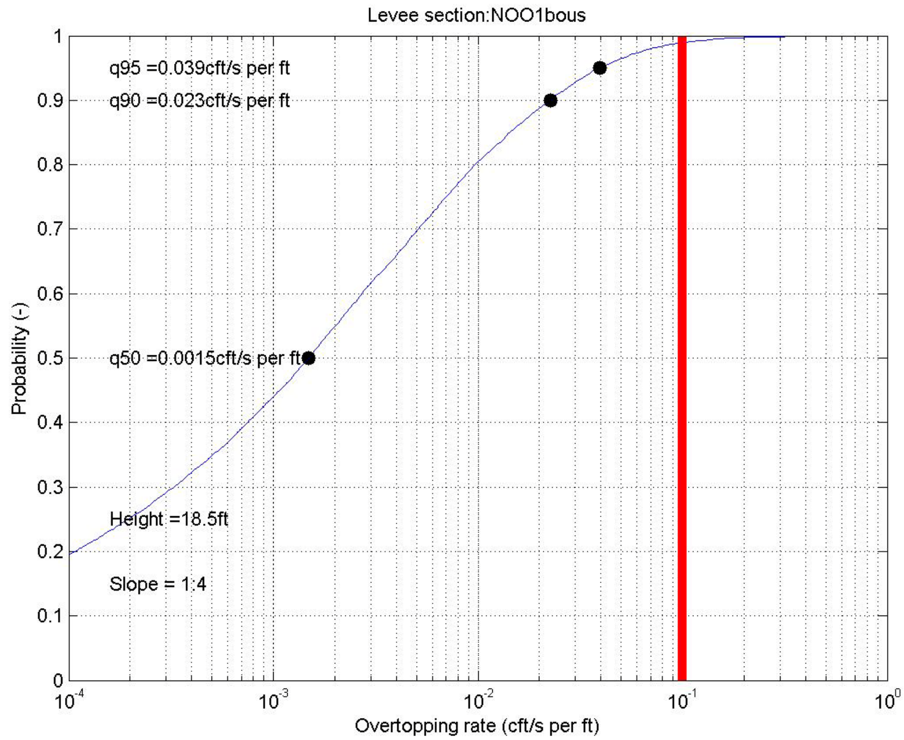


Figure 7 – Cross-section New Orleans Lakefront (the applied berm length is 85ft)

	Empirical Approach	**Boussinesq Run**
Surge Level	10.3 / 0.9 ft	10.3 / 0.9 ft
Significant Wave Height	5.3 / 0.5 ft (depth-limited)	8.1 / 0.81 ft
Peak Period	7.2 / 1.44 s	7.2 / 1.44 s
Levee Height	18.5 ft	See Figure
Composite Slope	1:5	
Berm Coefficient	0.6	

Table 4 – 1% Design Values New Orleans Lakefront Levee



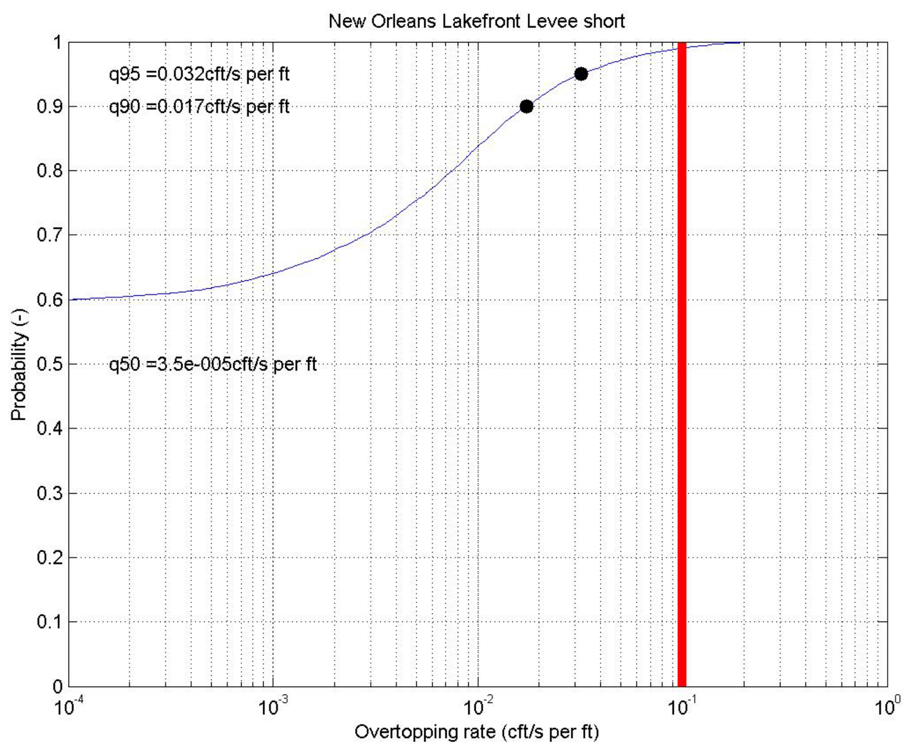


Figure 8 – Result from MCS with empirical approach (upper panel) and Boussinesq approach (lower panel)

Discussion of Results

The previous sections show a comparison between the results from a Boussinesq and an empirical approach to derive levee or floodwall heights for four cases. The results of these cases are summarized in the table below:

Case	Empirical Approach (q_{50} / q_{90})	Boussinesq (q_{50} / q_{90})	Difference in 90%-Overtopping Rate
1. Lakefront Airport Floodwall	0.0088 / 0.073	0.0081 / 0.16	2
3. New Orleans East Lakefront	0.048 / 0.35	0.01 / 0.095	3
4. Jefferson Lakefront Levee	0.014 / 0.11	0.00027 / 0.047	3
5. New Orleans Lakefront Levee	0.0015 / 0.023	- / 0.017	1.5

Table 5 – 50% and 90% overtopping rate according to empirical approach and Boussinesq approach and difference in 90% overtopping rate between empirical and Boussinesq approach.

The results show some remarkable differences and similarities:

- For low overtopping rates (say less than 0.001 cfs/ft), both methods give totally different results. Examples are the 50%-overtopping rate for Jefferson Lakefront levee (Case 3) and the New Orleans Lakefront Levee (Case 4). As already stated at the start of this appendix, both approaches are not accurate for this range of overtopping rates. These differences are not very relevant for the design approach, because the main focus is between 0.01 – 0.1 cfs/ft.
- The empirical approach and the Boussinesq approach result in comparable overtopping rates in the overtopping rates of interest (0.01 – 0.1 cfs/ft) even for complex cross-sections. The differences of the 90%-overtopping rates are limited between a factor 2 – 3.
- The presented cases suggest that the Boussinesq approach results in a lower overtopping rate than the empirical approach.

A difference between say a factor 1.5 – 3 in overtopping rate seems to be high, but should be considered in the perspective of the levee height. It can be shown that:

$$R_{c2} / R_{c1} = 1 - \frac{H_{mo}\xi_o\gamma_b\gamma_f\gamma_\beta\gamma_v}{4.75R_{c1}} \ln(\overline{q}_2 / \overline{q}_1)$$

where R_c is the freeboard, H_{m0} is the wave height and q the overtopping rate (see textbox). The subscript 1 and 2 refer to two different approaches: Boussinesq and empirical approach. For example, with a value of $\frac{R_c}{H_{mo}}$ equal to unity and all of the γ terms except for γ_b which is equal to 0.6 and ξ_o equal to unity, a difference in overtopping rate of a factor 3 (i.e. $q_2 = 3q_1$) results in $R_{c2}/R_{c1} = 0.85$. In other words, the freeboard differs about 15% if the overtopping rate differs a factor 3. The considered freeboard in the design cases are generally in the order of 3.0 – 7.0 ft depending on the incoming wave height. Hence, an overtopping rate difference of a factor 3 results in a difference in levee height of about 0.5 – 1.0 ft.

Summarizing: the final levee or floodwall heights will not be much different using the Boussinesq approach of the empirical approach. Several cases show that the 90%-overtopping rate differs about a factor 1.5 – 3 and the empirical approach appears to be conservative for all cases. In terms of levee height the differences are expected to be 1.0 ft at maximum.

REDUCTION IN OVERTOPPING ASSOCIATED WITH AN INCREASE IN LEVEE ELEVATION (Dean & Edge, 2007)

The equation governing average overtopping rate is:

$$\bar{q} = 0.067 \frac{\sqrt{g H_{mo}^3}}{\tan\alpha} \gamma_b \xi_o \exp\left(-4.75 \frac{R_c}{H_{mo}\xi_o} \frac{1}{\gamma_b \gamma_f \gamma_\beta \gamma_v} \right) \tag{1}$$

which can be differentiated with respect to R_c and rearranged to

$$\frac{\partial \bar{q} / \bar{q}}{\partial R_c / R_c} = -4.75 \frac{R_c}{H_{mo}} \frac{1}{\xi_o \gamma_b \gamma_f \gamma_\beta \gamma_v} \tag{2}$$

which represents the proportionate decrease in overtopping for a proportionate increase in levee elevation. For example, with a value of $\frac{R_c}{H_{mo}}$ equal to unity and all of the γ terms and ξ_o equal to unity, increasing the crest elevation by 10% will result in an overtopping decrease by 48%. For γ terms less that unity, the proportionate decrease would be greater.

Eq. (2) is valid for small changes in freeboard, R_c. For larger changes in freeboard, the ratios of freeboard, R_{c2} / R_{c1} to achieve a discharge ratio, $\bar{q}_2 / \bar{q}_1$ can be shown to be

$$R_{c2} / R_{c1} = 1 - \frac{H_{mo} \xi_o \gamma_b \gamma_f \gamma_\beta \gamma_v}{4.75 R_{c1}} \ln(\bar{q}_2 / \bar{q}_1) \tag{3}$$

As an example, to achieve an order of magnitude reduction in $\bar{q}$ with H_{mo} / R_{c1}=1.0 and all of the γ terms and ξ_o equal to unity, the required ratio of freeboards, R_{c2} / R_{c1} = 1.48. Thus, for relatively large reductions in overtopping rates, it is necessary to apply Eq. (3).

Appendix E

Goda Formula Data Input

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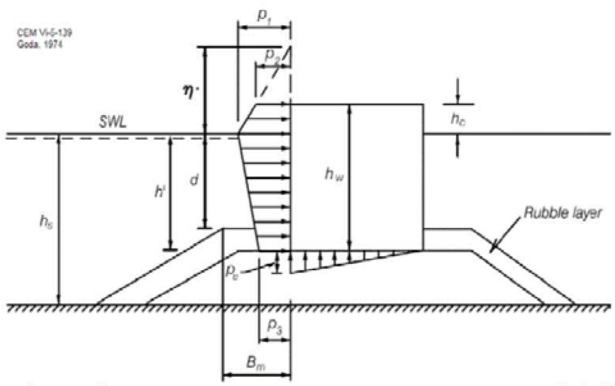
CALCULATION OF WAVE LOADING ON VERTICAL WALL USING CAISSON SCHEMATISATION BY GODA

ACTUAL LOADING (QUANTITY AND DIRECTION) ON STRUCTURES SHOULD BE VERIFIED FOR EVERY SINGLE CASE



Remark	Item	Symbol/abbr.	Unit	
	Top of Wall	TOW	ft*	User Input
	SWL	SWL	ft*	User Input
	Standing water (back side)	SW	ft*	User Input
	Top of Rock	TOR	ft*	User Input
	Top of Fill	TOF	ft*	User Input
	Base Elev	Invert	ft*	User Input
	Width of rock forward of wall	Bm	ft	User Input
	Cotangent of slope of berm	$\cot \theta = m$	-	User Input
	Water depth at foot of structure	h_s	ft	
	Height of wall above fill	h_w	ft	
	Height of wall above SWL	h_c	ft	
	Depth of still water above fill	h'	ft	
	Depth of still water above rock	d	ft	
	Significant Wave Height	H_S	ft	User Input
changed coeff.	Design Wave, see note at the bottom of page	H_{design} = **1.80** H_s	ft	
on 8/22/06	Depth of water at 5Hs from wall	h_b	ft	
	Significant wave period	T	s	User Input
	Wavelength			
	Deep water	$L_o = g\,T^2/2\pi$	ft	
	Shallow water (by iteration, check convergence)			
CEM eqn II-1-10	$$L = \frac{gT^2}{2\pi}\tanh\left(\frac{2\pi d}{L}\right)$$			
	Incident wavelength	L	ft	
	Angle, wave crest to shore	β	degrees	User Input

Modifiers, see CEM, VI-5-139, use 1	λ_1	-	User Input
Modifiers, see CEM, VI-5-139, use 1	λ_2	-	User Input
Modifiers, see CEM, VI-5-139, use 1	λ_3	-	User Input
Gravity, use 32.1719	g	ft/s2	User Input
Density of water, use 1.99	ρ_w	slugs/ft3	User Input
Stochastic variable, see table VI-5-53	U_{FH}	-	
Stochastic variable, see table VI-5-53	U_{MH}	-	

$$\alpha_{I0} = \frac{H_{design}}{d} \; for \; \frac{H_{design}}{d} \leq 2, 2.0 \; for \; \frac{H_{design}}{d} > 2$$ -

$$\alpha_{I1} = \frac{\cos\delta_2}{\cos\delta_1} \; for \delta_2 \leq 0, \frac{1}{\cosh\delta_1 (\cosh\delta_2)^{0.5}} \; for \delta_2 > 0$$ -

$$\delta_1 = 20 \cdot \delta_{11} for \delta_{11} \leq 0, 15 \cdot \delta_{11} for \delta_{11} > 0$$ δ_1 -

$$\delta_{11} = 0.93\left(\frac{B_m}{L} - 0.12\right) + 0.36\left(\frac{h_s - d}{h_s} - 0.6\right)$$ δ_{11} -

$$\delta_2 = 4.9 \cdot \delta_{22} for \delta_{22} \leq 0, 3 \cdot \delta_{22} for \delta_{22} > 0$$ δ_2 -

$$\delta_{22} = -0.36\left(\frac{B_m}{L} - 0.12\right) + 0.93\left(\frac{h_s - d}{h_s} - 0.6\right)$$ δ_{22} -

$$\alpha_I = \alpha_{I0} \cdot \alpha_{I1}$$ α_I -

α_* = α_2 (for irregular waves), α_* = largest of α_2 and α_I (for head on breaking waves) α_* -

$$\alpha_1 = 0.6 + 0.5\left[\frac{\frac{4\pi h_s}{L}}{\sinh\left(\frac{4\pi h_s}{L}\right)}\right]^2$$ α_1 -

$$\alpha_2 = smallest\left\{\frac{h_b - d}{3h_b}\left(\frac{H_{design}}{d}\right)^2, and \frac{2d}{H_{design}}\right\}$$ α_2 -

$$\alpha_3 = 1 - \frac{h_w - h_c}{h_s}\left[1 - \frac{1}{\cosh\left(\frac{2\pi h_s}{L}\right)}\right]$$ α_3 -

eqn VI-5-147	$\eta^* = 0.75(1 + \cos\beta)\lambda_1 H_{design}$	ft	Wave loads are computed for both breaking and non-breaking waves
eqn VI-5-148	$p_1 = 0.5(1 + \cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha_*\cos^2\beta)\rho_w g H_{design}$	lbf/ft²	the higher wave load is used in design
eqn VI-5-149	p_2 = if $\eta^* > h_c$ then $(1 - h_c/\eta^*)p_1$ else 0	lbf/ft²	
eqn VI-5-150	$p_3 = \alpha_3 p_1$	lbf/ft²	
**********	$F_H = (p_1+p_2)/2 * h_c + (p_1+p_3)/2 * h'$	lbf/ft	Note - hydrostatic force and moment are subtracted from the computation
**********	$M_H = (2p_1 + p_3)h'^2/6 + (p_1 + p_2)h'h_c/2 + (p_1 + 2p_2)h_c^2/6$	lbf*ft/ft	The hydrostatic force is computed separately

Notes

1: Design wave height
Design wave height defined as the highest wave in the design sea state at a location just in front of the breakwater. If seaward of a surf zone Goda (1985) recommends for practical design a value of 1.8*Hs to be used corresponding to the 0.15% exceedence value for Raileigh distributed wave heights. This corresponds to H1/250 (mean of the heights of the waves included in 1/250 of the total number of waves, counted in descending order of height from the highherst wave). Goda's recommendation includes a safety factor in terms of positive bias as discussed in Table VI-5-55.

If within the surf zone, Hdesign is taken as the highest of the random breaking waves at a distance 5*Hs seaward of the structure. The CEM is not clear on how this wave heigth should be determined. Taking Hdesign=1.8*Hs also for within the surf zone, will lead to a conservative estimate of the wave forces. To make a more accurate assessment of the wave heigths, the approach presented by Battjes and Groenendijk (Wave height distributions on shallow foreshores, 2000), could be applied. However this approach should be studied in more detail.

2: Goda for irregular waves and for head on breaking waves
The CEM does not provide guidance which Goda schematisation (Goda for irregular waves vs. head on breaking waves) should be used when. Given the present uncertainties in the wave climate, schematisation and loading, it is recommeded to use, for the time being, the schematisation which leads to the highest loading.

3: Comparison spreadsheet with interactive CEM
The results generated with this spreadsheet should correspond with the results generated with the interactive CEM. Elaborate comparisons have been made, but the calculation of the facor alfa_2 in the interactive CEM remains unclear. The interactive CEM for both irregular and head-on breaking waves, using the same parameters, give different results, which can not be the case as the same formulas are used. However the impact on the calculated forces is limited. This issue should be looked into by the programmers of the interactive CEM.

Appendix F

Overtopping Criterion (Author: S. A. Hughes, ERDC)

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Evaluation of Permissible Wave Overtopping Criteria For Earthen Levees Without Erosion Protection

Steven A. Hughes, PhD, PE[1]

Background

Ideally, all levees would have a crown elevation with ample freeboard to prevent wave and/or surge overtopping for any conceivable storm scenario. However, economics dictate more practical levee designs having lower crown elevations, but with the risk that some wave/surge overtopping will occur during extreme events. Design of the South Louisiana levee system to withstand various levels of storm surge and waves requires an understanding of a permissible level of wave overtopping that can be tolerated by a well-constructed, grass-covered earthen levee without sustaining damage to the levee top clay layer.

Earthen levees constructed without slope protection or armoring must rely on the erosion resistance of the outer soil layer during episodes of wave and/or storm surge overtopping. Usually erosion resistance for wave or surge overtopping is most needed on the levee crown and down the rear slope on the protected side of the levee. Levees constructed with a top layer of good clay and well-established vegetation with a healthy root system have much better erosion resistance than top layers of sandy soil with sparse or unhealthy vegetation.

Empirical methods for estimating wave overtopping at coastal structures caused by irregular waves typically give an average overtopping rate for the duration of the specific wave condition and water level. This overtopping rate is a function of the structure freeboard (difference between the levee crown elevation and the surge level), wave characteristics, and levee seaward (flood side) slope. The average overtopping rate can be thought of as the sum of the overtopping water volume contained in all the individual waves that overtop the levee divided by the duration of the wave exposure. Some individual waves will have overtopping volumes (and associated flow parameters) many times the average.

Specifying a permissible average wave overtopping rate for an earthen levee is a difficult undertaking for several reasons:

a) Soil erodibility in flow varies substantially depending on soil type, compaction, vegetation cover, and root system.

b) Localized soil weaknesses may create initial “hot spots” where head cut erosion begins. Expansion of the head cut leads to wider damage.

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c) Local flow accelerations may occur due to other constructions placed on the levee.

d) Flow velocities of overtopping waves depend on the protected-side slope, so levees with milder protected-side slopes can tolerate more wave overtopping than levees with steeper protected-side slopes.

Nevertheless, it should be possible to determine a range of average wave overtopping rates that would safely bracket the variations noted above. This criterion would most likely be established as the threshold for initiation of damage on levees of particular soil type and vegetation cover, and it is important to convey exact specification for the levee soil, grass cover, and necessary maintenance to achieve performance meeting the criterion. Several criteria already exist in the technical literature.

A more problematic issue might be specifying a permissible wave/surge overtopping criterion that combines a damage threshold with duration of exposure. Such a criterion could be described as essentially a wager that storm conditions will subside before levee erosion progresses to the point that significant damage occurs. The payoff is reduced levee heights in exchange for increased maintenance after major storms. However, losing the wager has far greater consequences than designing against initiation of damage. For this reason any allowable wave overtopping criterion that includes overtopping duration must be supported by significant engineering studies.

Study Objectives

The primary objective of this study was to examine critically existing permissible wave overtopping criteria for unprotected earthen levees. In addition, established criteria for embankment erosion by steady flow overtopping of weirs and dams were examined, and a linkage between steady overtopping and average wave overtopping was pursued to boost confidence in the wave overtopping criterion. Finally, gaps in knowledge were identified, and suggestions were made for improving the permissible wave overtopping criterion to add greater confidence to risk assessment of the South Louisiana levee system.

Average Wave Overtopping Criteria

The time-varying discharge from waves overtopping a coastal structure is unevenly distributed in both time and space with the volume of overtopping water differing considerably between waves. Where the storm surge level is lower than the levee crown elevation, the major portion of the overtopping discharge is due to a small proportion of larger waves. Studies have shown that local overtopping discharge per unit levee length from individual waves can be more than 100 times the average overtopping rate (van der Meer and Janssen 1995).

Several coastal engineering design guidance publications contain a table showing critical values of average wave overtopping discharges. For example, the Coastal Engineering Manual (Burcharth and Hughes 2002) on Table VI-5-6 shows levels of overtopping discharge with columns for vehicular and pedestrian safety, and various levels of structural damage for buildings, embankments and seawalls, grass sea dikes, and revetments as shown on Figure 1. This table was compiled from several published sources dating as far back as 1968.

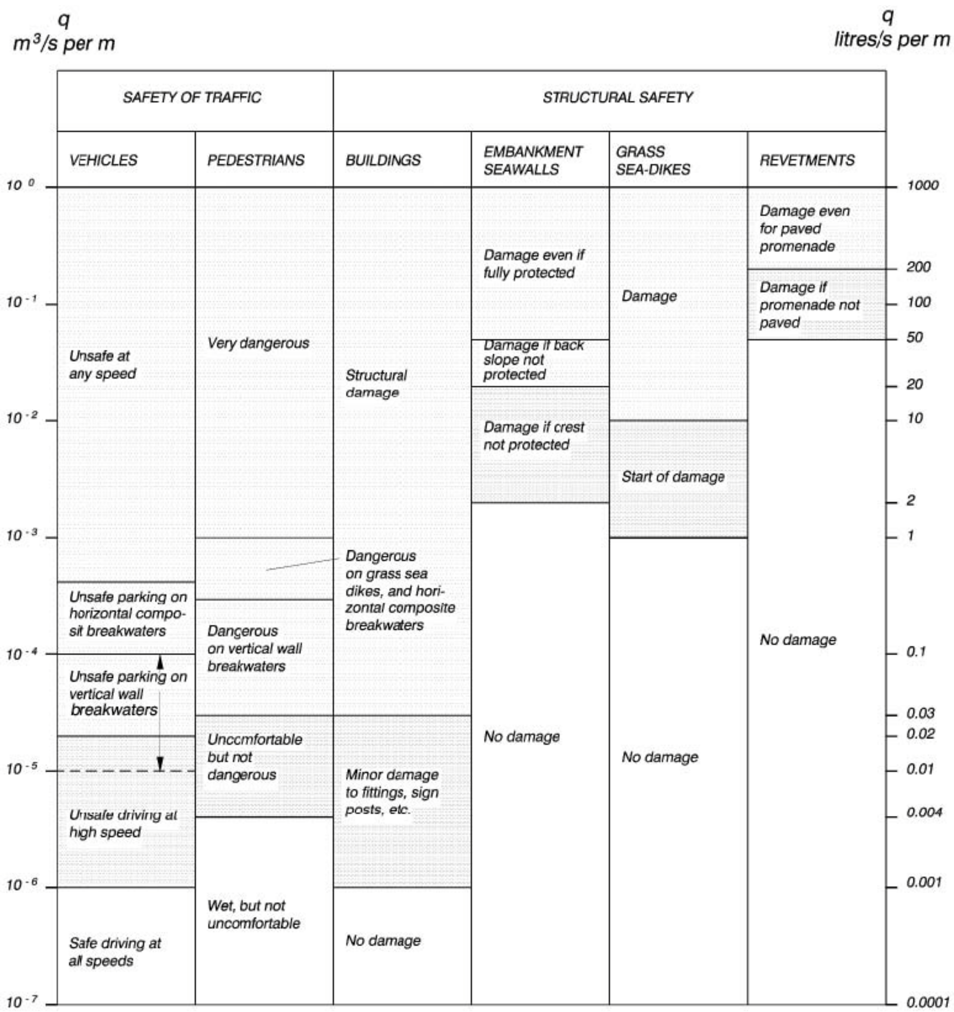

Figure 1. Table of permissible overtopping from the Coastal Engineering Manual

The original author of the table was not identified during the course of this investigation, but some aspects of the table evolution were uncovered. An earlier version of the permissible overtopping table appeared in the "Rock Manual" (CIRIA/CUR 1991) without attribution. Van der Meer (1993) noted that most of the permissible overtopping values in the table referred to "old Japanese data," and he augmented the table by adding overtopping values for vehicles and pedestrians on vertical walls from de Gerloni, et al. (1991) and pedestrians on grass dikes from work conducted in the Delta flume. Van der Meer's (1993) version of the table was reproduced unchanged by d'Angremond and van Roode (2001). The version of the table shown on Figure 1 from the Coastal Engineering Manual (CEM) included all the information contained on van der Meer's (1993) version of the table with an additional column for grass sea-dikes. The grass sea-dike information was previously reported in van der Meer and Janssan (1995) and TAW (1989). Undoubtedly the table appears in other literature as well.

A cautionary note about this table is included in the CEM that reads, in part…
"The values given in this table must be regarded only as rough guidelines because, even for the same value of q [average wave overtopping], the intensity of water hitting a specific location is

very much dependent on the geometry of the structure and the distance from the front of the structure. Moreover, what is regarded as acceptable conditions is to a large extent a matter of local tradition and individual opinions."

This statement probably pertains more to the overtopping danger posed to pedestrians and vehicles than to erosion of the leeward structure slope, but the caution is still relevant.

Table 1 below presents ranges of average wave overtopping discharge damage criteria extracted from CEM Table VI-5-6 (Figure 1) that have applicability to overtopping of unprotected earthen levees (and perhaps floodwalls located on top of levees). Average wave overtopping is given as volumetric discharge per unit length of structure in both metric and equivalent customary English units. The reference column gives representative sources for the suggested overtopping criteria.

Table 1. Irregular Average Wave Overtopping Damage Criteria

Situation	Metric Units (m^3/s per m)	English Units (ft^3/s per ft)	References
Grass Sea Dikes			
Start of damage	0.001 – 0.01	0.011 – 0.11	TAW (1989), van der Meer and Janssan (1995)
Embankments and Seawalls			
Damage if crest not protected	0.002 – 0.02	0.022 – 0.22	Goda (1971, 1985)
Damage if back slope not protected	0.02 – 0.05	0.22 – 0.54	Goda (1971, 1985)

In the subsections below the genesis for the average overtopping is examined to the extent possible in order to provide a better understanding on how the values were established and to determine potential uncertainties in the damage criteria that might be improved with focused studies. Certainly key literature references have been missed, so this review should not be considered definitive nor exhaustive.

Dutch Criterion for Grass Sea Dikes

The wave overtopping criterion for initiation of damage on grass-covered earthen dikes was included in the Dutch Guideline for river dikes (TAW 1989). The guidance was summarized by van der Meer and Janssen (1995), and it has been reproduced in Table 2. The range given in Table 2 that includes "Clayey soil with relatively good grass" and "Clay protective layer and grass according to the standards…" is the range demarked on the Figure 1 table for "Start of Damage" in the Grass Sea-Dikes column.

Table 2. Dutch Guidelines for Average Wave Overtopping on Grass-Covered Sea Dikes		
Situation	**Metric Units (m^3/s per m)**	**English Units (ft^3/s per ft)**
Sandy soil with a poor turf	0.0001	0.0011
Clayey soil with relatively good grass	0.001	0.011
Clay protective layer and grass according to the standards for an outer slope (or with revetment)	0.01	0.11

More recently, van der Meer, et al. (2006) noted that only a few Dutch guidelines on strength of inner slopes of dikes, levees or embankments exist, and all of them were developed for steady overflow of water and not wave overtopping. Van der Meer, et al. went on to state that information contained in CIRIA report 116 (Hewlett, et al. 1987) was *"reworked to wave overtopping in The Netherlands, but without validation."* This statement suggests that the present Dutch guidelines given in Table 2 are based on a theoretical correspondence between average wave overtopping and steady flow overtopping rather than observation of dike damage due to wave overtopping. No reference has been found that describes a technique used to relate permissible steady flow overtopping to comparable average wave overtopping (if, in fact, such a relationship was developed prior to appearance of the guidelines).

Young and Hassan (2006) noted that "*Current design practice for the inner slope still relies on criteria, set largely from experience and judgement, for allowable overtopping discharge*." And they state that the graphs presented by Hewlett, et al. (1987) were used to determine erosion resistance of grass subjected to wave overtopping. Young and Hassan (2006) applied the procedures outlined by Schüttrumpf and van Gent (2003) to estimate overtopping flow parameters associated with a range of wave conditions and heavy overtopping. They compared the estimated velocities and durations with the duration curves of Hewlett, et al. (1987) and concluded the criteria based on the steady overtopping flow curves were not safe for short-duration, high velocity flows on steep dike slopes. The main focus of Young and Hassan's paper was determining the probability of failure associated with stability of the turf layer against sliding over the underlying clay layer. (The overtopping flow estimation methods of Schüttrumpf and van Gent are described in more detail in the section below titled, *Estimation of Wave Overtopping Flow Parameters*).

The CIRIA report 116 (Hewlett, et al. 1987) referenced by van der Meer, et al. (2006) and by Young and Hassan (2006) focuses primarily on stability against steady water overflow of backside (protected side) levee slopes. They examined backside slopes protected with either grass or a variety of slope reinforcement schemes such as placed blocks, turf reinforcement mats, etc. A short section of the report discussed wave overtopping with graphics illustrating wave overtopping where the surge level is lower than the levee crest elevation, and where the surge level exceeds the levee crest. Hewlett, et al. (1987) noted in reference to irregular wave overtopping...

"…overtopping discharge at any location will be unsteady and, although the concept of using reinforced grass as protection on the downstream face is still valid, the value of peak design discharge for the waterway is a matter of engineering judgment. Owing to the random nature of wind-generated waves, the local peak discharge intensity when a particular section of the embankment is overtopped by a large wave could be between one and two orders of magnitude larger than the time-averaged mean discharge intensity."

Hewlett, et al. (1987) listed the permissible values of average wave overtopping given by Goda (1985), and they stated (without reference) that Dutch practice was to use a maximum value of q = 0.002 m^3/s per m (0.022 ft^3/s per ft) for grassed slopes. Hewlett, et al. (1987) gave design curves for erosion resistance of plain and reinforced grass for the case of steady flow overtopping (see Figure 2 below). The curves, based partly on field experiment and observation, related steady limiting flow velocity to flow duration for poor, average, and good cover of plain grass. It is presumed that these steady flow limiting velocity curves form the basis for the present Dutch guidelines as given by TAW (1989) and van der Meer and Janssen (1995). The section below titled, Steady Flow Overtopping Criteria gives greater detail on the developmental history of the steady flow curves given by Hewlett, et al. (1987).

Goda's Criteria for Embankments and Seawalls

The wave overtopping damage criteria listed in Table 1 for embankments and seawalls is based on studies performed by Y. Goda in Japan with the principal English reference being Goda (1985). This guidance is presented in Figure 1 as the column labeled, "Embankment/Seawall."

Professor Goda analyzed damaged and undamaged cases of 20 coastal dikes and 5 seawalls exposed to typhoon waves. Most of the structures were located within bays, and storm duration was limited to a few hours. Goda personally inspected some of the damaged structures after the Ise-Bay Typhoon of 1959, and he analyzed the remainder using technical reports that described the design conditions and damage state. The damage modes depended on the structural type. In some cases coastal dikes disappeared over the length of several hundred meters (Goda, personal communication, 2007a).

Goda estimated the wave overtopping rate for each case (details below) and combined the estimates with his observations and analysis to produce the tolerable wave overtopping rates given in Table 3. This information was originally reported in Goda (1970) in Japanese, and it appeared a year later in English (Goda 1971). The 1971 paper includes a plot showing the average wave overtopping estimates for the 25 cases. The damage categories of "none, little, breach, and collapse" were identified for each case data point. The table of tolerable overtopping rates was reproduced in Goda's widely available book (Goda 1985). Qualitative descriptions of damage beyond the tolerable overtopping limits for the different structure types were provided by Professor Goda in a personal communication (Goda 2007a) and included in Table 3.

Table 3. Goda's Tolerable Wave Overtopping Limits for Structural Safety		
Situation/Damage	**Metric Units (m^3/s per m)**	**English Units (ft^3/s per ft)**
Coastal Dike		
Concrete on front slope, with soil on crown and back slope (damage: total collapse)	< 0.005	0.054
Concrete on front slope and crown, with soil on back slope (damage: washing away of back slope and total collapse)	0.02	0.22
Concrete on front slope, crown and back slope (damage: collapse of parapet, failure of crown and total collapse)	0.05	0.54
Revetment		
No pavement on ground (damage: heavy scouring of ground, collapse of seawall, etc.)	0.05	0.54
Pavement on ground (damage: overbreakage of parapet walls, cracking and/or partial subsidence of pavement, etc.)	0.2	2.15

Two disparities are seen between Goda's (1985) values as given in Table 3 and the values given on Table 1 taken from the CEM and several earlier publications. First, the lower limit of $q < 0.005\ m^3$/s per m for coastal dikes with unprotected crown and backside slope is given as a lower value of $q = 0.002\ m^3$/s per m in the CEM. However, Goda (1985) did cite a case of a coastal dike exposed to the open ocean on the Niigata Coast that lost part of its sand fill and suffered slumping of concrete paving blocks on the crown due to wave suction. Wave overtopping for this specific case was estimated to be only 0.002 m^3/s per m, and this is possibly the source for the lower value reported in the CEM and other places.

The second difference is that the CEM (see Table 1) reports the permissible wave overtopping range of $0.02 \le q \le 0.05\ m^3$/s per m for coastal dikes having an unprotected soil backside slope, whereas Goda (1985) specified the lower discharge of the range ($q = 0.02\ m^3$/s per m) for unprotected soil slopes and the upper discharge of the range ($q = 0.05\ m^3$/s per m) for backside slopes protected by concrete.

Professor Goda (2007b) reported the following about Japanese design practice:

> "The Ports and Harbor Bureau of the Ministry of Land, Infrastructure, and Transport of Japan has been using the threshold of 0.01 m^3/s per m (0.11 ft^3/s per ft) for design of seawalls for urban areas for more than 30 years. For the area less inhabited the tolerable

rate is usually taken at 0.02 m^3/s per m (0.22 ft^3/s per ft). However, the River Bureau of the same ministry, which is responsible for general coastal areas, has maintained its philosophy of designing seawalls with wave runup heights mostly based on old regular wave tests."

The fact that the Japanese have not felt the need to revise the tolerable wave overtopping guidelines in over 30 years lends additional credibility to the criterion.

Estimation of wave overtopping rate. Tsuruta and Goda (1968) compared small-scale laboratory measurements of irregular wave overtopping at a vertical wall to predictions based on the irregular wave height distributions and linear superposition of regular wave overtopping results. Good agreement was found. This led to development of two diagrams relating irregular wave parameters to average wave overtopping for a vertical wall and for a vertical wall with a sloping rubble-mound absorber in front. Waves were assumed to be Rayleigh-distributed, and the curves were constructed as the weighted mean of the regular wave overtopping curves (Goda 2007a). It was noted in Goda (1971) that scatter in the data indicated the curves are best used as "an order-of-magnitude estimate only." These wave overtopping prediction curves were used to estimate the overtopping rates for the criteria proposed in Goda (1970, 1971). Although coastal dikes had front slopes ranging from 1:0.5 to 1:3.5, the design diagram for vertical seawalls was applied (Goda 1971, 2007a). An advanced version of the wave overtopping prediction curves for approach bottom slopes of 1:10 and 1:30 were included in Goda (1985).

Measured wave data during the typhoons were not available at any of the damage sites studied by Goda. Therefore, wave conditions used for estimating average overtopping rates at each site were taken from descriptions in the technical reports used for the damage study. These wave estimates were all hindcast using estimates of the wind parameters, and Goda implies he was conservative when using the reported wave heights in his analysis (Goda 2007b).

Potential errors in estimating the typhoon wave parameters using wind data add some uncertainty in Goda's wave overtopping criteria. The damage state of the structures is undoubtedly accurate, and the estimates of average wave overtopping are reasonably reliable for the input wave conditions. However, overtopping for coastal dikes was estimated using curves for vertical walls with a rubble-mound absorber in front. Intuitively, these overtopping estimates would be expected to be less than the overtopping that occurs for the same wave condition on a levee with a smooth, impermeable slope on the seaward side.

Structure freeboard is determined as the vertical difference between structure crest elevation and the surge level. Errors in estimating the combined effects of storm surge level and any associated wave setup would directly impact estimates of average wave overtopping. For example, if the surge levels were underestimated, then the calculated average overtopping would be less than what actually caused the documented damage.

Goda used storm surge values given in the damage and rehabilitation reports, and he recollects being reasonably confident in the reported values (Goda 2007b). The tradition in Japan after typhoons is to determine surge levels by surveying inundation traces on the leeside of buildings where wave action was less. Tide gauge records were available for damage episodes documented for the Ise-Bay Typhoon of 1959 (Goda 2007b).

Finally, Goda (1985) cautioned that the criteria given in Table 3…

> "…are applicable to seawalls built along embayments and exposed to storm waves a few meters high which continue for a few hours only, since most of the seawalls examined belong to this category. It is believed that the tolerable limit should be lowered for seawalls facing the ocean and exposed to the attack of large waves, or for seawalls subject to many hours of storm wave action."

Goda (1985) also urged caution when applying the tolerable overtopping criteria…

> "The amount of damage to a coastal dike of the earth-filled sloping type by wave overtopping is largely dependent on the size of gaps existing between the earth fill and the armor surfaces of the sloping face and crown [referring to armored dikes]. The setting of tolerance limits according to structural type may be too crude without consideration of the particular construction conditions, but it is hoped that the criteria will serve as a guideline for design engineers. **The user is encouraged to consider some lowering of the values, taking into account the magnitude of the wave height and the duration of the storm waves."**

Recent Research Related to Wave Overtopping Erosion

Van der Meer, et al. (2006) noted that tests conducted by Smith (1994) in the Delta flume with average wave overtopping discharge up to 0.025 m^3/s per m (0.27 ft^3/s per ft) did not show damage after many hours of testing. The dike inner slope was 1:2.5 covered with grass in good condition with good clay. This value of average wave overtopping from the experiment is over twice the value given in Table 2 for a "clay protective layer and grass according to the standards for an outer slope good grass on a clay soil," and the backside slope is slightly steeper than used in the New Orleans levee system, so flows would be slightly faster.

Much credence must be given to the permissible average overtopping found by Smith (1994) because it was obtained directly from tests conducted at full scale under controlled conditions, and it is the first full-scale controlled test of grass-covered slope resistance to wave overtopping. However, this overtopping value represents the ideal condition of healthy grass and good root system, and the permissible wave overtopping should be decreased where grass is not as healthy, or in a dormant condition such as wintertime.

Möller, et al. (2002) conducted full-scale wave overtopping tests in the large wave flume in Hannover, Germany. The dike structure had a 1:6 flood-side slope, a 2-m-wide crown, and a 1:3 backside slope. The backside slope was constructed of compacted fresh clay without any grass covering. The intent of the experiment was to verify a theoretical model of the overtopping flow process, and to measure erosion and water infiltration on the backside slope. Three types of clay were tested: a very resistant clay with low permeability; an acceptable clay with higher permeability; and an easily eroded sandy clay. Composition of the three clay layers is shown in Table 4. Möller, et al. noted that the erosion process started with washing out of small soil particles leaving irregularities on the surface. These surface irregularities spawned more extensive erosion features such as gullies and holes. The researchers defined the time when erosion gullies appeared on the slope as the "initiation of erosion" because it was easier to identify when this occurred. Table 4 shows the average wave overtopping discharge and time to initiation of erosion for the three tested clays.

Table 4. Results from Möller, et al. (2002) tests.						
	Clay	**Silt**	**Sand**	**Average Wave Discharge**		**Time to Initiation**
				m^3/s per m	**ft^3/s per ft**	
Clay 1	35%	53%	12%	0.001	0.011	2 hrs
Clay 2	20%	45%	35%	0.001	0.011	1 hr
Clay 3	10%	30%	60%	0.0005	0.0054	10 mins

The tests of Möller, et al. (2002) prove that unprotected bare soil on the backside levee slopes has little to no tolerance to wave overtopping, particularly where soils have high sand content.

Van der Meer, et al. (2006) wisely stated that the true value of tolerable average wave overtopping of grass-covered dikes lies somewhere between the values obtained by Smith (1994) and Möller, et al. (2002), i.e., $0.001 < q_{ave} < 0.025$ m^3/s per m (or in English units $0.011 < q_{ave} < 0.27$ ft^3/s per ft).

Steady Flow Overtopping Criteria

Erodibility of grass-covered slopes subjected to steady flow overtopping has been studied in relation to overtopping of dams and design of spillway channels, and some of these results are applicable to steady flow overtopping of earthen levees. The paragraphs below summarize design criteria suggested by various authors and agencies. This is not a complete summary by any means.

Steady Flow Design Curves of Hewlett, et al. (1987)

As mentioned in the preceding sections, the Dutch guidelines for permissible wave overtopping of grass-covered dikes were derived from steady flow overtopping design curves given by Hewlett, et al. (1987). Figure 2 is the diagram from Hewlett, et al. showing erosion resistance for grass and various armoring systems when used in steady flow channels. According to van der Meer, et al. (2006) and Young and Hassan (2006), these curves form the basis for the present Dutch guidelines for permissible wave overtopping. The three curves on Figure 2 for plain grass cover were based, in part, on field experiment and observation, and they are slightly modified versions of similar curves contained in an earlier technical by Whitehead, et al. (1976). The limit state is given in terms of a limiting steady flow velocity combined with duration of flow. Good grass cover was assumed by the authors to be dense, tightly-knit turf established for at least two growing seasons, whereas poor grass cover was described as uneven tussocky grass growth with bare ground exposed or significant portion of weeds.

Hewlett, et al. (1987) stressed that these recommended erosion resistance values are applicable only to grassed waterways with a low permeability subsoil and subjected to unidirectional flow with its associated seepage flow beneath the soil surface. They emphasized that the curves did not apply to direct wave attack on the grass surface such as occurs on the seaward side of levees. For intermittent wave overtopping, the surface flow may be temporarily similar to steady overtopping flow, but development of the seepage flow parallel to the soil surface would not be the same. They also point out four basic requirements for good erosion resistance of grass covers: (1) full and intimate cover of the subsoil surface, (2) reduction of seepage flow parallel to the slope, (3) good integration of the soil/root mat with the underlying soil, and (4) avoiding surface irregularities that cause higher localized drag.

Seijffert and Verheij (1998) reproduced the curves from Hewlett, et al. (1987) shown on Figure 2, and then went on to state, “Grass covers can resist flow velocities of up to 2.0 m/s (6.6 ft/sec) without any problem.” No reference is given for this stated permissible flow velocity, nor is any description given of required grass and soil quality necessary to meet this criterion, but it is assumed they referred to some mean value extracted from Hewlett, et al.’s data as given in Figure 2.

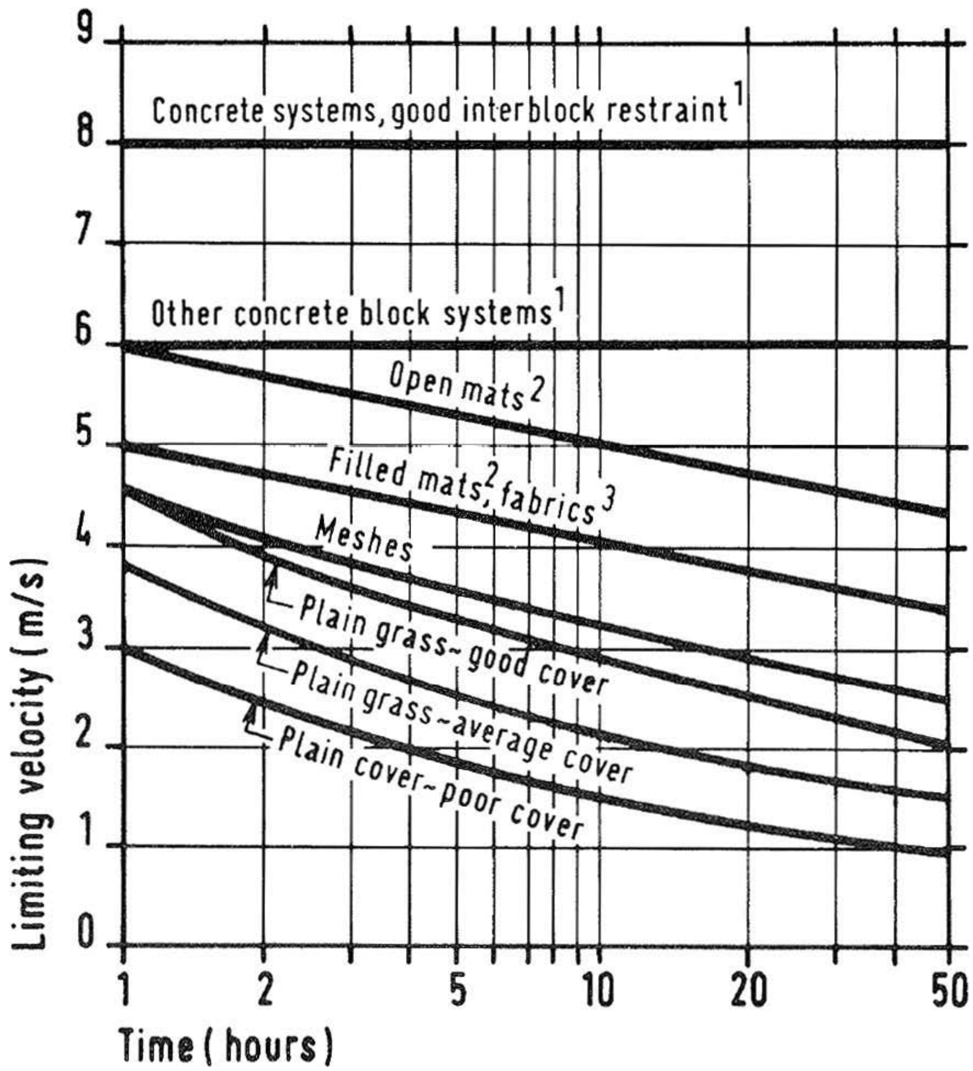

Figure 2. Erosion resistance of plain grass to steady overtopping (Hewlett, et al. 1987)

Steady Flow Design Curves of Whitehead, et al. (1976)

The steady flow design curves from Hewlett, et al. (1987) shown in Figure 2 were derived from similar curves given in an earlier technical note by Whitehead, et al. (1976). The steady flow design curves presented by Whitehead, et al., are shown on Figure 3, and they were based on various laboratory investigations and reports of prototype observations that are documented in the report. The data points shown on Figure 3 are full-scale test data principally from the U.S. Soil Conservation Service, the Water Research Foundation of Australia, and the University of New South Wales Water Research Laboratory. The upper dashed curve is for a *"dense, tightly-knit turf established for at least a year."* The lower dashed curve is for *"an established cover exclusively made up of tussock grasses, or a grass cover of any type established for only 5 to 6 weeks."* The solid center curve was drawn as an average of the two bounding curves. Whitehead, et al. stated that a well-chosen grass cover can withstand flows up to 2 m/s for prolonged periods (more than 10 hrs), between 3 and 4 m/s for several hours, and up to 5 m/s for brief periods (less than 2 hrs).

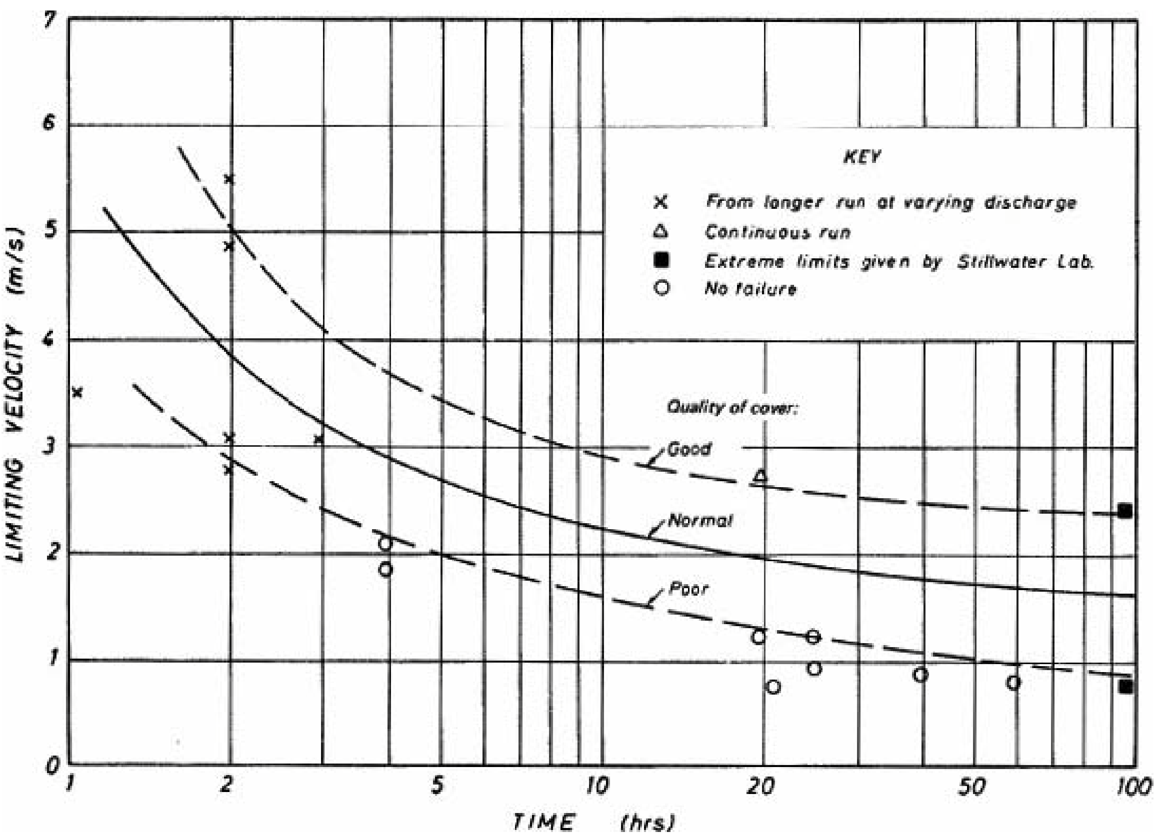

Figure 3. Erosion resistance of grass-lined spillways (Whitehead, et al. 1976)

Comparing the steady flow design curves in Figures 2 and 3 reveals that the later design guidance of Hewlett, et al. (1987) lowered the limiting velocities from those given earlier by Whitehead, et al. (1976). In particular, the lowering is more pronounced on the short-duration end on the left side of the plot. Hewlett, et al. give no reason why this modification was done, but it could be conjectured that new limiting velocity data for turf reinforcement mats and other armoring systems suggested the upper limit for good grass needed to be adjusted downward. In other words, grass should not out-perform the stronger armoring systems. No evidence is given to support this conjecture.

Steady Flow Design Guidance from U.S. Department of Agriculture (1966)

The U.S. Department of Agriculture (USDA 1966) produced permissible steady flow velocities for grassed-lined irrigation channels having mild slopes up to 10% (1:10). The USDA recommendations are shown on Figure 4 (taken from the Virginia Minimum Standard 3.03 Vegetated Emergency Spillway). The USDA guidance stressed that the velocity criteria should not be applied to slopes greater than 1:10. Thus, the values in Figure 4 are not directly applicable to the typically steeper slopes used for the protected sides of earthen levees. Nevertheless, the velocity magnitudes in Figure 4 are similar to the long-duration range (+50 hours) given by Hewlett, et al. (1987) as shown in Figure 2, and it fact, these data are represented as the “Stillwater Lab” data points on Figure 3.

Templeton, et al. (1987) presented a detailed procedure for designing grass lining used in floodways, drainage canals, and emergency spillways. They reanalyzed available data and developed a more generalized “effective stress” semi-empirical procedure that improved the separation of the independent variables in the design relationships. The determined effective stress can be combined with soil erodibility data to given a design procedure with more flexibility than the permissible velocity procedures used previously. Application of Templeton, et al.’s method is best accomplished using a computer program.

Steady Flow Design Guidance from Australia

The following information about permissible steady flow velocities for grass-lined channels was extracted from summaries given in Whitehead, et al. (1976) and not from the original source material. Cornish, et al. (1967) tested four grass species and a pasture mix on a slope of 1:4.5. Kikuyu grass and Rhodes grass withstood velocities of 5.5 m/s before failure; Couch grass failed at flows between 3 and 4 m/s; and the pasture mix failed at 2.7 m/s. In the tests, failure was defined as continuing scour after one hour at a constant velocity, or scour that was unacceptably large.

During tests the flow velocities were increased in increments of 0.6 m/s and held constant at each step for one hour. Whitehead, et al. calculated that the total test durations to failure lasted between 7 and 16 hours without repair to the turf. Eastgate (1969) tested the same grass species on a slope of 1:14 for four hours with flow velocities between 1.5 and 2.0 m/s without sustaining any scour. Table 4 presents maximum allowable velocities for Australian grasses as presented by the Queensland Soil Conservation Service. Table 4 is reproduced from Whitehead, et al. (1976).

Vegetative Cover	Permissible Velocity [2] *(ft/s)*			
	Erosion Resistant Soils [3]		Easily Erodible Soils [4]	
	Slope of Exit Channel		Slope of Exit Channel	
	0-5%	*5-10%*	*0-5%*	*5-10%*
Bermuda Grass Bahiagrass	8	7	6	5
Buffalograss Kentucky Bluegrass Smooth Bromegrass Tall Fescue Reed Canary Grass	7	6	5	4
Sod Forming Grass-Legume Mixtures	5	4	4	3
Lespedeza Weeping Lovegrass Yellow Bluestem Native Grass Mixtures	3.5	3.5	2.5	2.5

[1] SCS-TP-61
[2] Increase values 25 percent when the anticipated average use of the spillway is not more frequent than once in 10 years.
[3] Those with a high clay content and high plasticity. Typical soil textures are silty clay, sandy clay, and clay.
[4] Those with a high content of fine sand or silty and lower plasticity or non-plastic. Typical soil textures are fine sand, silt, sandy loam, and silty loam.

Figure 4. Permissible velocities in vegetated channels (from Virginia Minimum Standard 3.03)

Table 5. Australian Guidelines for Permissible Steady Flow in Grass-Lined Channels			
Cover	**Slope range (%)**	**Maximum Permissible Velocity (ft/s)**	
		Erosion Resistant Soils	**Easily Eroded Soils**
Kikuyu	0 to 5 5 to 10 Over 10	8 8 8	7 7 7
African star grass Couch grass Carpet grass	0 to 5 5 to 10 Over 10	8 7 6	6 5 4
Rhodes grass	0 to 5 5 to 10 Over 10	7 6 5	5 4 3
Rhodes grass on black soil (native)	0 to 5	5	4

Tussock grasses Lucerne Sudan grass	0 to 5	3.5	2.5

Correspondence Between Wave and Steady Flow Overtopping Criteria

A direct comparison between the guidance for allowable average wave overtopping discharge on the protected side of an earthen levee and the allowable steady flow velocity for a sloping embankment would add greater confidence to the present wave overtopping criteria. However, this comparison is not easy to formulate because of the fundamental differences between steady flow and unsteady, periodic flow. This section attempts a comparison by characterizing the peak flow velocities on the protected side levee slope for a specified average wave overtopping discharge.

Estimation of Wave Overtopping Flow Parameters

Experiments have been conducted in Europe at small and large scale with the aim of quantifying the overtopping flow parameters on the inner slope of dike and levees (Schüttrumpf, et al., 2002; van Gent, 2002; Schüttrumpf and van Gent, 2003; and Schüttrumpf and Oumeraci, 2005). These authors developed analytical expressions to represent the velocity and flow depths at the toe of the crest on the flood side, at the toe of the crest on the protected side, and down the backside slope as illustrated in Figure 5.

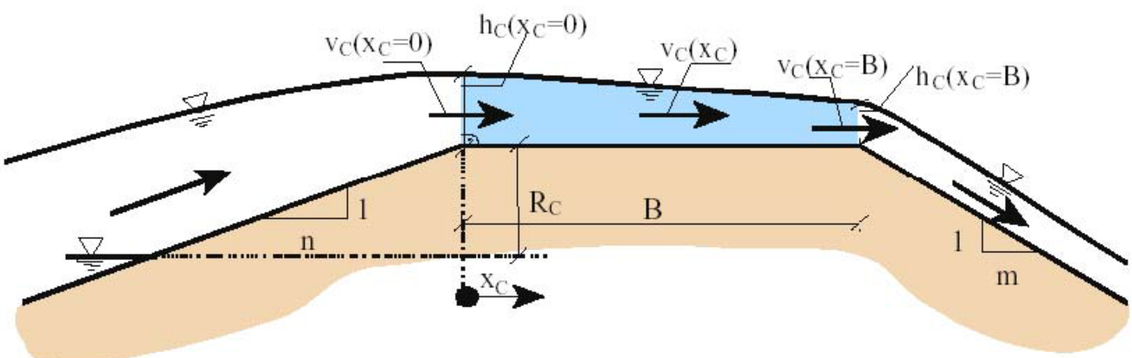

Figure 5. Wave overtopping definition sketch (from Schüttrumpf and Oumeraci 2005)

The key parameters necessary for estimating the flow velocities and depths are the levee freeboard, R_c, the runup elevation exceeded by 2 percent of the waves, $R_{u2\%}$, and a friction factor, *f*, that accounts for frictional energy loss as the overtopping wave travels across the crest and down the protected side slope.

Independent laboratory experiments were conducted in The Netherlands (van Gent 2002) and in Germany (Schüttrumpf, et al. 2002). These two studies produced very similar estimation analysis techniques with only minor differences in the details. A joint paper (Schüttrumpf and van Gent 2003) reconciled the differences to the extent possible.

Van Gent's (2002) small-scale experiments had a 1:100 foreshore slope with a 1:4 slope on the flood side of the dike. Two levee crest widths (0.2 and 1.1 m) were combined with two protected side slopes (1:2.5 and 1:4) to give four different dike geometries using a smooth dike surface. A fifth test series was conducted with a rough surface. Velocity and flow thickness was measured at the toes of the crest and at three locations spaced down the protected-side slope. Micro-impellers were used to measure velocity. Eighteen irregular wave tests were performed for each dike geometry, ten with single-peaked spectra and 8 with double-peaked spectra. Incident wave conditions were determined by measuring the generated waves without the structure in place, and applying the Mansard and Funke (1980) frequency-domain method to remove reflection caused by the dissipating beach profile. Van Gent (2002) used the wave parameter $H_{1/3}$ in the analysis, but did not indicate how this time-domain parameter was determined from the frequency-domain value of H_{mo} found from the reflection analysis. Wave period was specified as mean period $T_{m-1.0}$, and it was estimated from the moments of the incident wave frequency spectra. The mean period is reported to better represent double-peaked spectra.

Schüttrumpf, et al.'s (2002) experiments included both small- and large-scale tests. The small-scale tests utilized three flood-side slopes (1:3, 1:4, and 1:6), a crest width of 0.3 m, and five different protected-side slopes (1:2, 1:3, 1:4, 1:5, and 1:6). A total of 270 tests were run using regular waves and irregular waves conforming to the JONSWAP spectrum. Flow depths were measured with resistance wave gauges, and overtopping flow velocity was recorded using micro-impellers. The large-scale test setup was the same one used for protected-side erosion tests conducted by Möller, et al. (2002). The flood-side slope was 1:6, the crest width was 2 m, and the protected-side slope was 1:3. A total of 250 model tests were run using some regular waves, but mostly irregular waves. Flow depth and velocity were measured using wave gauges and micro-impellers. Wave data were analyzed in the frequency domain using the reflection method of Mansard and Funke (1980). The time-domain wave height parameter $H_{1/3}$ was used in their overtopping analysis with the conversion from the frequency domain wave height given as $H_{1/3} = 0.94\, H_{mo}$ (Schüttrumpf 2006, personal communication). This conversion may have been a typographical error because we should expect $H_{1/3}$ to be greater than H_{mo} for shallow water waves. Also, the conversion is strictly only valid for these tests and not in general because it was determined for wave flume data with a constant water depth for all tests. The wave period was specified as the mean wave period, and it was determined from the calculated incident wave spectra by the simple relationship $T_m = 0.88\, T_p$ (Schüttrumpf 2006, personal communication).

Flow Parameters at the Flood-Side Levee Crest Toe

At the flood-side toe of the levee crest (denoted by the subscript letter A in this report) the flow parameters are given by the equations

$$\frac{h_{A2\%}}{H_s} = c_{Ah2\%}\left[\frac{(R_{u2\%} - R_c)}{H_s}\right] \qquad (1)$$

and

$$\frac{u_{A2\%}}{\sqrt{gH_s}} = C_{Au2\%}\sqrt{\frac{(R_{u2\%} - R_c)}{H_s}} \qquad (2)$$

where

$h_{A2\%}$	-	peak flow depth exceeded by 2% of the waves
$u_{A2\%}$	-	flow depth-averaged peak velocity exceeded by 2% of the waves
H_s	-	significant wave height
$R_{u2\%}$	-	runup elevation exceeded by 2% of the waves
R_c	-	crest freeboard [= crest elevation minus surge elevation]
g	-	acceleration of gravity
$C_{Ah2\%}$	-	empirical depth coefficient determined from test data
$C_{Au2\%}$	-	empirical velocity coefficient determined from test data

The values of $h_{A2\%}$ and $u_{A2\%}$ were determined from the peaks of the overtopping wave time series, and these parameters represent the levels exceeded by only 2% of the total waves during the tests. For example, if a test had 1000 waves, perhaps only 200 waves overtopped the crest. The 2% exceedance level would be the level exceeded by 20 of the 1000 waves (0.02 x 1000), but this is 10% of the overtopping waves. Schüttrumpf, et al. (2002) also provided coefficients for the average overtopping parameters $h_{A50\%}$ and $u_{A50\%}$. All of the equations pertain to the maximum velocity at the leading front of the overtopping wave. Flows associated with a single wave decrease after passage of the wave front.

Note in Eqns (1) and (2) that significant wave height H_s in the denominator cancels on both sides of the equations. Thus, the flow depth is directly proportional to the difference between the 2%-runup and levee freeboard, and the depth-averaged flow velocity is proportional to the square root of the difference. Wave parameters enter into the estimation of flow depth and velocity at the flood-side crest toe through the estimation of the 2%-runup parameter $R_{u2\%}$. As noted by van Gent (2002), the calculated $R_{u2\%}$ is a fictitious value in cases where runup exceeds the structure freeboard. It is the level that would be exceeded by 2% of the waves if the front slope was continued upwards indefinitely.

The values of the empirical coefficients determined for the two studies are given in Table 6. The superscripts behind each number refer to the references given in the list below Table 6.

Table 6. Empirical Coefficients for Flood-Side Crest Toe Flow Parameters

Coefficient	Schüttrumpf	van Gent
$C_{Ah2\%}$	0.33 [2,3] and 0.22 [4]	0.15 [1,3]
$C_{Au2\%}$	1.37 [2,3]	1.30 [1,3]
$C_{Ah50\%}$	0.17 [2,4]	-
$C_{Au50\%}$	0.94 [2, 4]	-

[1] van Gent (2002)
[2] Schüttrumpf, et al. (2002)
[3] Schüttrumpf and van Gent (2003)
[4] Schüttrumpf and Oumeraci (2005)

The value for $C_{Ah2\%}$ given by Schüttrumpf was revised from 0.33 to 0.22 in the most recent paper (Schüttrumpf and Oumeraci 2005), and this probably represents a better value as shown by the data plot given in their paper, and the fact it is closer to the value obtained by van Gent. Also, in Schüttrumpf, et al. (2002) the value of $C_{Au2\%}$ = 1.37 comes from a table that is identified as "$C_{Au10\%}$ for the large-scale tests." This is thought to be a typographical error, and the label was supposed to be "$C_{Au2\%}$ for the large-scale tests." The small-scale tests gave a value of $C_{Au2\%}$ = 1.55.

Schüttrumpf and van Gent (2003) attribute differences in empirical coefficients to different dike geometries and instruments, but noted the differences are not too great. Van der Meer, et al. (2006) suggested an error in measurement or analysis might have caused the factor of two difference seen for the coefficient $C_{Au2\%}$, but the revised value of 0.22 brings the results closer. Another cause for variation might be in the method each investigator used to estimate the value of 2%-runup, $R_{u2\%}$.

Van Gent (2002) estimated $R_{u2\%}$ using a formula he developed earlier (van Gent 2001) that uses $H_{1/3}$ and $T_{m-0.1}$ as the wave parameters. Schüttrumpf estimated $R_{u2\%}$ using the equations of de Waal and van der Meer (1992) with wave height $H_{1/3}$ and wave period T_m instead of spectral peak period T_p. Both formulas give reasonable estimates that fall within the scatter of the 2%-runup data, so whichever formula is selected for calculating $R_{u2\%}$ the estimates for overtopping flow parameters should be reasonable.

In this study the values of $C_{Ah2\%}$ = 0.22 and $C_{Ah2\%}$ = 1.37 are used to estimate the overtopping flow parameters associated with the flow depth and velocity exceeded by 2% of the incoming waves.

Flow Parameters at the Protected-Side Levee Crest Toe

Overtopping waves flowing across the dike or levee crest decreases in height, and the velocity decreases as a function of the surface friction factor, *f*. The flow depth (or thickness) can be estimated at any location on the crest with the equation

$$h_{B2\%} = h_{A2\%} \, exp(-C_3 \frac{x_c}{B}) \qquad (3)$$

where *B* is the crest width, x_c is distance along the crest from the flood-side toe, and C_3 is an empirical coefficient. The flow thickness at the protected-side crest toe (denoted by the subscript letter B in this report) is given when $x_c = B$. Different values of the coefficient were given in the various publications, i.e., $C_3 = 0.89 – 1.11$ (Schüttrumpf, et al. 2002); $C_3 = 0.40$ and 0.89 (Schüttrumpf and van Gent 2003); and $C_3 = 0.75$ (Schüttrumpf and Oumeraci 2005). For calculations in the present study, a value of $C_3 = 0.75$ was selected on the assumption that earlier values had been corrected. Note that Eqn. (3) is applicable for estimating $h_{B50\%}$ if the flow depth $h_{A50\%}$ is used instead of $h_{A2\%}$. In fact, Schüttrumpf and Oumeraci (2005) presented only the 50% exceedance values.

Flow velocity along the dike crest exceeded by 2% of the waves is given by a similar equation

$$u_{B2\%} = u_{A2\%} \exp\left(-\frac{x_c f}{2 h_{B2\%}}\right) \qquad (4)$$

where f is the friction factor and $h_{B2\%}$ is the flow depth at that location on the crest obtained via Eqn. (3). At the protected-side crest toe, evaluate Eqn. (4) with $x_c = B$. Van Gent (2002) had a different expression for $u_{B2\%}$, but in Schüttrumpf and van Gent (2003) both authors agreed on Eqn. (4). A theoretical derivation for Eqn. (4) is given in Schüttrumpf and Oumeraci (2005).

Friction factor has a significant influence on flow velocity across the crest and down the backside slope. The small-scale experiments of Schüttrumpf, et al. (2002) had a structure surface constructed of wood fiberboard, and the friction factor was determined experimentally to be $f = 0.0058$ (Schüttrumpf and Oumeraci 2005). The structure in the companion large-scale experiments was constructed with a bare, compacted clay surface; and experimental results gave the friction factor as $f = 0.01$ (Schüttrumpf, et al. 2002). Schüttrumpf and Oumeraci (2005) also list the following representative values for friction coefficient: $f = 0.02$ (smooth slopes), $f = 0.1 – 0.6$ (rough revetments and rubble-mound slopes). Grass-covered slopes would have a friction coefficient somewhere between 0.02 and 0.10 (see section below for more detail).

Flow Parameters on the Protected-Side Levee Slope

Both investigators derived theoretical expressions for the wave front depth-averaged, slope-parallel flow velocity down the protected-side slope based on simplification of the momentum equation. Schüttrumpf and Oumeraci (2005) presented an iterative solution, whereas van Gent (2002) derived an explicit formula. A comparison between the two solutions revealed only small differences in the solution, and both formulations approached the same equation in the limit as distance down the slope becomes large (Schüttrumpf and van Gent 2003). For ease of application, van Gent's formula is preferred, and it was given as

$$u_{sb2\%} = \frac{K_2}{K_3} + K_4 \exp\left(-3 K_2 \cdot K_3^2 \cdot s_b\right) \qquad (5)$$

with

$$K_2 = (g \sin \alpha)^{1/3} \qquad (6)$$

$$K_3 = \left[\frac{f}{2} \frac{1}{(h_{B2\%} \cdot u_{B2\%})} \right]^{1/3} \tag{7}$$

$$K_4 = u_{B2\%} - \frac{K_2}{K_3} \tag{8}$$

and α is the angle of the protected-side slope, s_b is the distance down the slope from the crest toe, and $h_{B2\%}$ and $u_{B2\%}$ are the flow depth and flow velocity, respectively, at the protected-side crest toe. For long distances down slope, the exponential term in Eqn. (5) vanishes, and the velocity equation reduces to

$$u_{sb2\%} = \frac{K_2}{K_3} = \left[\frac{2\, g \cdot h_{b2\%} \cdot u_{b2\%} \cdot \sin \alpha}{f} \right]^{1/3} \tag{9}$$

Flow thickness perpendicular to the slope at any point down the protected-side slope is found from the continuity equation as

$$h_{sb2\%} = \left[\frac{h_{b2\%} \cdot u_{b2\%}}{u_{sb2\%}} \right] \tag{10}$$

Equations (1) – (10) give an estimate of the wave overtopping peak velocity and associated flow depth over a levee that is exceeded by only 2% of the incoming waves.

Figure 6 shows the measured time series of waves overtopping a levee in which the surge level exceeded the levee crest. Model-scale values recorded near the protected-side crest toe have been scaled to full-size. The velocity time history of the overtopping waves is characterized by a triangular, sawtooth shape with a steep forward face rising to the peak velocity, followed by a somewhat linear decrease in velocity with the passage of the wave front.

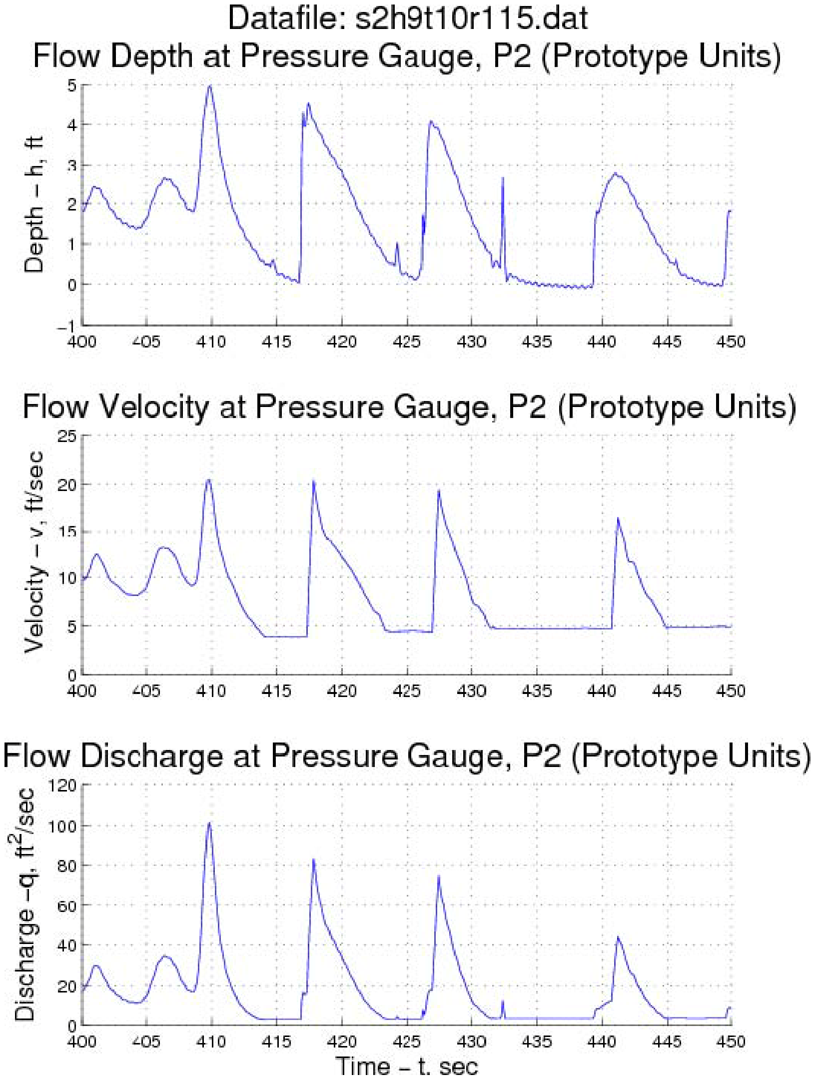

Figure 6. Laboratory measurements of waves overtopping a levee

The equations above solve for the velocity and flow depth peaks, and the levee is only subjected to the peak velocities momentarily with lower velocities for the rest of the wave passage. Thus, duration of maximum flow is fleeting, and little erosion would be expected unless the erosion velocity threshold is quite a bit lower than the peak velocity.

Estimation of an Appropriate Friction Factor

The bottom friction factor is an influential parameter for estimating peak overtopping velocities. An estimate of a friction factor appropriate for grass-covered slopes was not suggested in any of the reviewed papers, so the following ad hoc procedure is offered until better methods become available.

Hewlett, et al. (1987) recommended a value of Manning's *n* = 0.02 for grass-covered slopes steeper than 1:3. Manning's *n* can be related to the Chezy coefficient, C_z, by the expression (e.g., Henderson 1966)

$$C_z = \frac{R^{1/6}}{n} \qquad (11)$$

where *R* is the hydraulic radius, and *n* is given in metric units. For wide channels, *R* is essentially the same as the depth, *h*. Assuming the friction factor given in the overtopping flow literature is the same as the Darcy friction factor, the Chezy coefficient is also given as (Henderson 1966)

$$C_z = \sqrt{\frac{8g}{f}} \qquad (12)$$

Equating (11) and (12), substituting *h* for *R*, and using the value of $n = 0.02$ results in an equation (in metric units) relating *f* to flow depth *h* in meters.

$$f = \frac{8\,g\,n^2}{h^{1/3}} = \frac{8(9.816)(0.02)^2}{h^{1/3}} = \frac{0.0314}{h^{1/3}} \qquad (13)$$

From Eqn. (13) flow thickness over the levee of 0.5 ft (0.15 m), 1 ft (0.3 m), and 2 ft (0.6 m) have friction factors of $f = 0.06$, 0.047, and 0.037, respectively. Therefore, it seems reasonable as an initial assumption to use a value of $f = 0.05$ as a representative average for overtopped grass-covered levee slopes.

Estimation of Freeboard for a Specified Average Wave Overtopping

The next step is to estimate the overtopping flow velocity associated with specific values of average wave overtopping discharge. The necessary inputs to the overtopping flow equations are the 2%-runup for a given wave condition and the levee freeboard that permits the specified average overtopping discharge for the given wave condition.

The average wave overtopping equations of van der Meer and Janssen (1995) give the discharge as a function of

$$q = f(H_{mo}, T_p, \tan\alpha, R_c)$$

Inverting the equations gives the freeboard as a function of

$$R_c = f(q, H_{mo}, T_p, \tan\alpha)$$

Van der Meer and Janssen (1995) gave two overtopping equations with the proper choice depending on the value of the Iribarren number

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{s_o}} = \frac{\tan\alpha}{\sqrt{H_{mo}/L_{op}}} \qquad (14)$$

where L_{op} is the deepwater wave length based on peak spectral period, T_p. Inverting these equations yields

For $\xi op < 2$

$$R_c = -\frac{H_{mo}\,\xi_{op}}{5.2}\, ln\left[\frac{q}{\sqrt{g\,H_{mo}^3}}\cdot\frac{\sqrt{tan\alpha}}{\xi_{op}}\cdot\frac{1}{0.06}\right]\cdot(\gamma_r\gamma_b\gamma_h\gamma_\beta) \qquad (15)$$

For $\xi op > 2$

$$R_c = -\frac{H_{mo}}{2.6}\, ln\left[5\cdot\frac{q}{\sqrt{g\,H_{mo}^3}}\right]\cdot(\gamma_r\gamma_b\gamma_h\gamma_\beta) \qquad (16)$$

The "gamma factors" account for slope roughness, berm effect, shallow depth, and wave direction. See van der Meer and Janssen (1995), or the Coastal Engineering Manual for details.

Figures 7 and 8 show plots of freeboard versus significant wave height for several values of average wave overtopping associated with the criteria discussed earlier in this report. The levee flood-side slope was specified as 1:4, and the peak wave periods were 8 s (Figure 7) and 12 s (Figure 8). The solid curves represent the four criteria for average wave overtopping with the ordinate giving the values of freeboard corresponding to values of wave height on the abscissa. The dashed line is the 2%-runup value for the given wave conditions and levee slope, and in this case the values on the ordinate are runup rather than freeboard. Overtopping flow parameters cannot be estimated for any curve or portion of a curve that lies above the dashed runup line.

It is interesting to note that the runup curves for these two wave periods are nearly equidistant to the curves for discharge of q = 0.1 and 0.25 ft^3/s per ft over a substantial range of wave heights. Therefore, the difference between 2%-runup and freeboard is nearly a constant, and the overtopping flow parameters (which are proportional to $R_{u2\%}$ - R_c) will not vary much for a wide range of wave heights.

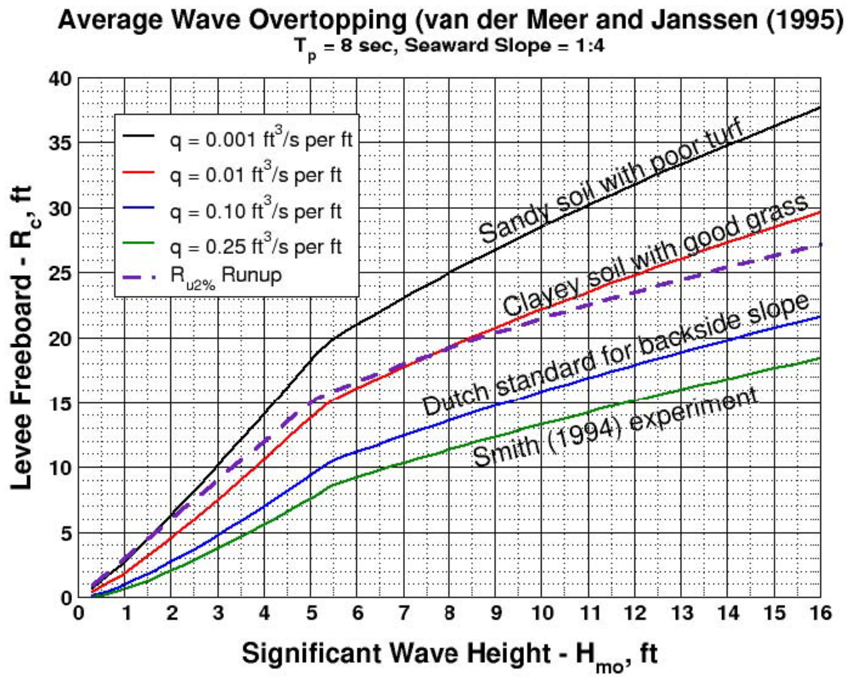

Figure 7. Average wave overtopping for 8-second peak period waves

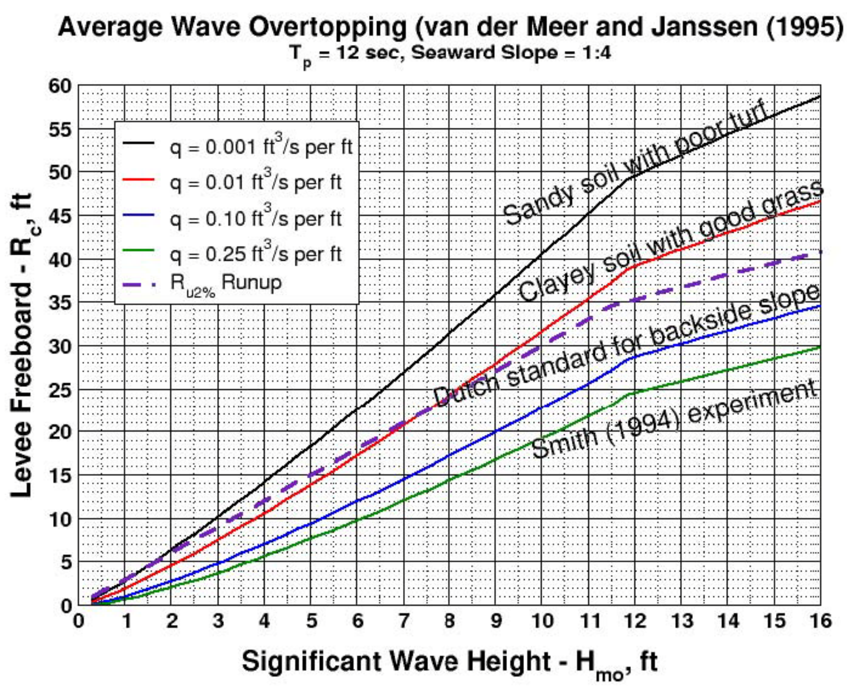

Figure 8. Average wave overtopping for 12-second peak period waves

Estimation of Representative Overtopping Flow Parameters

The formulations given in this section were used to estimate the peak velocity on the protected-side slope (1:3) that is exceeded by 2% of the incoming waves. The initial calculations were for a peak wave period of 8 s, a wave height of 8 ft, a flood-side slope of 1:4, and a crest width of 10 ft. As noted above, these estimates for the 8 ft wave height should be similar for a range of wave heights at this peak period.

Figure 9 shows the slope-parallel, depth-averaged velocity as a function of down-slope distance for three cases. The black line is for a discharge of $q = 0.1$ ft^3/s per ft and a very low friction factor of $f = 0.01$. The initial velocity at the protected-side toe of the 10-ft-wide crest is high because of little bottom friction dissipation over the crest, and the velocity continues to rise toward the terminal velocity with distance down slope. The red line is for the same discharge, but with a more reasonable friction factor of $f = 0.05$. The flow reaches terminal velocity soon after passing the crest toe. The blue curve is the estimate for a higher average wave overtopping discharge of 0.2 ft^3/s per ft.

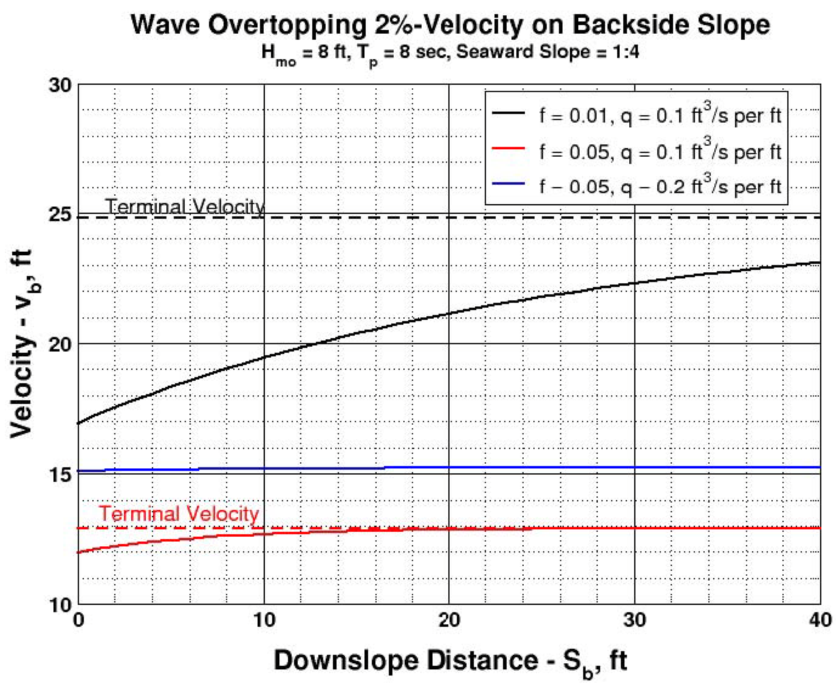

Figure 9. Peak velocity on levee protected-side slope exceeded by 2% of the waves

The calculation of overtopping flow parameters was performed for a range of typical wave heights (H_{mo} = 4, 8, and 12 ft) at two peak wave periods (T_p = 6, 12 sec), and for two average wave overtopping conditions (q = 0.1 and 0.27 ft^3/s per ft), the latter discharge being the same as Smith's (1994) experiments. A friction factor was $f = 0.05$ for all estimates, and the crest width was set at 10 ft. Resulting estimates of required freeboard (R_c); 2%-runup ($R_{u2\%}$); flow depth ($h_{B2\%}$), velocity ($u_{B2\%}$), and discharge ($q_{B2\%}$) at the protected-side crest toe; and terminal flow depth ($h_{S2\%}$) and velocity ($u_{S2\%}$) on the protected side slope are given in Table 7. Accuracy is not as great as implied by the significant digits shown in the Table 7 calculations.

Table 7. Typical Wave Overtopping Flow Parameters Exceeded by 2% of the Waves								
T_p (sec)	H_{mo} (ft)	R_c (ft)	$R_{u2\%}$ (ft)	$h_{B2\%}$ (ft)	$u_{B2\%}$ (ft/s)	$q_{B2\%}$ ($ft^3/s/ft$)	$h_{S2\%}$ (ft)	$u_{S2\%}$ (ft/s)
q_{ave} = 0.1 ft^3/s per ft, *f* = 0.05								
6	4	5.9	10.2	0.44	9.06	3.95	0.37	10.73
	8	9.6	14.4	0.49	10.15	4.93	0.43	11.55
	12	12.7	17.6	0.50	10.48	5.26	0.45	11.80
12	4	6.9	12.0	0.52	10.78	5.57	0.46	12.03
	8	17.1	24.0	0.71	14.37	10.16	0.69	14.69
	12	28.6	35.3	0.68	13.96	9.55	0.66	14.39
q_{ave} = 0.27 ft^3/s per ft, *f* = 0.05								
6	4	4.6	10.2	0.57	11.82	6.72	0.52	12.80
	8	7.8	14.4	0.67	13.78	9.28	0.65	14.26
	12	10.5	17.6	0.73	14.78	10.81	0.72	15.00
12	4	5.4	12.0	0.67	13.76	9.26	0.65	14.25
	8	14.0	24.0	1.02	19.22	19.58	1.07	18.29
	12	24.1	35.3	1.14	20.90	23.87	1.22	19.54

Flow depths ranged between 0.44 ft and 1.22 ft, indicating the selection of *f* = 0.05 was a reasonable choice. The maximum terminal velocity exceeded by 2% of the waves given in Table 7 for discharge of q = 0.1 ft^3/s per ft is 14.69 ft/s (4.48 m/s). This value is right at the maximum permissible velocity for good grass cover exposed to steady overtopping flow of 1-hour duration according to Hewlett, et al. (1987). Considering that the peak velocity in an overtopping wave is a small fraction of each wave period, the levee exposure to flow velocities at the peak will be quite small over the course of a typical storm.

For example, assume a storm with peak period of 12 seconds remains steady at the peak storm surge for 6 hours. This equates to about 1,800 waves during the storm. Two percent of 1,800 waves is 36 waves. In other words, during the 6-hour storm, the 2% velocity on the protected-side slope is exceeded by 36 waves. Van der Meer, et al. (2006) suggested the duration of larger individual wave overtopping events is about 0.5 – 0.8 times T_p, so a rough estimate of the time water is flowing on the rear levee slope for these 36 waves is about six minutes (36 waves x 12 sec/wave x 0.8). The maximum velocity occurs only for a small fraction of the six minutes. The rest of the flow is at lower velocity that varies almost linearly between zero and the maximum velocity. Thus, the overtopping exposure to the highest velocities is limited. Given the fact that maximum velocity estimated for the range of conditions shown in Table 7 for an average wave overtopping of q = 0.1 ft^3/s per ft is near the 1-hour duration limit for steady flow overtopping, it can be concluded that this is a safe criterion.

The maximum velocity exceeded by 2% of the waves associated with an average wave overtopping discharge of q = 0.27 ft^3/s per ft is 19.54 ft/s (5.96 m/s). This velocity exceeds the Hewlett, et al. (1987) criterion for good grass by a significant amount. However, it is still within the bounds given in the earlier steady flow guidance given by Whitehead, et al. (1976). The fact that the grass levee surface is exposed to these higher velocities for a relatively short period of time over several hours may partially explain the grass-slope stability found in Smith's (1994) full-scale overtopping test when subjected to the same overtopping discharge.

Summary

This paper has been an attempt to shed some light on the validity and developmental background of present design guidelines for permissible average wave overtopping for grass-covered earthen levees. The generally accepted criterion for levees with good quality grass cover on the crest and protected-side slope is an average discharge per unit length of levee of q = 0.01 m^3/s per m (q = 0.11 ft^3/s per ft). This criterion first arose from recommendations made by Goda in 1970, and it also appeared in Dutch guidelines in the late 1980s.

Goda's recommendation was based on observed response (damaged and undamaged) of coastal dikes and seawalls following typhoons in Japan. The analytical method for estimating the average wave overtopping was shown to be reasonably accurate, but it was intended for vertical walls fronted by a rubble-mound absorber. Structure freeboard was estimated from post-storm surveys of surge level in the protected lee of buildings, and these estimates should be considered good. Waves used to calculate average wave overtopping were hindcast based on estimates of typhoon winds. Goda recognized that the wave estimates introduced a degree of uncertainty, and he was deliberately cautious in applying the hindcast results.

Three factors suggest that the overtopping criterion published by Goda might be slightly conservative. First, estimates for wave overtopping were made using a method developed for overtopping of vertical walls with rubble absorber. For impermeable coastal dikes with a sloping seaward slope, actual overtopping rates would be expected to be a little higher than estimated. Second, if Goda was unsure about the wave estimates, he would have chosen values that gave a conservative estimate of the overtopping. Third, the fact that the overtopping criterion q = 0.01 m^3/s per m (q = 0.11 ft^3/s per ft) has proven successful for over 30 years in Japan indicates the criterion is either ideal or slightly conservative.

The Dutch permissible average wave overtopping criteria for different soil/grass condition was reportedly based on design curves for permissible velocity versus duration for steady flow overtopping. However, it is not immediately apparent how the correspondence was established between unsteady wave overtopping flow and steady overtopping velocity. Van der Meer, et al. (2006) confirmed the Dutch criteria stem for Hewlett, et al.'s (1987) steady flow curves, but they stated the criteria were never validated. Recent full-scale experiments by Smith (1994) proved that protected-side dike slopes covered with healthy grass could withstand wave overtopping over two times the present guideline of q = 0.01 m^3/s per m (q = 0.11 ft^3/s per ft). This important data point suggests the present criterion is slightly conservative; but keep in mind test conditions were ideal, and the grass cover performance would not be as good for dormant winter grass or otherwise deteriorated grass covers.

Recent methodology was estimating overtopping flow parameters on dikes and levees was reviewed for the purpose of developing a link between unsteady wave overtopping and steady flow overtopping. Two independent studies of overtopping flow parameters arrived as similar methods, and a joint paper resolved some of the differences. This methodology was applied in this paper for a range of overtopping wave conditions that produced average wave overtopping discharges of q = 0.1 and 0.27 ft^3/s per ft (0.010 and 0.025 m^3/s per m). The maximum terminal velocity on the protected-side slope exceeded by 2% of the incoming waves was found to be right at the permissible steady flow velocity for 1-hr duration. Because this wave overtopping maximum flow velocity occurs for only a brief portion of the overtopping episode, it was reasoned that the q = 0.1ft^3/s per ft (0.010 m^3/s per m) criterion was safe. Maximum wave overtopping flow velocity for the higher average wave overtopping discharge used in Smith's (1994) experiments exceeded the permissible steady flow velocity at 1-hr duration; but once again, this exceedance has short duration with the bulk of the overtopping flow having velocities below the steady flow criterion.

Based on the analysis given in this report, it is concluded that the criterion presented in the literature for permissible wave overtopping of an earthen levee with a healthy grass cover is competent, if not slightly conservative. The criteria for poorer quality soils and grass coverings are probably safe, but less evidence exists to support a definitive conclusion.

Knowledge Gaps and Recommended Actions

The most apparent need is for more full-scale field and laboratory evidence to support the permissible wave overtopping criteria for a range of levee soil types and grass coverings. Van der Meer, et al. (2006) described full-scale tests of protected-side dike slopes that are scheduled to commence in 2007. They have constructed an overtopping simulator that can be installed on the crest of existing levees. Discharge from the simulator is controlled to reproduce typical time series of unsteady discharge experienced during wave overtopping. These extremely important tests will usher in new understanding about how grass covers fail along with the corresponding level of wave overtopping.

In the wake of Hurricane Katrina an unparalleled opportunity exists to augment full-scale experimental findings with detailed field observations similar to those Goda conducted many years ago. Some sections of the south Louisiana levee system experienced various degrees of damage ranging from minor to catastrophic while other reaches survived intact. Extensive wave and surge hindcasts at an unprecedented level of detail and sophistication have provided the necessary hydrodynamic input to estimate with reasonable certainty the hydrograph of average wave overtopping at nearly every location that experienced waves. Coupling observed levee damage to the causative hydrodynamic conditions would provide tremendous new information about damage due to wave and surge overtopping. A key aspect of this undertaking is documenting the levee soil type and condition for each of the studied reaches. Soil information is needed to unite both the hydrodynamic and geotechnical criteria into a single recommended standard for future design. One difficulty with quantifying wave overtopping damage might be establishing pre-storm levee crest elevations, but work on this aspect of the problem is also being addressed.

More analytical and laboratory work is needed to refine the estimation procedures for comparing steady wave overtopping results with unsteady wave overtopping. Two aspects in particular need attention. First, a better understanding is needed for specifying an appropriate value for the friction factor for various slope surfaces. Second, a robust representation of the time-varying flow down the slope is required to make accurate estimates of shear stress. A validated procedure for estimating shear stresses acting on the protected-side levee slope experiencing unsteady flow overtopping is applicable to a wide range of slope protection solutions including grass, turf reinforcement, soil strengthening, and armoring systems.

Finally, the average wave overtopping criteria discussed in this paper apply only to earthen levees where the overtopping wave flows over the levee crest and down the protected-side slope. The criteria are not intended for the case where waves overtop a vertical floodwall situated on the levee crest, and water plunges as a jet to the levee surface before continuing to flow down the protected-side slope. It may be that flow velocities on the protected-side slope in this case are similar to those experienced by overtopping of a levee without a floodwall, but no studies have been conducted to examine this hypothesis.

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Appendix G

Surge and Wave Modeling For River and Sea Level Rise Sensitivity (Version 2 July 2010, Authors: T. Wamsley, M. Cialone, D. Resio, ERDC)

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APPENDIX G – SURGE AND WAVE MODELLING FOR RIVER AND SEA LEVEL RISE SENSITIVITY (VERSION 2 JULY 2010)

Authors: Ty Wamsley, Mary Cialone, Don Resio, Engineering Research and Development Center, Coastal Hydraulics Laboratory

A significant issue in the design of flood protection in Southern Louisiana is the consideration of relative sea level rise (SLR) due to climate change and local subsidence. Relative sea level and its change over time alter the extent and degree that surge generated by hurricanes impacts coastal areas (Smith et al. 2010). The Mississippi River levee design must also take into account the effect of the river flow rate at the time of a particular storm on surge levels in the river. The combined effect of sea level rise and river flow rates on surge levels in the Mississippi River were examined in this study. This report documents the surge and wave numerical modeling and statistical analysis conducted for the New Orleans District (MVN) by the Coastal and Hydraulics Laboratory (CHL) staff. The purpose of this effort was to examine surge and wave sensitivity to variations in 1) the Mississippi River flow rate and 2) mean water level. In this way, the potential impact of relative SLR and river flow rate on hurricane surge in the lower Mississippi River can be assessed by examining the range of surge response to these two varying conditions. This was accomplished through numerical surge modeling of hypothetical hurricanes for a base condition and with the inclusion of sea level rise and various river flow rates. A storm simulation suite consisting of 17 storms was developed and simulations were made for six flow rate-sea level rise cases. A total of 102 storm simulations were made for this project.

Grid Development and Identification of Save Location

The ADCIRC and STWAVE grids were updated in the project area to represent project features that were provided by MVN, including geo-referenced project alignments and structure heights relative to NAVD88 (2004.65). The majority of grid development work was covered under the New Orleans to Venice (also referred to a Oakville to LaReusitte) project. The grid was updated based on work performed for New Orleans to Venice (NOV). The base grid for that study was the IHNC grid (as modified for NOV). Note that this is a modified version of the SL15v7 validated for application in the Mississippi River for different discharges. The grid was updated to include the following project alignment features:

- West Bank and Vicinity – all elevations set to non-overtopping; eastern tie-in in the Hero to Oakville area were updated
- Western Closure Complex – were included and set to non-overtopping
- Caernarvon Alignment – alignment were updated to include the C-SBLP alignments and set to non-overtopping
- Braithwaite Levee – were checked to ensure the grid has the correct existing heights
- Mississippi River – were checked and set to MR&T grade if existing elevation is lower
- Plaquemines Non-Fed Levees – updated eastern tie-in in the Hero to Oakville area; set height based on NOV simulations
- Plaquemines Fed Levees – set height based on NOV simulations

MVN provided geo-referenced project alignment to incorporate the project into the ADCIRC grid. A map summarizing the levee heights is provided in Figure 0. Once the project features were incorporated and levee heights updated, the updated grid was provided to MVN for final review by District personnel prior to the updating of the STWAVE grid and the execution of any model simulations. The STWAVE grid was modified to replicate the updates made to the ADCIRC grid applying codes developed by ERDC. In addition, the friction fields for the sea level rise runs were updated to reflect changes in vegetation type with higher water levels. Save locations were selected in consultation with MVN (Figure 1).

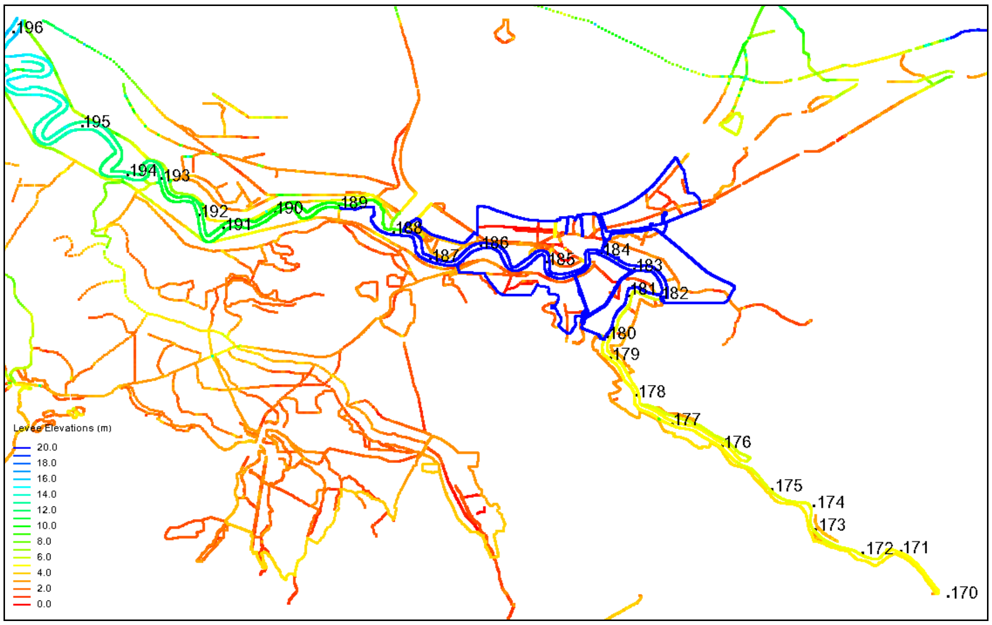

Figure 0. Map summarizing levee heights.

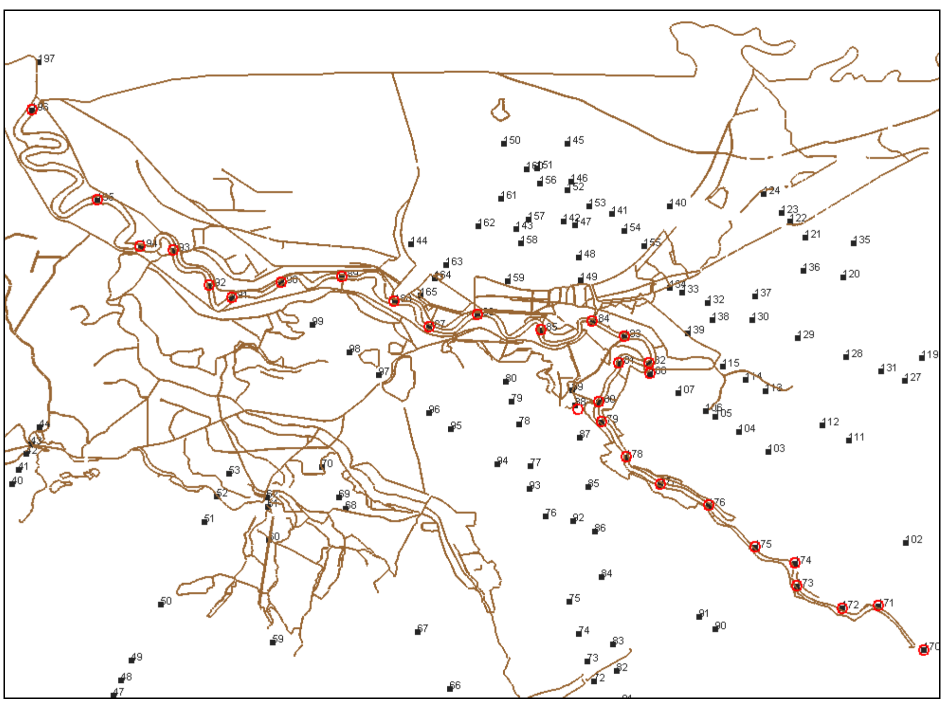
Fig 1. Save locations in the Mississippi River are circled in red

Surge and Wave Model Simulation Methodology

The potential impact of SLR and flow rate on surge due to hurricanes was evaluated using ADCIRC and STWAVE models for southeast Louisiana. A base case was run using post-Katrina bathymetry and levee heights that were expected to be in place in 2010. Then 0.3, 0.6, and 0.9 m of additional water were added to represent future SLR cases and the runs were repeated. A river flow rate of 11330 m^3/sec (400,000 cfs) representing high flow conditions was simulated for the base and 3 SLR conditions and a river flow rate of 4730 m^3/sec (167,000 cfs) representing low flow conditions was simulated for the base and 0.6-m SLR condition. For this evaluation, 17 hypothetical hurricanes were simulated. These hurricanes generated approximately 100-yr water levels in areas in southeast Louisiana (Resio 2007). For these simulations, the bathymetry and topography were not modified to represent coastal erosion or wetland loss, but bottom roughness values and frictional wind resistance were updated to reflect vegetation changes consistent with the increased water levels. The 17 hypothetical storms selected for simulation have central pressure (C_p) of 900 or 930 mb, radius of maximum winds (R_{max}) of 23-48 km, and a forward speed (V_f) of 5.6 m/s. Additional information on the modeling methodology is given by Bunya et al. (2010) and Dietrich et al. (2010).

The 17-storm simulation suite was selected from the original JPM-OS suite of storms (Table 1). By selecting storms from the original suite, archived WAM results from these previously simulated runs could be used and rerunning WAM was not needed.

Table 1. JPM-OS Storms Suite
014
015
017
018
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057
069
073
077

The 17 storms were simulated for the following six cases:

1. River flow at 11330 m^3/sec (400K cfs), existing condition
2. River flow at 4730 m^3/sec (167K cfs), existing condition
3. River flow at 11330 m^3/sec (400K cfs), sea level rise of 0.3 m
4. River flow at 11330 m^3/sec (400K cfs), sea level rise of 0.6 m
5. River flow at 4730 m^3/sec (167K cfs), sea level rise of 0.6 m
6. River flow at 11330 m^3/sec (400K cfs), sea level rise of 0.9 m

Each case required an ADCIRC river spin up simulation. A one-day river spinup was sufficient for the lower flow rate to reach a dynamic steady state and a two-day river spinup was sufficient for the higher flow rate to reach a dynamic steady state, however a two-day spinup was applied to all simulations for consistency. ADCIRC and STWAVE were run in a coupled fashion consistent with previous work and water levels, wave heights and periods were computed. The results were QA/QC'd with procedures similar to that done for LaCPR. Peak plots for both waves and water levels in the project area were produced and analyzed for quality assurance. Sample maximum surge envelopes for Case 1-Storm 014 and Case2-Storm 014 are shown in Figures 2 and 3, respectively.

Analysis of River Peak Surge Levels: Storm 014

As part of the quality check of model results, an analysis of maximum surge levels in the Mississippi River was done for all cases and all storms. By way of example, maximum surge levels in the Mississippi River from the Mississippi River Delta to Baton Rouge for hypothetical Storm 014 are shown in Figure 4 for the six SLR and flow rates combinations that were simulated. By comparing Case 1 (SLR=0.0; Flow Rate=11.33 Km3/s) to Cases 3, 4, and 6 (SLR of 0.30, 0.6, and 0.9 m and Flow Rate=11.33 Km3/sec), it is evident that the surge increase is not simply linearly proportional to the change in water level due to sea level rise. At the Mississippi River Delta, the increase in water level actually is fairly linear, but at River Point 176 (where the two-jetty system begins), the surge response to sea level rise becomes non-linear, and at River Point 180, near Oakville and south of Belle Chasse, the surge response to sea level rise becomes highly non-linear. The nonlinearity is primarily a function of increased surges propagating across the Caernarvon marsh area, overtopping the levees. For storm 14, the surges for no sea level rise are at or near the river levee crests in the Caernarvon area. With sea level rise and the associated loss in the Caernarvon wetlands, the surges at this location increase and result in water overtopping the levees, introducing water into the river. Simulations with the lower flow rate have lower peak water levels at Baton Rouge than the higher flow rate simulations, but water levels "down river" near the delta are similar.

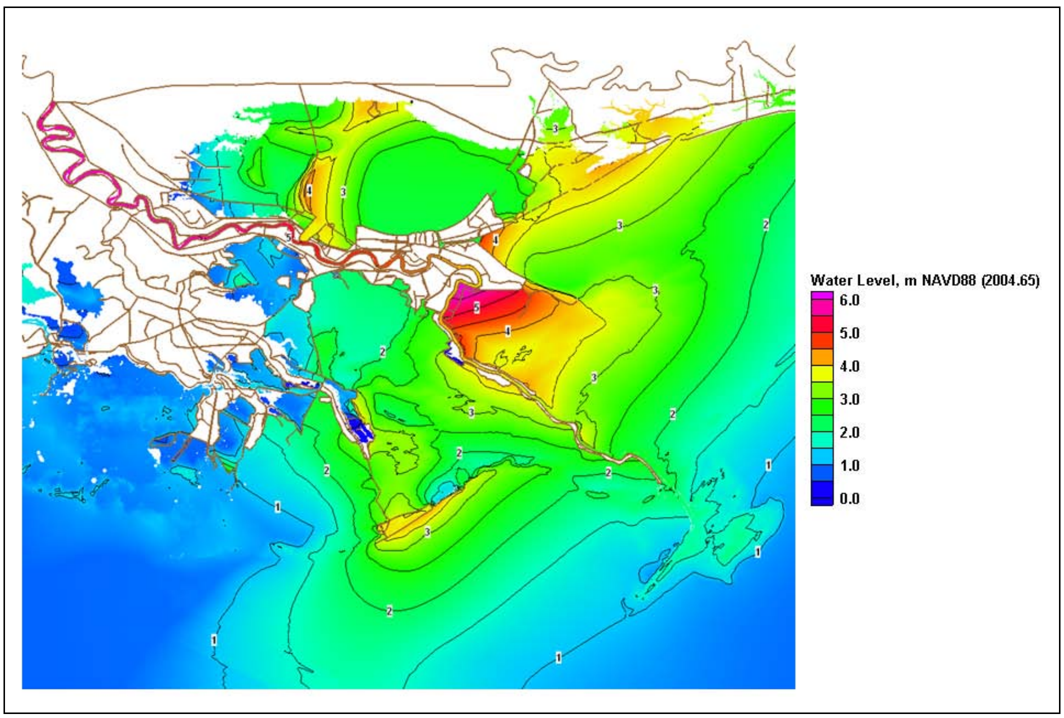

Fig 2. Maximum surge envelope for Storm 014 – Case 1

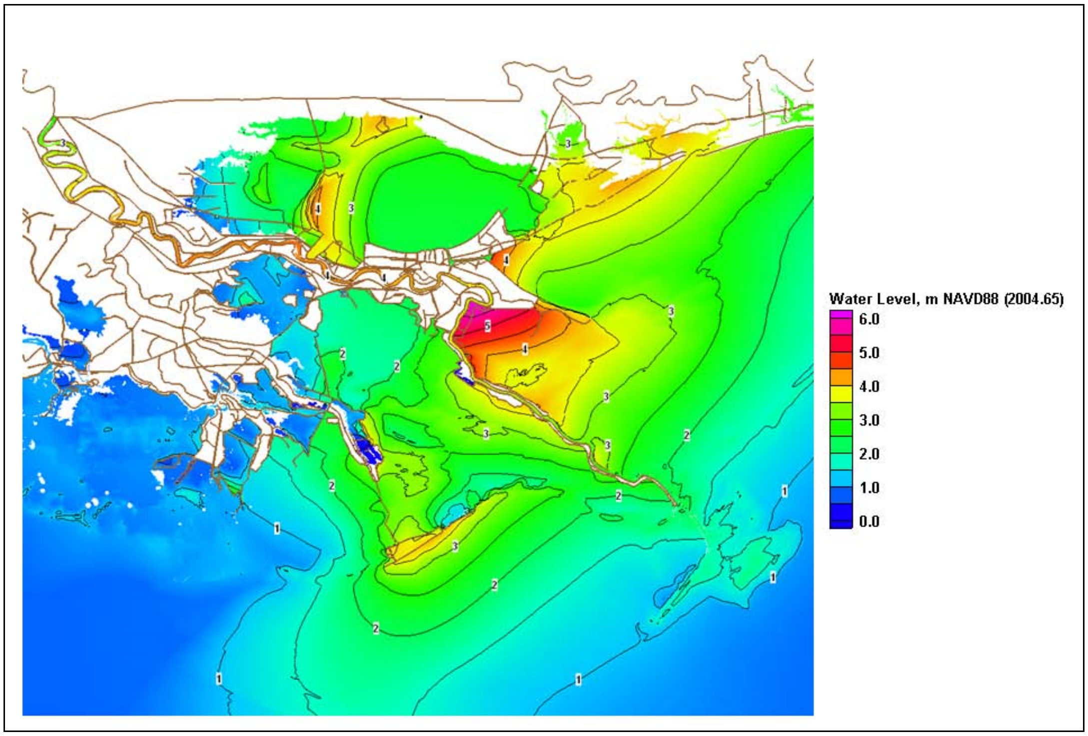

Fig. 3. Maximum surge envelope for Storm 014 – Case 2

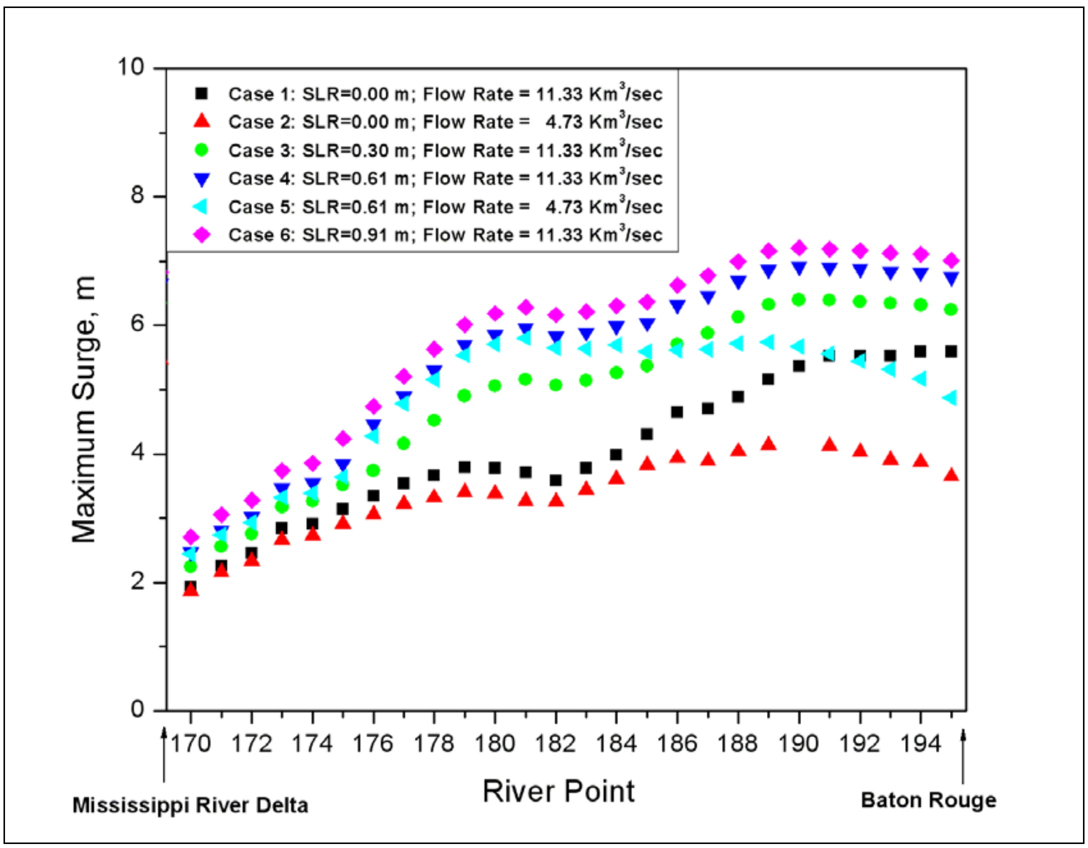

Fig. 4. Maximum surge for Storm 014 for varying sea level rise and river flow rates

Analysis of River Peak Wave Heights and Periods: Storm 014

The primary purpose of the modeling was to evaluate the combined effect of sea level rise and river flow rates on surge levels in the Mississippi River. However, the full-plane version of STWAVE was applied to provide wave height and period estimates within the river. The STWAVE grid used is consistent with grids applied in previous studies and was not optimized for wave estimation in the river.

Sample maximum wave height envelopes for the South and Southeast STWAVE domains for Case 1-Storm 014 and Case2-Storm 014 are shown in Figures 5 through 8.
As part of the quality check of model results, an analysis of maximum wave height and wave period in the Mississippi River was done for all cases and all storms. By way of example, an analysis of maximum wave heights and wave periods in the Mississippi River from the Mississippi River Delta to Baton Rouge for hypothetical Storm are described here.

The maximum wave heights in the Mississippi River vary between 0.7 m and 2.6 m for this storm. However, the vast majority of river points have maximum wave height values between 0.7 m and 1.5 m. Case 6 generally has the largest max waves, though the variability among cases is not significant, in general. The peak period in the Mississippi River varies between 2.8 sec and 6.0 sec for this storm. There is less variability between the various cases for river points closer to Baton Rouge. For River Points 182-192, the periods are nearly identical for all cases. The periods are also shorter in this part of the river, ranging from 2.8 sec to 3.7 sec. For the river points closer to New Orleans, Case 6 generally has the longest periods.

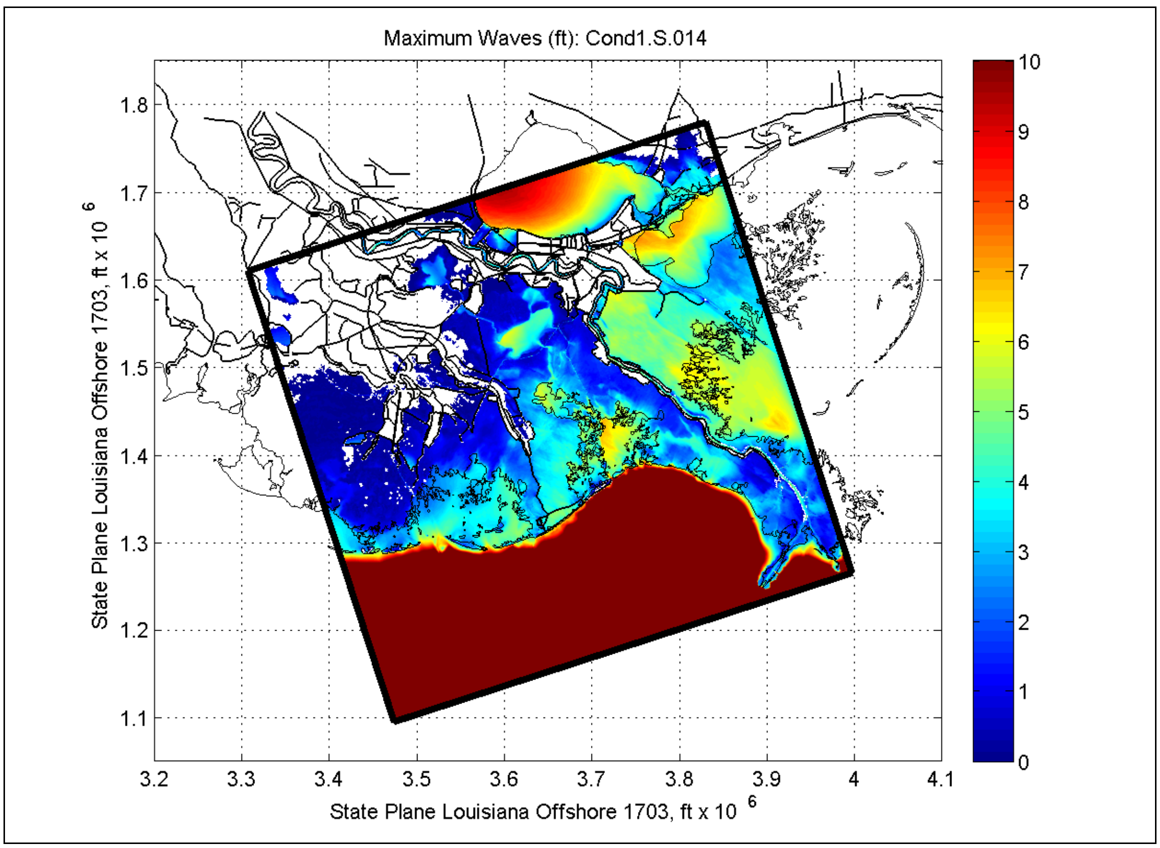

Fig 5. Maximum wave height envelope: S grid Case 1 Storm 014

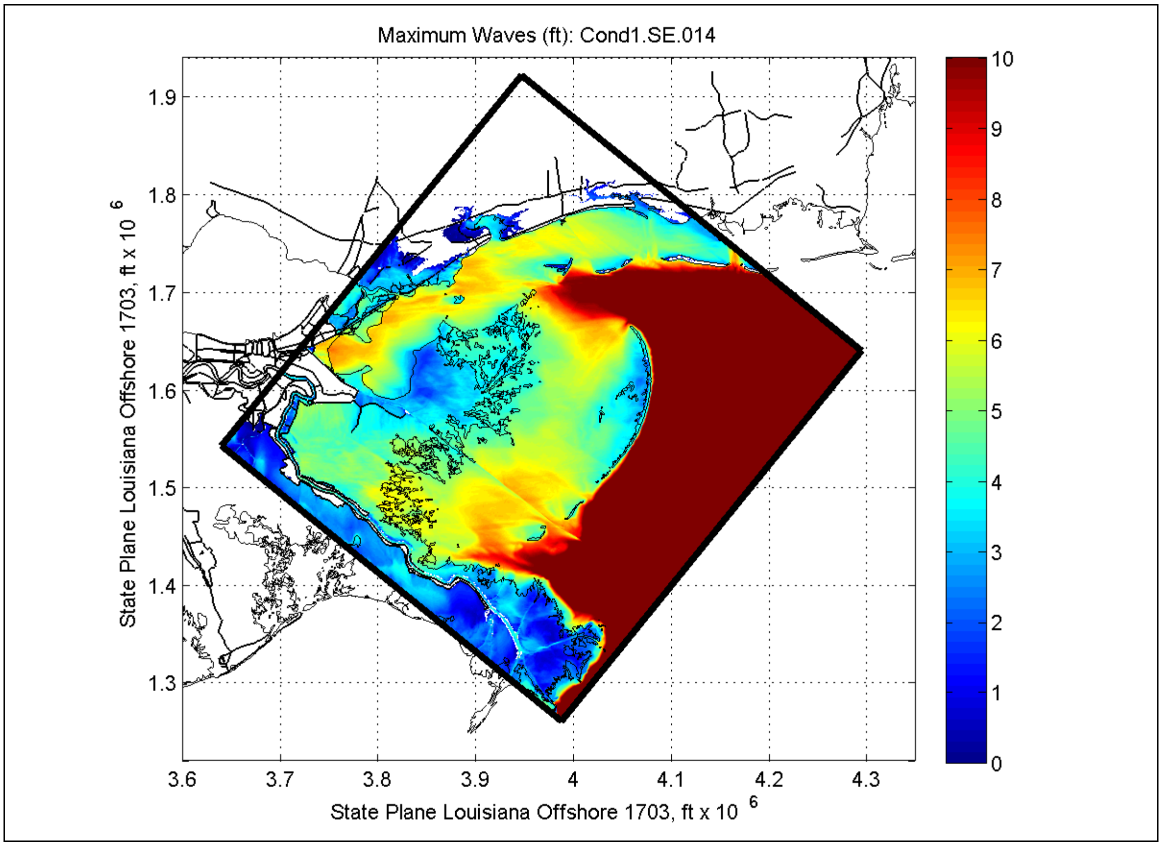

Fig 6. Maximum wave height envelope: SE grid Case 1 Storm 014

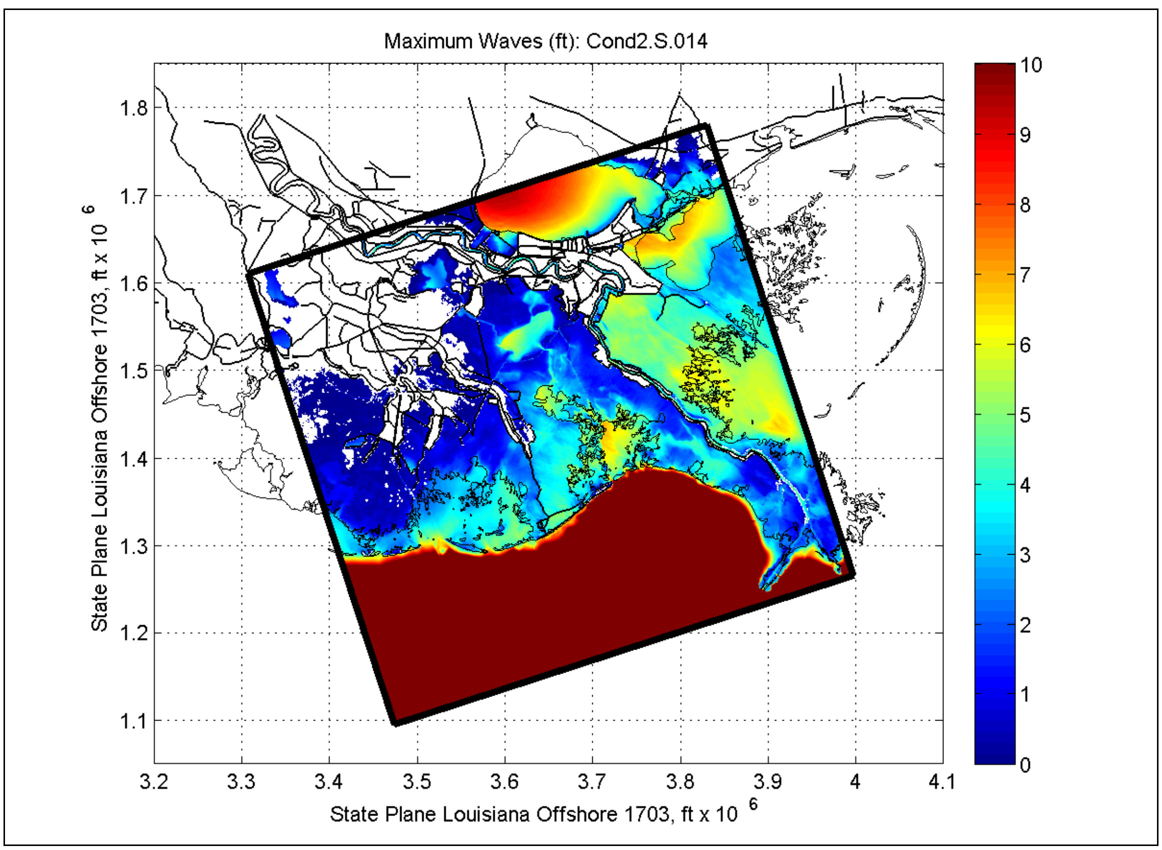

Fig 7. Maximum wave height envelope: S grid Case 2 Storm 014

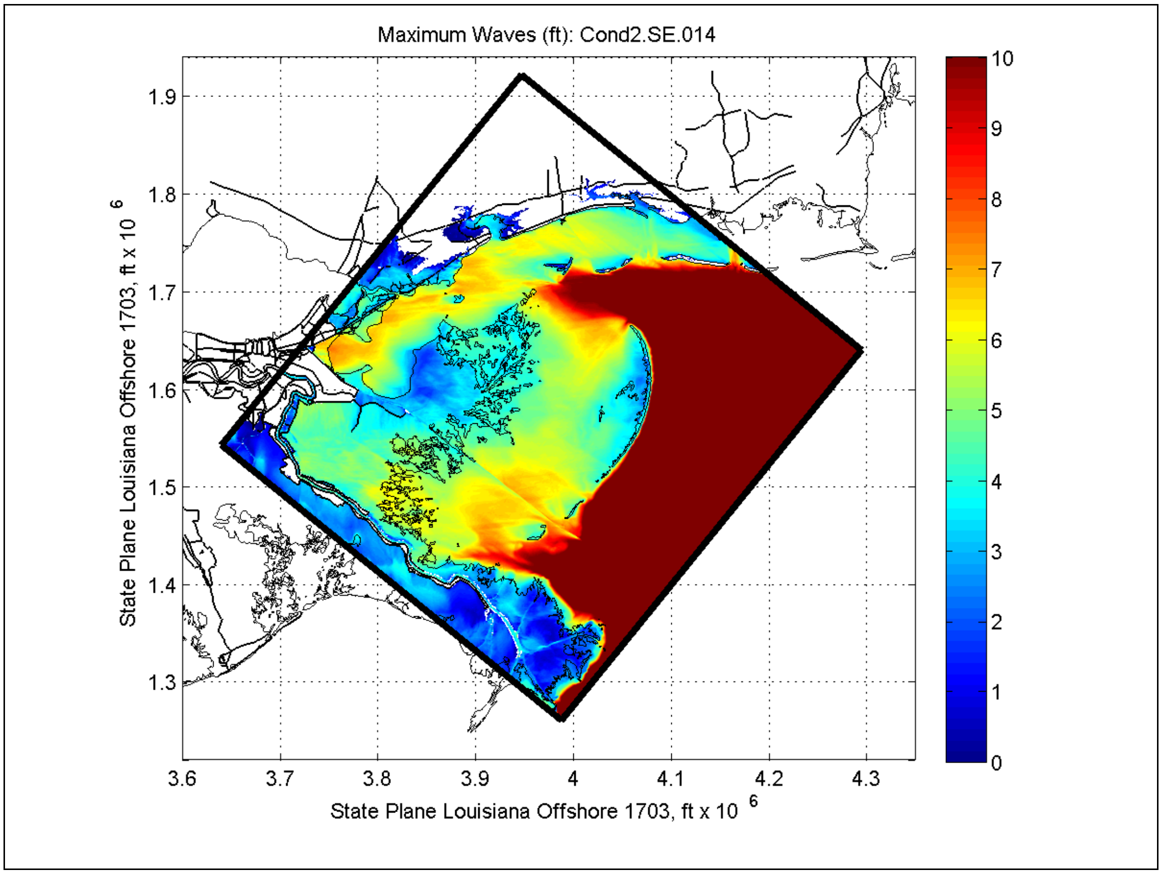

Fig 8. Maximum wave height envelope: SE grid Case 2 Storm 014

Statistical Analysis

The JPM-OS statistical analysis was performed based on the results for the model simulations. Results (water level, wave height, and wave period) were developed into csv files at desired save locations provided by the District. Results were provided for each case for the 100-yr frequency results.

The JPM-OS code was modified to work with the storm suite identified for this work and to estimate the effects of sea level and river flow rate variations as discussed below. The 17-storm suite was simulated at the currently existing sea level and at the current sea level plus 0.6 m (2 ft) for river discharges of 4,730 m^3/sec (167,000 cfs) and 11,330 m^3/sec (400,000 cfs). The two discharge values provide a means to estimate the surge level for any specified river discharge via interpolation/extrapolation.

<u>1. Estimation of the Effects of Sea Level Variations on Surge Levels within the Mississippi River</u>

Equation 1 is given as:

$$p(\eta) = \frac{1}{\Delta\eta}\sum\sum p(\eta \mid Q)\, p_m(Q)\, p(m)\delta\eta$$

where (1)

$$\Delta\eta = \sum \delta\eta$$

Estimated values for surges at selected return periods are obtained directly from equation 1 via the relationship below (Equation 2)

$$T(\eta) = \frac{1}{1 - F(\eta)}$$

where, $T(\eta)$ is the return period and $F(\eta)$ is the CDF, and (2)

$$F(\eta) = \sum_{i=1}^{\infty} p(\eta_i)\{1 - H[\eta - \eta_i]\}\delta\eta_i,$$ where H is the Heaviside function

From the available data, the CDF is defined at two representative values (+0 m or ft and + 0.6 m (+2 ft)). Values for additional variations in sea level can be obtained by interpolation in the range 0 – 0.6 m (2 ft) and extrapolation for the range beyond that.

<u>2. Estimation of Expected Water Levels in the Vicinity of the Mississippi River Including Effects of River Discharges: Including Smoothing to Minimize the Effects of Discrete Samples</u>

The overall probability of surge levels, including the effects of river discharge, can be written as shown below in Equation 3:

$$p(\eta) = \int p(\eta \mid Q)\, p(Q)\, dQ$$

where (3)

η is the surge level

Q is the river discharge.

The probability of river discharge associated with hurricane surges can be written as shown below in Equation 4

$$p(Q) = \sum p_m(Q)\, p(m)$$

where (4)

m designates the month and

$p(m)$ is take as the percentage of hurricanes which occur in that month.

Combining equations 3 and 4 yields Equation 5:

$$p(\eta) = \sum p(\eta \mid Q)\, p_m(Q)\, p(m) \tag{5}$$

Given a set of surge levels for fixed probability levels and three different discharges, a matrix of interpolated values of expected surge levels is formed as a function of both river discharge and probability level. This can be used to estimate $p(\eta \mid Q)$.

For the recent application, estimates of surge levels at the three discharges and 40 specified return periods at 50-year intervals were converted to probability estimates at each of the three discharges and then interpolated linearly between discharge levels before using the final descretized integration form shown in Equation 6 below:

$$p(\eta_i) = \bar{\delta}(\eta_i - \eta_{jkl})\, p_{jkl}$$

where

η_i is an incremental element of the surge level with size $\Delta\eta$

$\bar{\delta}$ is an incremental Dirac Operator: =1, if $|\eta_i - \eta_{jkl}| < \Delta\eta$; = 0, otherwise; and (6)

p_{jkl} is the probability of the combination of month, surge intensity, and discharge.

This probability was integrated to obtain the estimated Cumulative Distribution Function in the normal fashion, which is then used to estimate the return periods for surge levels including the effects of variable river discharges and their probabilities. Error band RMS values were then estimated from the resulting distribution based on the same approach as was used for the constant river discharge case.

Equation 5 is written as a sum of a discrete set of values rather than a continuous integral since the sample data on surges coming into the computer code represents estimates of surge probabilities at 50-year increments. In most cases, the results from such procedure produces relatively smooth spatial patterns along the Mississippi River. However, in a small number of situations the effect of the discrete set of values is to introduce spurious, small perturbations in the final surge estimates. These unwarranted perturbations can be minimized by adding a small additional sum to distribute each discrete value over a small (±0.15 m (.5 ft)) range. Equation 7 is given as:

$$p(\eta) = \frac{1}{\Delta\eta} \sum \sum p(\eta \mid Q)\, p_m(Q)\, p(m)\, \delta\eta$$

where (7)

$$\Delta\eta = \sum \delta\eta$$

Summary

The combined effect of climate change and river flow rates on maximum surge levels and wave heights in the Mississippi River was examined by analyzing the range of surge and wave response to these two varying conditions. Numerical simulations of 17 hypothetical hurricanes were made for a base condition and with the inclusion of sea level rise and various river flow rates. Surge response to sea level rise is not a linear process within the dual levee Mississippi River system. Simulations with the lower flow rate have lower peak water levels at Baton Rouge, but water levels closer to the Mississippi River Delta are similar. The maximum wave heights in the Mississippi River vary between 0.7 m and 2.6 m for this storm, but the majority are between 0.7 m and 1.5 m. In general wave heights are largest for the highest sea level rise case, though the variability among cases is not significant. The peak period in the Mississippi River does vary however there is less variability between the cases for river points closer to Baton Rouge. In this region, the periods are shorter (2.8-3.7 sec) and nearly identical for all cases. For river points closer to New Orleans, the highest sea level rise case generally has the longest periods. A statistical analysis of water levels in the Mississippi River was also presented.

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Appendix H

Joint Probability Method with Optimal Sampling (JPM-OS) Including River Discharge Variation (Author: D. Resio, ERDC)

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APPENDIX H – JOINT PROBABILITY METHOD WITH OPTIMAL SAMPLING (JPM-OS) INCLUDING RIVER DISCHARGE VARIATION

Author: Don Resio, Engineering Research and Development Center

This appendix contains a description of the modifications made to the original Joint Probability Method with Optimal Sampling (JPM-OS) as described in IPET (200?) to include the river discharge variation (section G.1). This appendix also elaborates on the justification of two assumptions made in this approach (section G.2 and G.3).

Appendix H – 1 Modifications to the JPMS-OS method with Optimal Sampling (Fall 2008)

The overall probability of surge levels, including the effects of river discharge, can be written as

1. $$p(\eta) = \int p(\eta \mid Q)\, p(Q)\, dQ$$

where
η is the surge level
Q is the river discharge.

The probability of river discharge associated with hurricane surges can be written as

$$p(Q) = \sum p_m(Q) p(m)$$

2. where
m designates the month and
$p(m)$ is take as the percentage of hurricanes which occur in that month.

Combining equations 1 and 2 yields

3. $$p(\eta) = \sum p(\eta \mid Q)\, p_m(Q)\, p(m)$$

Given a set of surge levels for fixed probability levels and three different discharges, we can form a matrix of interpolated values of expected surge levels as a function of both river discharge and probability level. This can be used to estimate $p(\eta \mid Q)$.

For the recent application, we converted estimates of surge levels at the three discharges and 40 specified return periods at 50-year intervals to probability estimates at each of the three discharges and then interpolated linearly between discharge levels before using the final descretized integration form

$$p(\eta_i) = \bar{\delta}(\eta_i - \eta_{jkl})\, p_{jkl}$$

where

4. η_i is an incremental element of the surge level with size $\Delta\eta$

$\bar{\delta}$ is an incremental Dirac Operator: =1, if $|\eta_i - \eta_{jkl}| < \Delta\eta$; = 0, otherwise; and

p_{jkl} is the probability of the combination of month, surge intensity, and discharge.

This probability was integrated to obtain the estimated Cumulative Distribution Function in the normal fashion, which is then used to estimate the return periods for surge levels including the effects of variable river discharges and their probabilities. Error band RMS values were then estimated from the resulting distribution based on the same approach as was used for the constant river discharge case.

Appendix H – 2 Investigations into the Relationship between River Discharge and Hurricane Surge Potential within the Mississippi River

Version July 2010

Earlier investigations of the impact of river discharge on surge levels along interior sections of the Mississippi River Levees have show that river discharge has a dramatic effect on such surge levels and should be considered probabilistically in estimating annual exceedance probabilities in these areas. The initial assumption utilized in these investigations was that river discharge rates and the likelihood of significant hurricane surges were independent of each other. The purpose of this study is to examine the possibility that significant correlations between hurricane activity and river discharge in this area.

Assessment of Hurricane Activity

Past studies of variability in the hurricane climate have typically used storm frequency (sometimes stratified by Saffir-Simpson scale; Simpson 1974) to categorize storm activity. However, here we are motivated to seek a single parameter that incorporates storm intensity, size and frequency which characterizes hurricane activity. Irish *et al.* (2008) have shown that coastal surge levels for a capped coefficient of drag (consistent with Powell *et al.*, 2003) scale with the square of the wind velocity and depend approximately linearly on storm size. Thus, a logical surrogate for hurricane surge potential should include both the square of the wind speed and a linear size parameter, i.e.

$$\Lambda = V_{\max}^2 R_p$$

where

2. Λ is the hurricane scale

$V_{\max}$ is the maximum wind speed within the hurricane

R_p is the size scaling parameter for the hurricane pressure field

The value of the parameter Λ at the time of maximum wind speed during a storm's passage through the Gulf of Mexico provides an objective integrated measure of storm intensity and size. Summing all values of Λ for a season yields an index for combined frequency, size, and intensity of storms in that year. Thus, for our purpose here, we define here an integrated measure of annual storm intensity, size, and frequency defined here as the H-Fucntion given by

$$H_n = \sum_{j=1}^{k} \Lambda_j$$

where

3. H_n is the estimated value of the annual hurricane parameter,
Λ_j is the measure of storm intensity and size (eq. 2) for a single storm, and
k is the number of storms in a given hurricane season.

Unfortunately, no measure of storm size – such as radius of maximum winds – is presently included as part of the HURDAT data set. Since this is an important parameter for both surge generation and wave generation potential of a storm, we will supplement the information from HURDAT with some additional analyses performed by Oceanweather, Inc. (OWI). The OWI information is widely used to drive ocean response models for estimating expected inundation levels along the US Gulf coast and has been shown to produce very high-quality results when used with existing surge models and wave models (IPET, 2007; Resio and Westerink, 2008).

River Discharge

Data on river discharge was provided by MVN adjusted flow values at Tarbert Landing) for the period January 1950 through December 2009. Average discharge values were defined for July through October for each year and this average value was taken as an indicator of the overall discharge during each hurricane season.

Data Smoothing

Since we are looking for a climatological basis for some relationship between river discharge and hurricane activity, it is necessary to avoid spurious results due to the small sample size in hurricanes. For this reason, some smoothing is necessary to separate actual multi-year patterns from background noise. To accomplish this, both the H-Function and the river discharges were smoothed via a running average over 5-year periods.

To facilitate visualization of variations in the discharge data and the H-Function in subsequent comparisons, both functions were normalized via the scaling

$$x_i' = \frac{(x_i - x_{\min})}{(x_{\max} - x_{\min})}$$

where x_i' is the ith value of the re-scaled variate, x is the ith value of the original variate, $x_{\max}$ is the maximum value of x for all i, and $x_{\min}$ is the minimum value of x for all i. This normalization provides a scale that goes from -1 to +1 for all variables and allows for improved direct visual comparisons.

Figure 1 shows the results of such a display for smoothed river discharges and H-Function values. As can be seen here, there are two epochs of high hurricane activity, basically the decades of the 1960's and the 2000's. River discharge has a minimum value during the first interval of high hurricane activity; however, river discharge is above the 0.5 level during the second interval of high hurricane activity. As might be expected from a pattern seen in Figure 1, the linear correlation coefficient is quite low, -0.049 with a Student T value of only 0.367, which is not significant at almost any level.

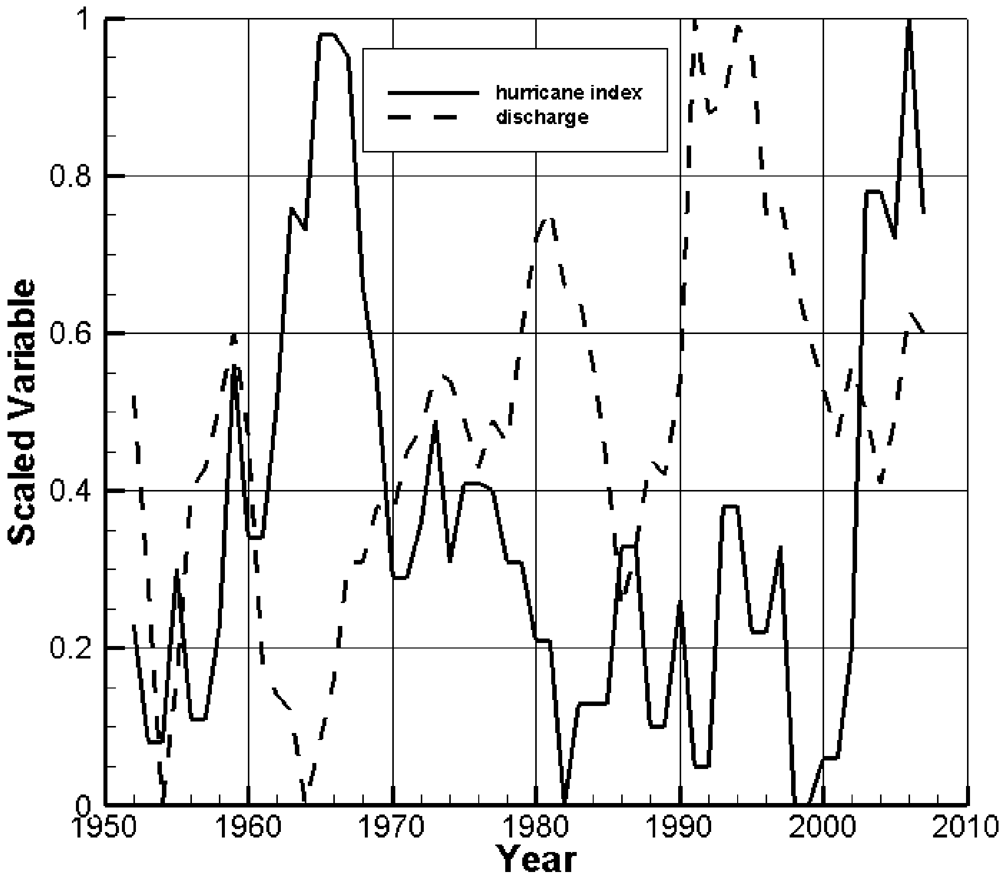

Figure 1. Display of scaled, smoothed hurricane index and river discharge values.

Appendix H – 3 Notes on Consideration of Monthly-Varying Intensities on Surge Levels within the Mississippi River

Table 1 shows the surge values estimated from an incorporation of monthly varying river discharges into their estimation, rather than assuming that the river flow is a fixed constant. In a recent review of the overall procedures used in the estimation of surge values in this area, a question was raised about the effect on these estimated values of also allowing storm intensities to varying on a monthly basis. A sensitivity study undertaken to answer this question is described here.

The first problem one encounters with the subdivision of the hurricane population into separate monthly sub-populations is the lack of sample size. Table 2 gives the number of hurricanes (using the same selection criterion as used the original study method described in IPET Volume 8). We see that both June and July only have 1 storm in their sample population. This precludes the use of any measure of intensity that depends on more than 1 parameter. In keeping with this constraint and since this investigation should be viewed only as a sensitivity study, we shall allow the distribution of surge levels, η, in each month to have the exponential form

1. $F_i(\eta) = e^{-(\lambda\eta - b_i)}$

where the subscript "i" denotes the i^th month and λ and b_i are the slope and offset parameters of the distribution. Only the "b" parameter is allowed to vary by month for two reasons. First, the limited data on a per monthly basis would not allow a suitable solution of two parameters; and second, the sum of the monthly distributions should be constrained to add up to the overall distribution, since this is a much more stable estimate of the overall hurricane population.

If we use the subscript "0" to denote the properties of the annual population, this constraint can be written as

2. $\sum p_i F_i(\eta) = \sum p_i F_0(\eta) \exp(b_0 - b_i) = F_0(\eta)$

or

3. $\sum p_i \exp(b_0 - b_i) = 1$

where p_i is the probability of a hurricane in the i^th month (the fraction the annual storms in that month). Again noting the lack of sample size and given the fact that surge levels have been shown to depend approximately linearly on the pressure differential, the best option for estimating the monthly "b" parameters is to assume that the value of b should be proportional to the average pressure different in each monthly sample. Table 3 gives the estimated average values of the pressure differential for each month, along with the values of p_i. If we allow the total population parameter b_0 to be given by the average pressure differential for all months considered together, we can solve equation 3 for the set of normalized $b_i{}'s$.

Using equation 2, we can proceed to estimate the effects of allowing the intensity to vary by month in this sensitivity study (Table 4). As can be seen there, the differences are not too large; and, given the lack of sample size in some of the sub-populations and the simplifying approximations that had to be used here, it is recommended that the original annual statistics be used as the design basis for surges within the Mississippi River rather than these new values.

Table 1

Location	η_{50}	ε_{50}	η_{100}	ε_{100}	η_{500}	ε_{500}
100	8.24	0.53	10.34	0.61	12.62	0.80
101	10.17	0.57	12.97	0.66	15.44	0.87
166	5.28	0.34	6.54	0.39	8.01	0.51
167	6.33	0.35	7.65	0.40	9.14	0.52
168	7.78	0.43	9.67	0.50	11.52	0.65
169	9.57	0.61	12.03	0.71	14.69	0.93
170	11.43	0.67	14.38	0.78	17.29	1.02
171	12.19	0.73	15.29	0.85	18.46	1.11
172	12.55	0.78	15.99	0.89	19.34	1.17
173	12.36	0.76	15.88	0.88	19.16	1.15
174	12.65	0.78	16.28	0.90	19.64	1.18
175	12.93	0.73	16.28	0.85	19.44	1.11
176	13.31	0.76	16.36	0.87	19.62	1.14
177	13.41	0.76	16.37	0.88	19.64	1.15
178	13.31	0.75	16.22	0.87	19.47	1.14
179	13.12	0.76	16.14	0.88	19.44	1.15
180	12.74	0.81	15.92	0.94	19.43	1.23
181	12.65	0.80	15.95	0.93	19.41	1.21
182	12.65	0.81	15.98	0.94	19.49	1.23
183	12.74	0.87	16.08	1.00	19.84	1.31
186	13.22	1.15	17.39	1.33	22.37	1.74
187	13.22	1.20	17.35	1.38	22.52	1.81
188	13.31	1.44	17.12	1.66	23.33	2.17
189	13.50	1.54	17.32	1.78	23.97	2.33
190	13.70	1.62	17.45	1.87	24.47	2.45
229	3.66	0.29	4.25	0.34	5.10	0.44
592	6.14	0.39	7.24	0.45	8.91	0.58
593	5.66	0.41	7.14	0.47	8.91	0.62
594	7.11	0.74	9.54	0.85	12.71	1.11
595	7.29	0.46	9.24	0.53	11.21	0.69
671	13.03	1.02	16.82	1.18	21.23	1.54
702	12.84	0.86	16.27	1.00	20.00	1.31
703	12.93	0.90	16.48	1.04	20.36	1.36
704	12.93	0.94	16.61	1.08	20.64	1.41
705	12.84	0.94	16.62	1.09	20.69	1.43
761	12.84	0.89	16.28	1.02	20.10	1.34

Table 2

	June	July	August	September	October
# of storms	1	1	3	6	4
Fraction of total (p_i)	0.0667	0.0667	0.2000	0.4000	0.2666

Table 3

	June	July	August	September	October
$\langle \Delta p_i \rangle$	50.0	60.0	85.0	61.7	53.3
$b_i -$ normalized	0.791	0.949	1.345	0.976	0.885

Table 4

Location	η_{50}	ε_{50}	η_{100}	ε_{100}	η_{500}	ε_{500}
100	8.25	0.53	10.35	0.61	12.65	0.80
101	10.18	0.58	12.98	0.66	15.46	0.87
166	5.29	0.34	6.55	0.40	8.03	0.52
167	6.34	0.35	7.66	0.40	9.17	0.53
168	7.80	0.44	9.68	0.51	11.57	0.66
169	9.59	0.62	12.05	0.72	14.73	0.94
170	11.45	0.67	14.45	0.78	17.36	1.02
171	12.21	0.74	15.32	0.86	18.53	1.12
172	12.59	0.78	16.02	0.90	19.38	1.17
173	12.40	0.78	15.92	0.90	19.27	1.17
174	12.68	0.78	16.32	0.90	19.68	1.18
175	12.97	0.73	16.32	0.85	19.49	1.11
176	13.35	0.74	16.49	0.86	19.70	1.12
177	13.45	0.74	16.51	0.85	19.70	1.12
178	13.35	0.76	16.32	0.88	19.62	1.15
179	13.16	0.78	16.19	0.90	19.55	1.18
180	12.78	0.83	16.04	0.96	19.62	1.26
181	12.68	0.82	15.98	0.95	19.53	1.24
182	12.68	0.76	16.41	0.88	19.69	1.15
183	12.78	0.85	16.33	0.98	20.00	1.28
186	13.26	1.21	17.45	1.40	22.68	1.83
187	13.26	1.19	17.98	1.37	23.11	1.79
188	13.35	1.51	17.16	1.74	23.69	2.28
189	13.54	1.63	17.35	1.88	24.38	2.46
190	13.73	1.70	17.63	1.96	24.98	2.57
229	3.67	0.29	4.26	0.34	5.11	0.44
592	6.15	0.39	7.25	0.45	8.92	0.59
593	5.67	0.41	7.15	0.48	8.93	0.63
594	7.12	0.74	9.55	0.85	12.74	1.12
595	7.29	0.46	9.25	0.53	11.23	0.70
671	13.07	0.94	17.40	1.08	21.45	1.42
702	12.87	0.87	16.43	1.00	20.18	1.31
703	12.97	0.93	16.57	1.07	20.58	1.40
704	12.97	0.97	16.68	1.12	20.86	1.46
705	12.87	0.88	17.10	1.02	20.91	1.33
761	12.87	0.88	16.47	1.02	20.29	1.34

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Appendix I

Wave Assessment Mississippi River
(Version 26 May 2010, Authors: M.V. Ledden, M. Agnew, M. Kluyver)

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APPENDIX I – WAVE ASSESSMENT MISSISSIPPI RIVER

Authors: Mathijs Van Ledden, Maxwell Agnew, Maarten Kluyver

Version 5/26/2010

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1.0 INTRODUCTION

1.1 PURPOSE

This report describes the derivation of the 1% exceedence significant wave heights and 1% peak wave periods for points along the co-located Hurricane and Storm Damage Risk Reduction System levees (HSDRRS) and Mississippi River levees (MRL).

Developing accurate design wave and surge conditions is important for the design of the MRL project, especially for the levee segments which overlap the HSDRRS system. **Figure 1** shows the location of the MRL with respect to the overall HSDRRS system. MRL that border the Lake Pontchartrain and Vicinity and West Bank and Vicinity projects are considered the “co-located” MRL/HSDRRS levees.

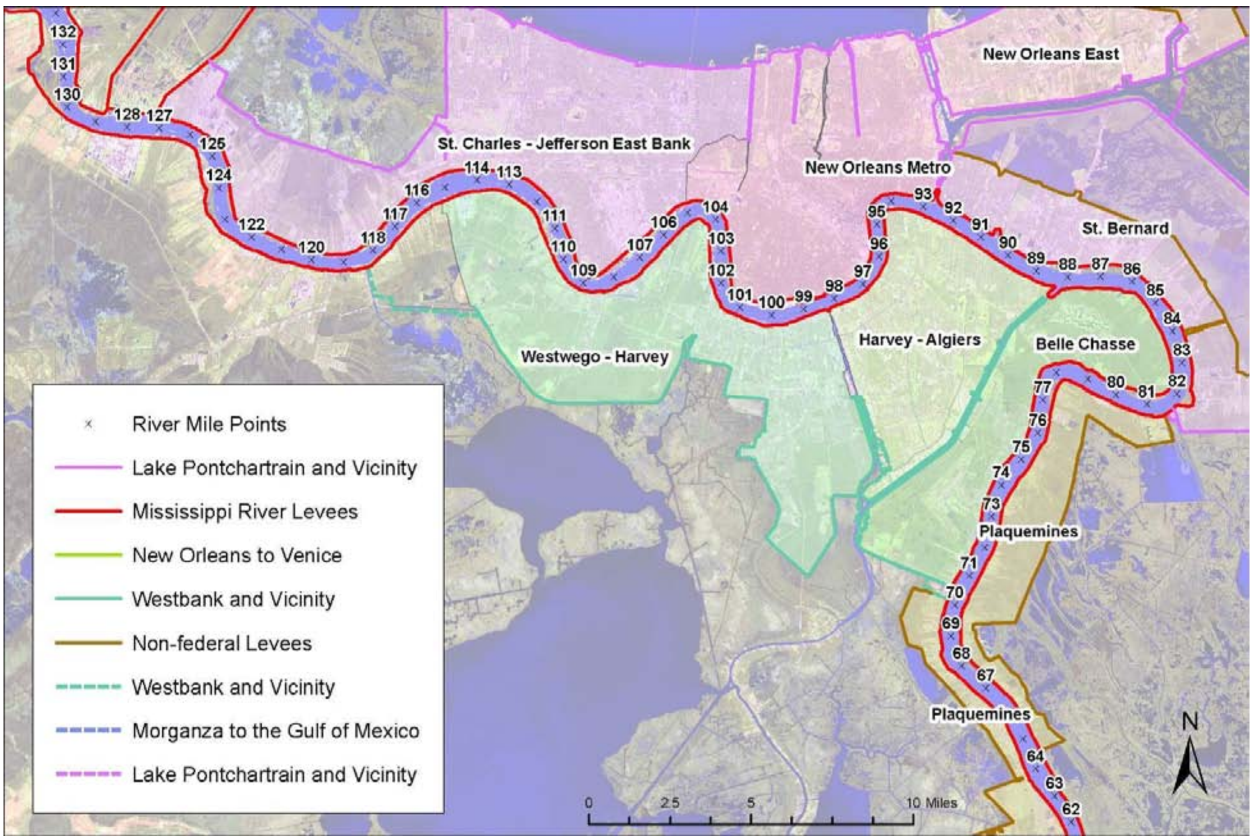


Figure 1 – Location of Co-Located MRL/HSDRRS Levees

The co-located MRL/HSDRRS levees are likely to be a challenge to upgrade, due to the levees close proximity to infrastructure, residential, and business properties. **Figure 2** shows an example of infrastructure in close proximity to the MRL floodwalls at the intersection of Canal and Poydras St. in downtown New Orleans.

Figure 2 – Floodwall at Canal and Poydras Street in Downtown New Orleans

1.2 WAVE CHARACTERISTICS: ORIGINAL APPROACH

In the original analysis, calculations were made using the Brettsneider wind-wave equation. The empirically based Brettsneider equation calculates significant wave height (Hs) and peak wave period (Tp) given a fetch, a constant water depth, and a wind speed. The Brettsneider wind-wave equations are provided below:

$$Hs = \left(0.283u^2/g\right)\tanh\left(0.578\left(gD/u^2\right)^{0.75}\right)\tanh\left(\frac{0.0125\left(gF/u^2\right)^{0.42}}{\tanh\left(0.578\left(gD/u^2\right)^{0.75}\right)}\right)$$

$$Ts = \left(2.4\Pi u/g\right)\tanh\left(0.8333\left(gD/u^2\right)^{0.375}\right)\tanh\left(\frac{0.0777\left(gF/u^2\right)^{0.25}}{\tanh\left(0.833\left(gD/u^2\right)^{0.375}\right)}\right)$$

$$Tp = Ts/0.9$$

Where:
Hs = significant wave height [foot or feet (ft)]
Ts = significant wave period (sec)
Tp = peak wave period (sec)
u = wind velocity (m/s)
D = water depth (m)
F = fetch (m)
g = acceleration due to gravity (m/s^2)

The design waves for the MRL were calculated with the Brettsneider equation using a fetch of 0.3 miles, a depth of 30 meters, and a 1% chance exceedence wind speed of 77 miles per hour (mph). One assumption of the original analysis was the use of a constant fetch of 0.3 miles. In the Mississippi River, fetch length is heavily dependent on wind direction. Another assumption of the original analysis was the use of 1% chance exceedence wind speed of 77 mph. This 1% wind speed was taken from the original HSDRRS design report and was used for design where STWAVE model results were not available. The average depth of the Mississippi River was assumed to be 30 meters. This approach resulted in a design significant wave of 2.5 ft and a corresponding wave period of 3.2 seconds (sec). These design wave characteristics were originally applied for all co-located MRL levees above river mile (RM) 44.

1.3 WAVE CHARACTERISTICS: NEW APPROACH

One of the outcomes of the MRL design summit was a request for re-evaluation of design wave characteristics for the MRL/HSDRRS co-location levees. Since no detailed wave information was available from any of the previous ADCIRC/STWAVE model runs, a new approach had to be developed. A more refined analysis determined the significant wave height and peak wave period for each of the 152 synthetic ADCIRC storms at a series of points in the river. The new process for determination of the design waves for the MRL system consists of 2 steps:

Step 1: Determine the wave characteristics at the peak surge level for all 152 storms using an empirical approach based on fetch, wind speed, and local water depth.

Step 2: Calculate the 1% wave height and wave period using the 152 storm wave heights and wave periods from the previous step using the storm probabilities.

The new analysis also utilizes the Brettsneider equation, but accounts for the varying wind direction, wind speed, and fetch of each of the 152 synthetic storms.

1.4 OUTLINE OF REPORT

This report is organized as follows: The determination of the wave characteristics for each storm at each river point (Step 1) is discussed in **Section 2.0**. The procedure to define the wave statistics based on the individual storm results (Step 2) follows in **Section 3.0**. **Section 4.0** and **Section 5.0** present the existing and future wave statistics. Finally, **Section 6.0** summarizes the wave characteristics proposed to be used in defining the 1% levee elevations for the co-located MRL system.

2.0 STEP 1: DETERMINE THE WAVE CHARACTERISTICS FOR 152 STORMS

2.1 OVERVIEW OF STEP 1

The following methodology has been applied to determine the wave characteristics for the 152 storm suite:

- Define the boundaries of the Mississippi River and also output points along the Mississippi River at both East Bank and West Bank **(Section 2.2)**.
- Extract for every output point along the Mississippi River for all 152 storms the wind speed, wind direction, and surge level from the ADCIRC computations **(Section 2.3)**.

- Calculate for every output point along the Mississippi River the fetch for all possible wind directions **(Section 2.4)**.
- Calculate for every output point along the Mississippi River the bed level elevation of the batture near the levee **(Section 2.5)**.
- Calculate for every output point along the Mississippi River and for all 152 storms the wave height and wave period at the moment of peak surge level based on the wind, surge, fetch, and bed level information from the previous steps **(Section 2.6)**.
- Apply a reduction factor to the wave heights to account the effect of non-perpendicular wave angles **(Section 2.7)**.
- Validate the computed wave characteristics against field data **(Section 2.8)**.

The final wave heights and wave periods from **Sections 2.6** and **2.7** are input for Step 2 to determine the wave statistics which is discussed in **Section 3.0**.

2.2 MODEL BOUNDARY AND OUTPUT POINTS

A levee boundary file was created for the area of interest using the EGIS database. The Brettsneider model covers RM 65 to 130. Model output points were assigned at every RM from 65 to 130 for the east and west banks of the river. **Figure 3** shows the location of the extracted levee alignment data and the assigned output points for the model.

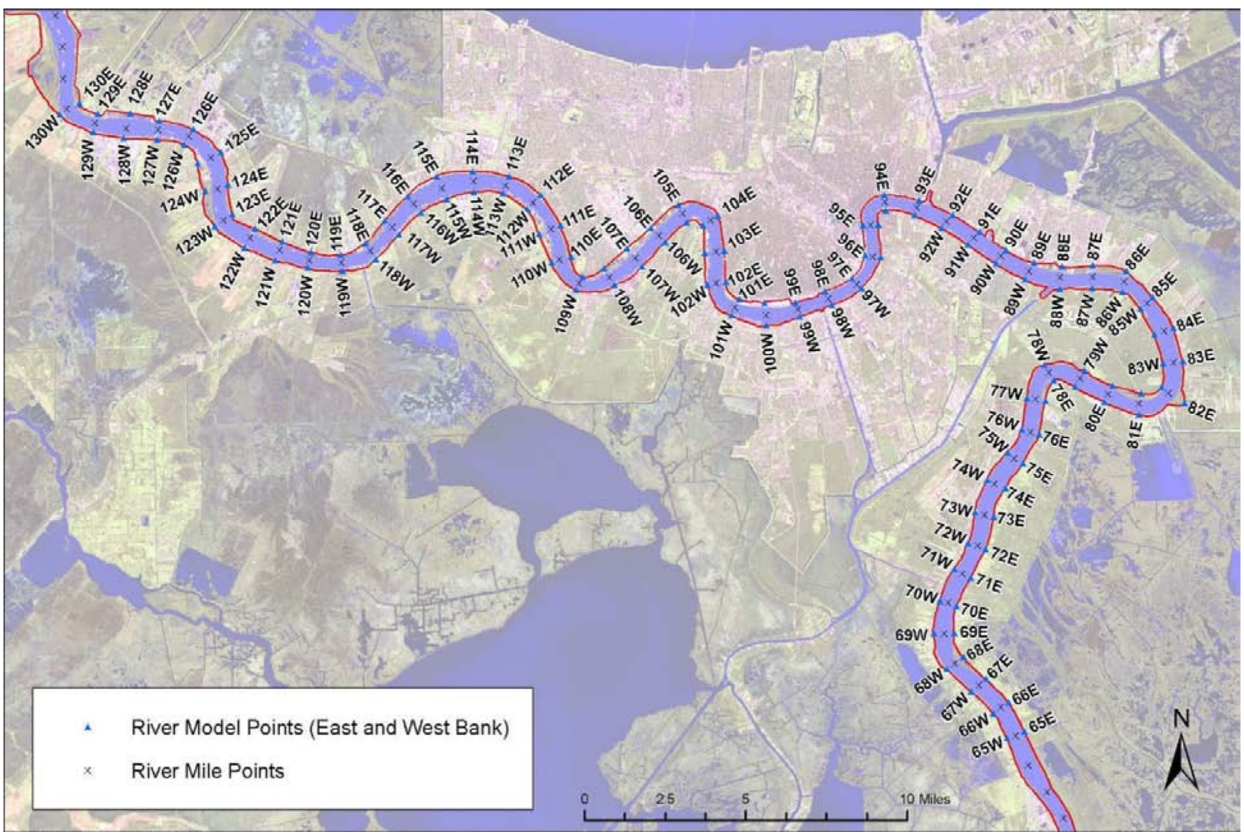


Figure 3 – Overview of Brettsneider Wind-Wave Model (River Mile 65 to 130)

2.3 WIND AND SURGE TIME SERIES

Wind and surge time series were extracted from each of the 152 storms that were used for the design of the Inner Harbor Navigational Canal (IHNC) Surge Barrier. This set of ADCIRC storms is known as the "IHNC set". The river discharge used for this storm set was 400,000 cubic feet per second (cfs). The wind and surge results of these storms were selected as input into the Brettsneider wave model because 1) The IHNC set is a complete set of 152 storms. 2) The river discharge of 400,000 cfs is close to the average flow during hurricane season. 3) The 1% surge elevations produced from the IHNC surge results are similar to the 1% surge elevations determined from JPM-OS with variable river flow.

Wind time series for each of the 152 synthetic storms are stored in the ADCIRC fort.74 output file. The fort.74 output file stores wind time series after the directional wind drag reduction values have been applied. Wind time series were extracted for each of the 152 storms, in the center of the river, from RM 1 to 199. The wind velocity time series from the ADCIRC fort.74 file are recorded in x and y velocity format. The x and y velocities were decomposed to direction and magnitude. In the model computations, the fetch values are interpolated from the fetch file for the given wind direction.

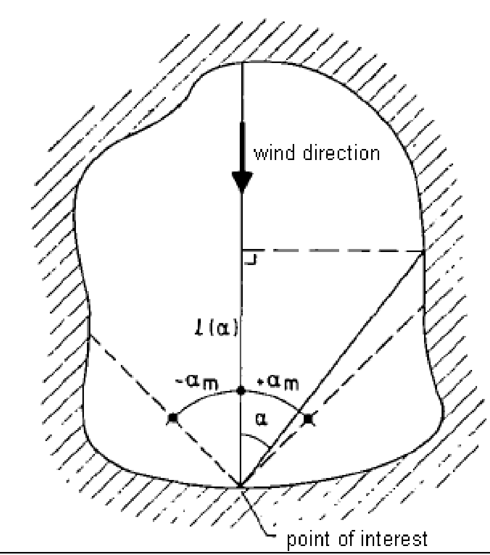
The water surface elevation time series are recorded in the ADCIRC fort.63 file. Water surface elevation hydrographs were extracted from each of the 152 synthetic storms. The discharge for the storm set was 400,000 cfs. In the wave model computations, water surface elevation at each time step is pulled from the ADCIRC hydrograph file.

2.4 FETCH LENGTH

The first step of the modeling process was to determine the fetch length for a variety of wind directions at every model output point. Lit. [1] recommends using the "effective fetch" for calculating the wave parameters along the river. This effective fetch Fe is equivalent to a weighted averaged of the projected lengths l(α) on the wind direction of fetches:

$$F_e = \frac{\int_{-\alpha_m}^{+\alpha_m} w(\alpha) l(\alpha) d\alpha}{\int_{-\alpha_m}^{+\alpha_m} w(\alpha) d\alpha} \qquad (1)$$

Herein, w(α) is a weighting function. It is recommended in Lit. [1] to use w(α) = cos(α) and αm = 45º. For definitions, see figure modified from lit. [1].



The actual fetch length for each wind direction is needed to compute the effective fetch length Fe in Equation (1). For this purpose, a polygon has been created of the Mississippi River with increments of 100 ft. The polygon was based on the centerlines of the MRL at the east bank and west bank and in EGIS database from the New Orleans District. Next, wind directions were chosen from 0 degrees to 360 degrees in 10 degree increments. The fetch has been determined for each wind direction for all points along the river.

A Matlab code was used to develop the actual fetch for each point and each wind direction. A visualization of the fetch calculations is provided in **Figure 4** for one point along the river. **Figure 5** shows the fetch and the effective fetch for each wind direction using Equation (1). It is clear and obvious from this figure that the effective fetch has a smoothing effect. This "effective fetch" has been carried forward in the computation of the wave parameters.

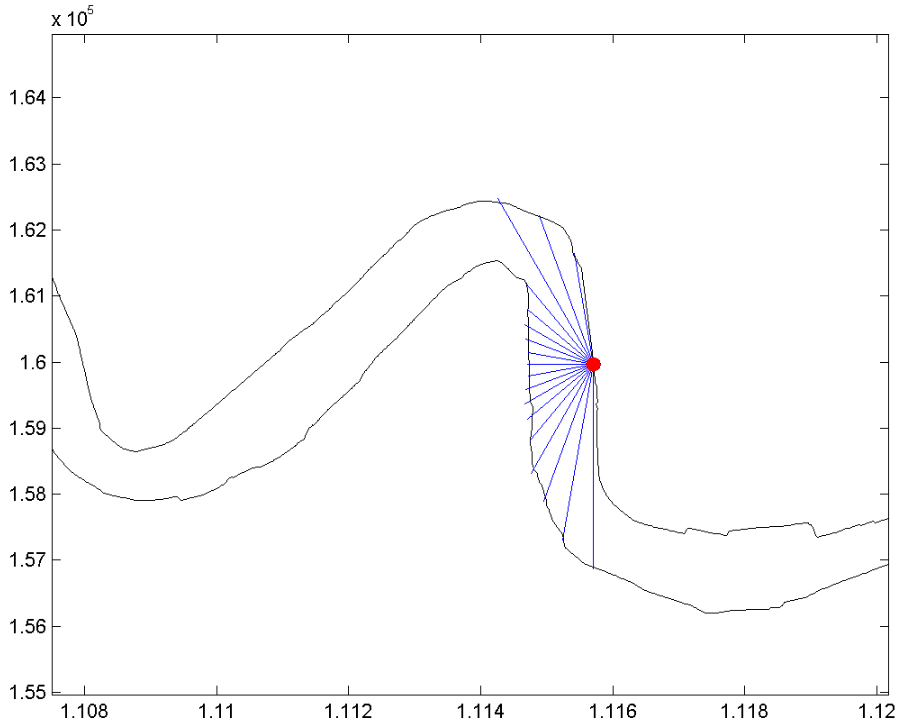


Figure 4 – Visualization of Fetch Length Determination near Carrolton at Location 103E

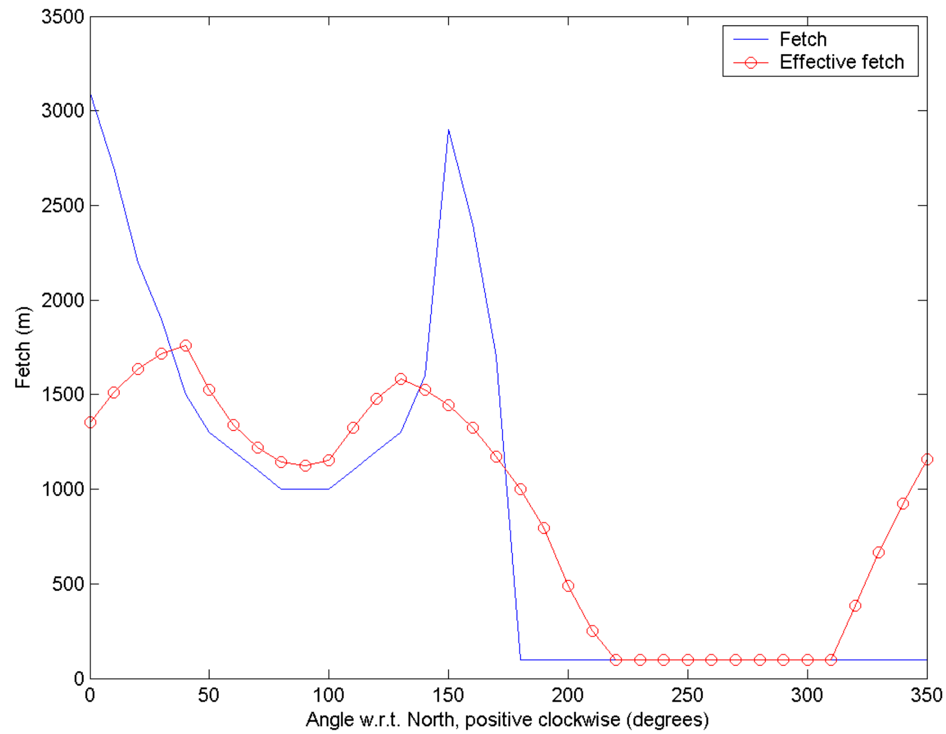


Figure 5 – Fetch and Effective Fetch for a Location near Carrollton at Location 103E

2.5 BATTURE WIDTH AND ELEVATION

The area between the levee toe and the river bank is known as the batture. Depending on the batture elevation, this area can become inundated with high stages in the Mississippi River, allowing waves to propagate over the batture and directly impact the levee. In other cases, there is no batture to separate the river from the levee. **Figure 6** shows a typical batture of the Mississippi River on the west bank at RM 94.

Figure 6 – Typical Batture at River Mile 94 on West Bank

The elevation and width of the batture has a strong influence on incoming waves. Two cases can exist during high stages in the river: 1) The batture elevation is sufficiently high to prevent surge and waves from reaching the levee. 2) The batture is inundated, allowing waves to propagate to the levee. **Figures 7** and **8** shows the two different wave scenarios which can exist during high stages.

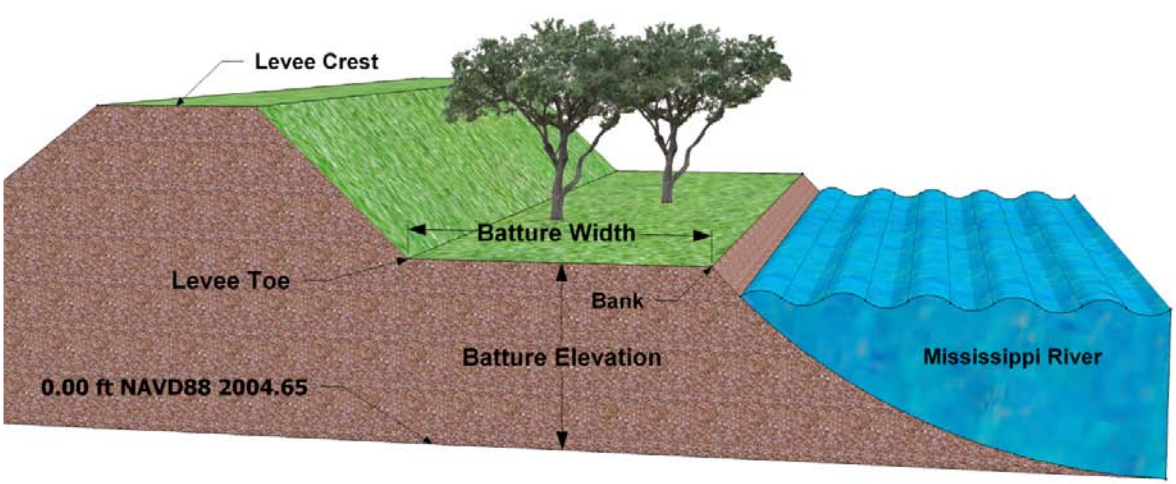


Figure 7 – Batture Schematic – No Inundation

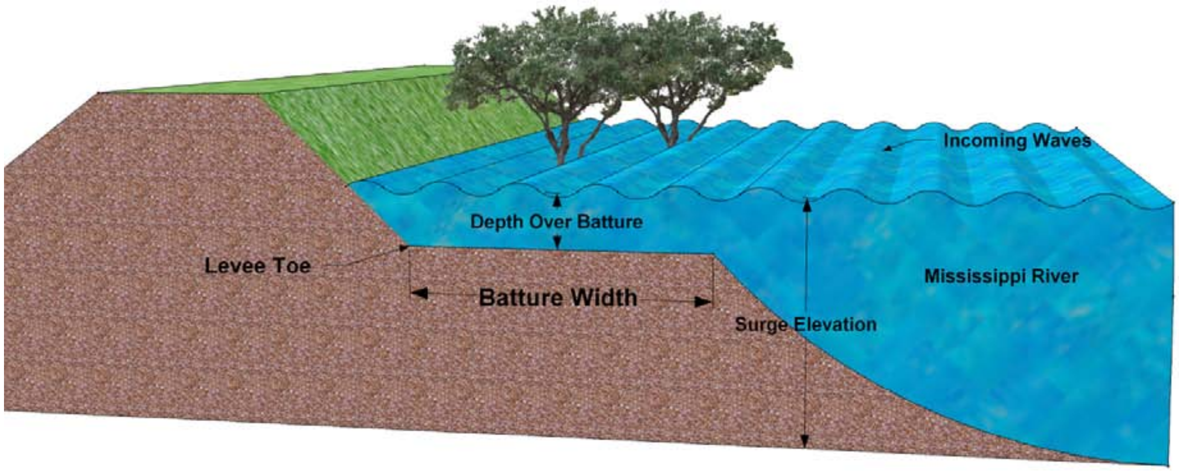


Figure 8 – Batture Schematic – With Inundation

When the batture is inundated, the depth of water over the batture and width of the batture become important parameters for wave calculations. A relatively small depth and long batture will force the incoming waves to reduce in height and become depth-limited. A relatively large depth and short batture will have little influence on incoming waves. Wave periods are assumed to not be altered by batture characteristics. The wave transformation process has been incorporated into the model using the following formulations:

1) When the surge elevation is less than the batture elevation, wave height and wave period at the levee toe equal zero.

2) When the surge elevation is greater than the batture elevation, and the incoming wavelength is less than the batture width, waves are depth-limited and reduced to 40% of the batture depth.

3) When the surge elevation is greater than the batture elevation and the incoming wavelength is greater than the batture width, waves do not become fully depth-limited. Wave heights are reduced linearly from the incoming value to 40% of the batture depth. The percent reduction is dependent upon the width of the batture compared to the incoming wavelength.

A representative batture width and elevation were determined for every model point from RM 65 to 130 using a 1 ft by 1 ft LIDAR Corps survey of the Mississippi River levee system which was published on August 1, 2007. A polygon was drawn to encompass the area between the levee toe and river bank at one half mile upstream to one half mile downstream from each river mile point. ArcMap with Spatial Analyst was used to determine the average and standard deviation of ground surface elevation. **Figure 9** shows the 2007 LIDAR coverage and batture polygon at RM 94 on the West Bank.

Figure 9 – Batture LIDAR on West Bank at River Mile 94

To avoid computational limitations associated with the large file size of the 1.0 ft by 1.0 ft resolution LIDAR, the resolution was re-sampled to be 10.0 ft by 10.0 ft. This allowed the use of the ArcMap Zonal Attributes tool, which calculates the average and standard deviation of the LIDAR data encompassed by a polygon.

A batture width at each river mile was determined by drawing and measuring a line from the levee toe to the river bank. The batture width was chosen to be representative of the entire mile. However, to be conservative, the minimum batture width was chosen. The batture width for RM 94 on the West Bank is shown on **Figure 9**. **Table 1** presents the average batture elevation, the standard deviation of batture elevation, and the selected batture width for the East and West banks at RM 65 to 130.

Table 1 – Batture Width and Elevation River Mile 65 to 130

West Bank				East Bank			
Point ID	Mean Batture Elevation (ft)	Batture Elevation Standard Deviation (ft)	Batture Width (ft)	Point ID	Mean Batture Elevation (ft)	Batture Elevation Standard Deviation (ft)	Batture Width (ft)
65W	5.26	1.06	90.3	65E	6.02	1.35	89.3
66W	6.09	1.27	132.8	66E	6.80	1.49	128.6
67W	6.43	1.46	192.9	67E	6.47	1.43	89.4
68W	6.18	1.32	148.4	68E	7.32	1.34	301.7
69W	5.81	1.01	131.9	69E	6.29	1.53	150.3
70W	5.44	1.16	84.7	70E	7.11	1.79	136.0
71W	6.13	1.28	235.8	71E	6.29	1.75	72.1
72W	7.44	1.73	112.9	72E	6.36	1.78	297.6
73W	8.04	1.96	158.3	73E	6.75	1.60	121.7
74W	6.73	1.17	264.9	74E	7.38	1.77	190.2
75W	6.24	0.80	300.3	75E	7.24	1.65	143.2
76W	8.60	2.29	110.0	76E	7.29	1.66	172.2
77W	7.83	1.55	112.0	77E	6.74	1.73	276.4
78W	8.44	1.80	312.0	78E	7.69	1.93	194.7
79W	8.64	1.83	213.8	79E	7.99	1.97	141.4
80W	8.61	1.70	220.6	80E	8.11	1.86	218.5
81W	9.29	1.75	820.4	81E	8.22	1.85	60.0
82W	9.42	2.01	101.5	82E	7.48	2.14	379.0
83W	8.85	1.75	171.9	83E	8.57	1.83	204.4
84W	7.21	2.14	103.2	84E	8.81	1.93	201.7
85W	8.85	4.49	454.9	85E	8.18	1.59	192.1
86W	9.14	2.01	115.2	86E	6.77	1.88	84.3
87W	8.48	1.89	170.2	87E	7.26	2.78	252.3
88W	7.62	2.24	90.4	88E	11.11	6.08	286.6
89W	9.08	2.11	80.1	89E	10.84	2.53	386.0
90W	8.88	1.85	54.2	90E	8.15	2.53	89.2
91W	9.42	2.06	112.0	91E	13.74	4.46	41.2
92W	9.06	1.72	143.2	92E	7.16	1.82	91.9
93W	10.21	4.16	67.1	93E	14.58	5.23	226.2
94W	10.47	3.69	150.4	94E	15.20	3.67	137.7
95W	11.86	3.41	140.4	95E	15.78	4.44	88.3
96W	11.43	4.33	140.5	96E	16.00*		252.6
97W	8.24	2.16	82.6	97E	14.00*		252.6
98W	9.64	2.90	72.1	98E	14.00*		331.4
99W	9.42	2.97	97.4	99E	12.00*		210.0
100W	11.21	2.96	220.0	100E	17.00*		1095.8
101W	11.72	3.29	313.8	101E	17.00*		708.7
102W	14.61	5.64	129.3	102E	17.35	4.37	220.6
103W	11.39	6.03	421.4	103E	12.20	3.59	162.8
104W	10.67	3.74	344.3	104E	12.86	3.76	277.6
105W	13.03	4.45	111.0	105E	12.01	3.72	190.3
106W	10.58	3.37	73.9	106E	13.50	4.65	712.9
107W	15.68	4.34	133.7	107E	19.90	5.66	1048.3
108W	15.70	4.07	260.8	108E	14.41	6.38	640.0
109W	10.52	3.40	90.8	109E	11.36	2.43	279.3
110W	9.60	3.86	204.9	110E	11.34	2.71	434.8
111W	12.93	7.84	1173.0	111E	12.23	2.33	197.9
112W	14.55	6.55	542.3	112E	13.89	3.28	330.8
113W	11.48	2.85	73.6	113E	12.08	2.60	317.8
114W	10.48	2.66	110.0	114E	12.89	2.69	290.0
115W	10.94	3.51	138.2	115E	15.69	4.51	363.2
116W	12.20	3.54	141.8	116E	15.30	6.97	207.1
117W	12.57	3.29	179.0	117E	11.03	2.79	332.9
118W	12.61	2.67	249.9	118E	12.66	1.94	570.8
119W	12.10	2.90	80.2	119E	13.42	4.27	90.7
120W	12.51	3.55	82.4	120E	13.18	4.45	112.6
121W	13.96	3.65	179.2	121E	11.99	2.54	211.9
122W	14.30	3.29	229.0	122E	13.19	2.62	223.3
123W	13.69	3.00	298.2	123E	13.05	3.97	191.5
124W	15.22	3.60	962.9	124E	14.74	3.17	200.3
125W	18.34	6.38	697.1	125E	15.02	2.99	287.8
126W	14.35	3.63	141.7	126E	12.74	3.81	128.4
127W	15.45	3.64	280.0	127E	11.07	4.97	333.0
128W	15.17	4.07	260.4	128E	7.41	2.59	762.4
129W	14.98	4.23	237.7	129E	14.16	4.72	598.5
130W	15.74	4.29	269.2	130E	15.44	2.95	410.5

* For RM 96 to 101 on the East Bank, the 1.0 ft by 1.0 ft resolution LIDAR data was unavailable. This area of the river is heavily developed and no natural batture exists. A different

LIDAR set, developed for FEMA in 1999, was sampled in this area to determine a representative batture elevation.

The 2007 LIDAR data was analyzed to determine whether or not batture geometry had been collected accurately. For validation purposes, cross-sections were extracted from the LIDAR data at various locations from RM 65 to 130. **Figure 10** shows the location of four of these cross-sections, two at RM 90 and two at RM 94. **Figure 11** shows plots of these four cross-sections. The most prominent features of each cross-section include the levee crest, the levee toe, the batture, the river bank, and the river channel. For each of these cross-sections, a clear and pronounced batture platform exists between the bank and levee toe. The bank is identified at the drastic change in slope from the batture to the river channel. Since the LIDAR survey was conducted at low water in August, a section of the river channel was also recorded. These channel elevations have been included in the calculations of mean batture elevation, which most likely results in a slightly lower mean batture elevation. A slightly lower batture elevation allows slightly larger waves to propagate to the levee, adding conservatism to the analysis. Given the variability of batture elevation, the extra conservatism is appropriate.

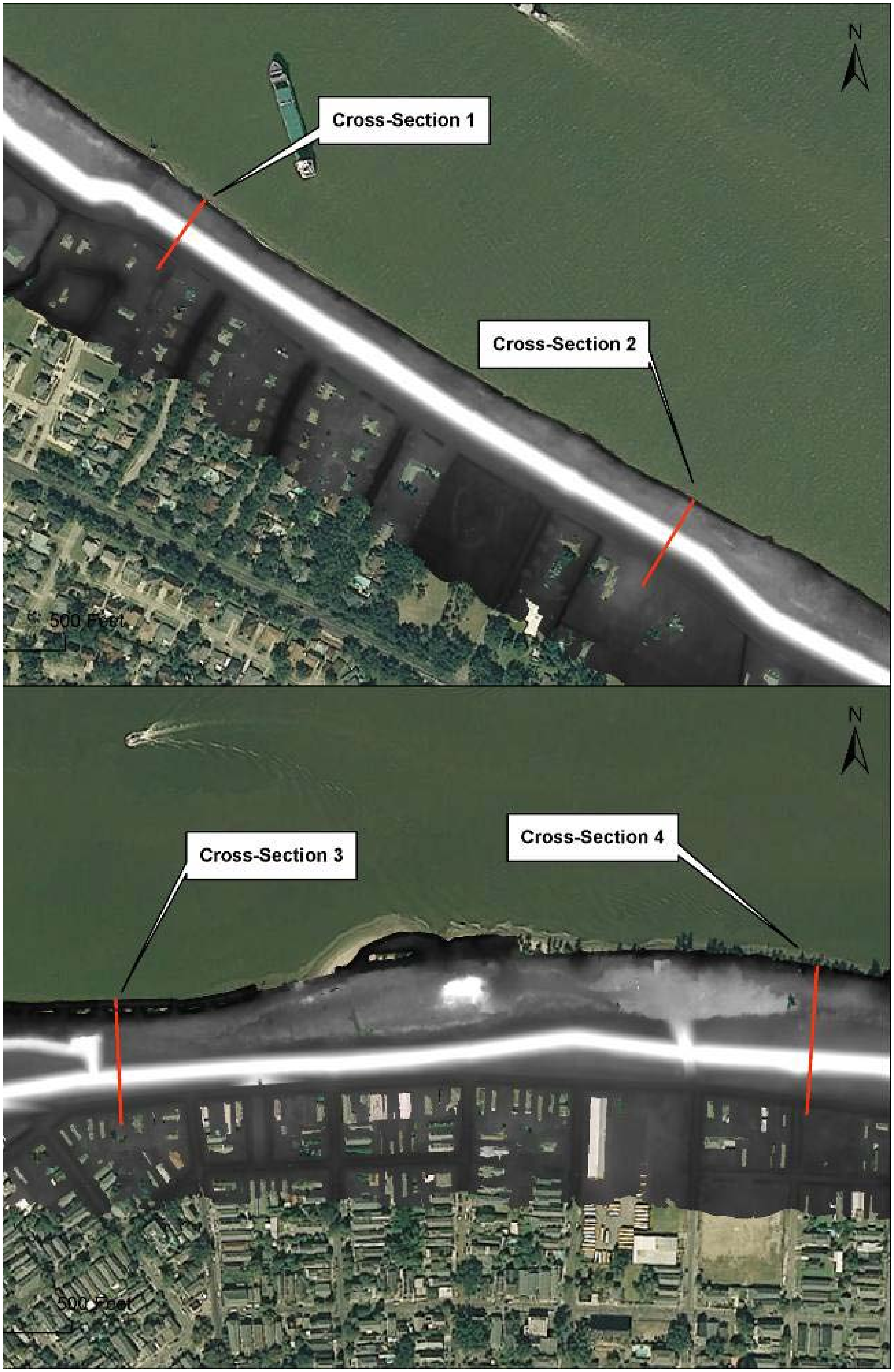


Figure 10 – Cross-Section Locations at River Mile 90 (top) and 94 (bottom)

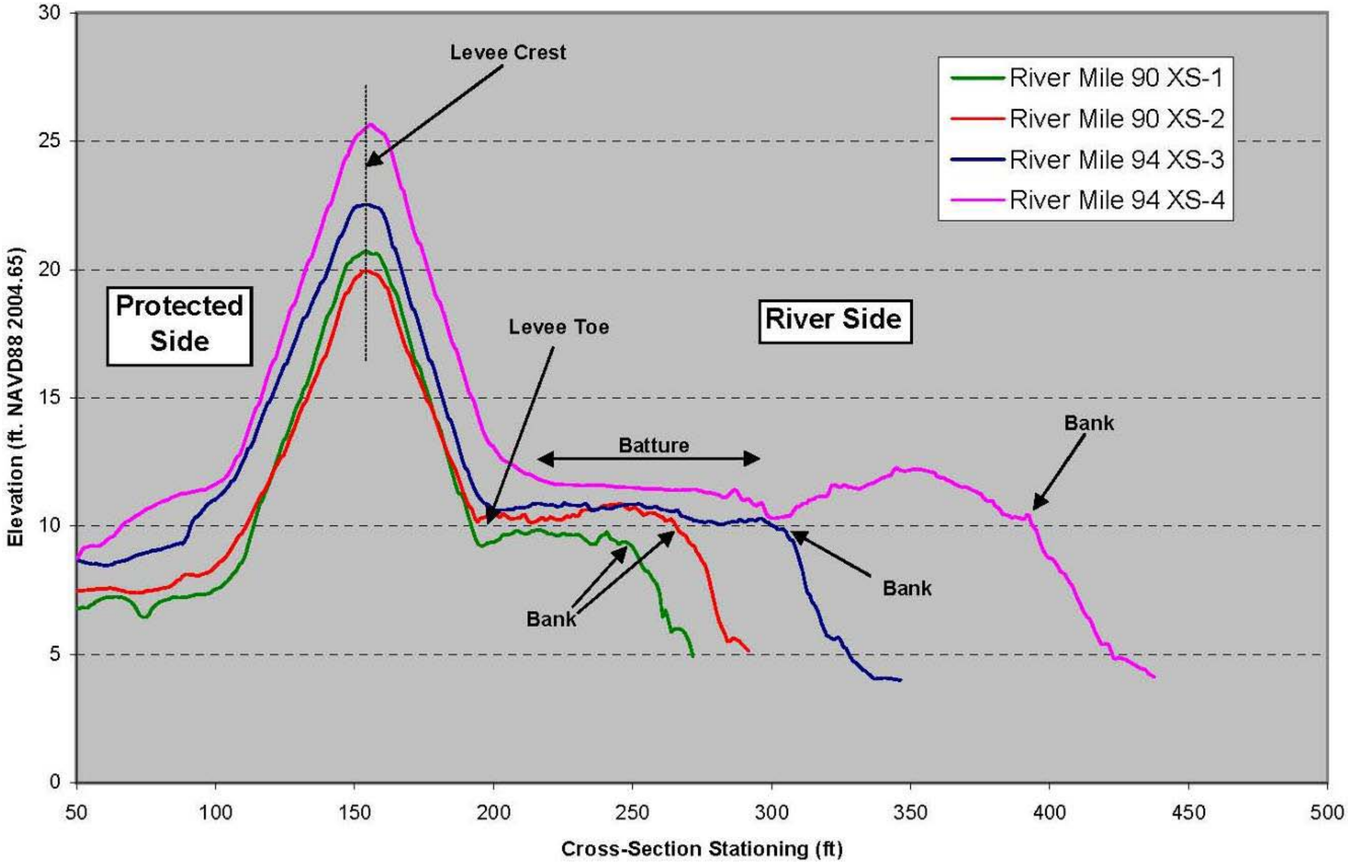


Figure 11 – Typical Levee Cross-Sections at River Miles 90 and 94

2.6 TIME SERIES OF WAVE CHARACTERISTICS

The outputs of each model run include time series of significant wave height and peak wave period in the deep water of the river. **Figure 12** is an example of the deep water output for Storm 053 at RM 65 on the West Bank. The next step of the modeling process is to modify the incoming wave heights given the batture characteristics. **Figure 13** is an example of the final model output time series at model point 65W for Storm 053.

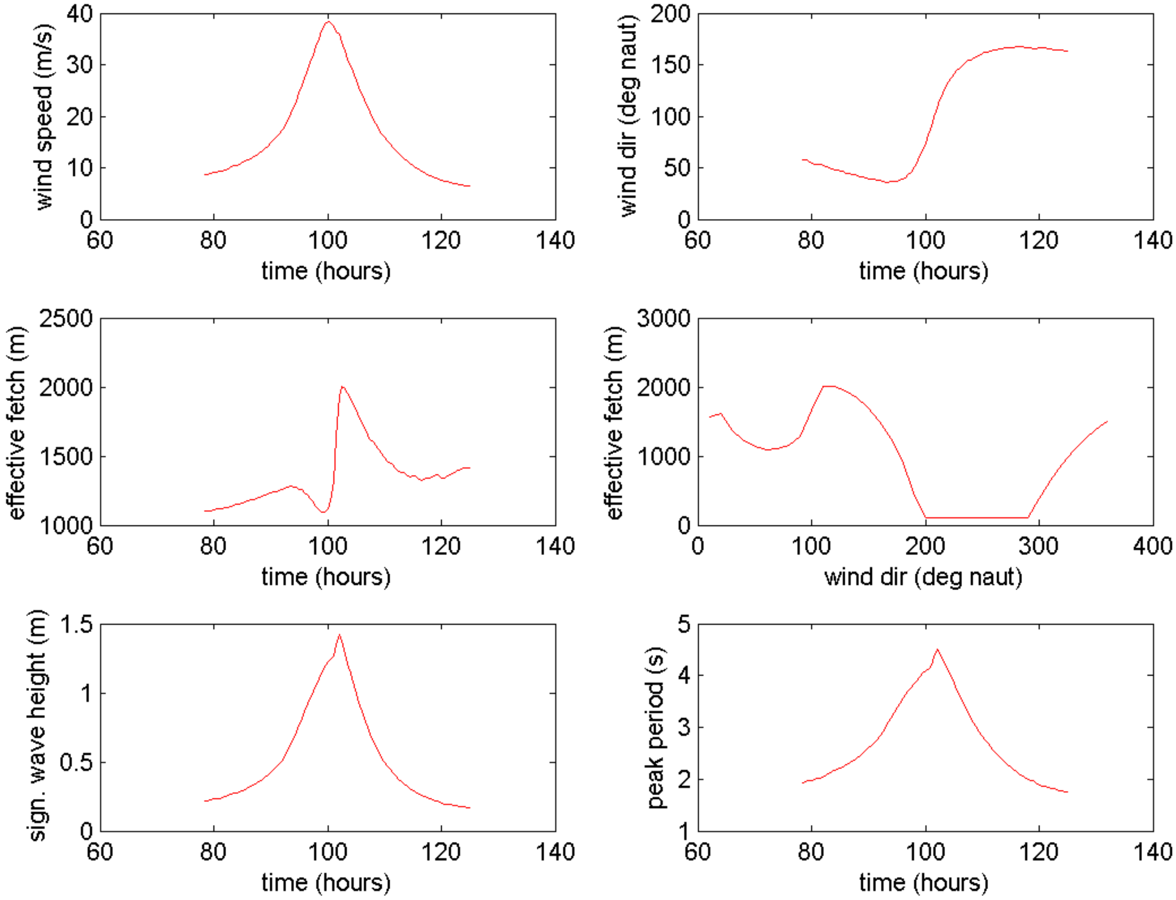


Figure 12 – Example of Deepwater Model Output Time Series at Model Point 65W for Storm 053

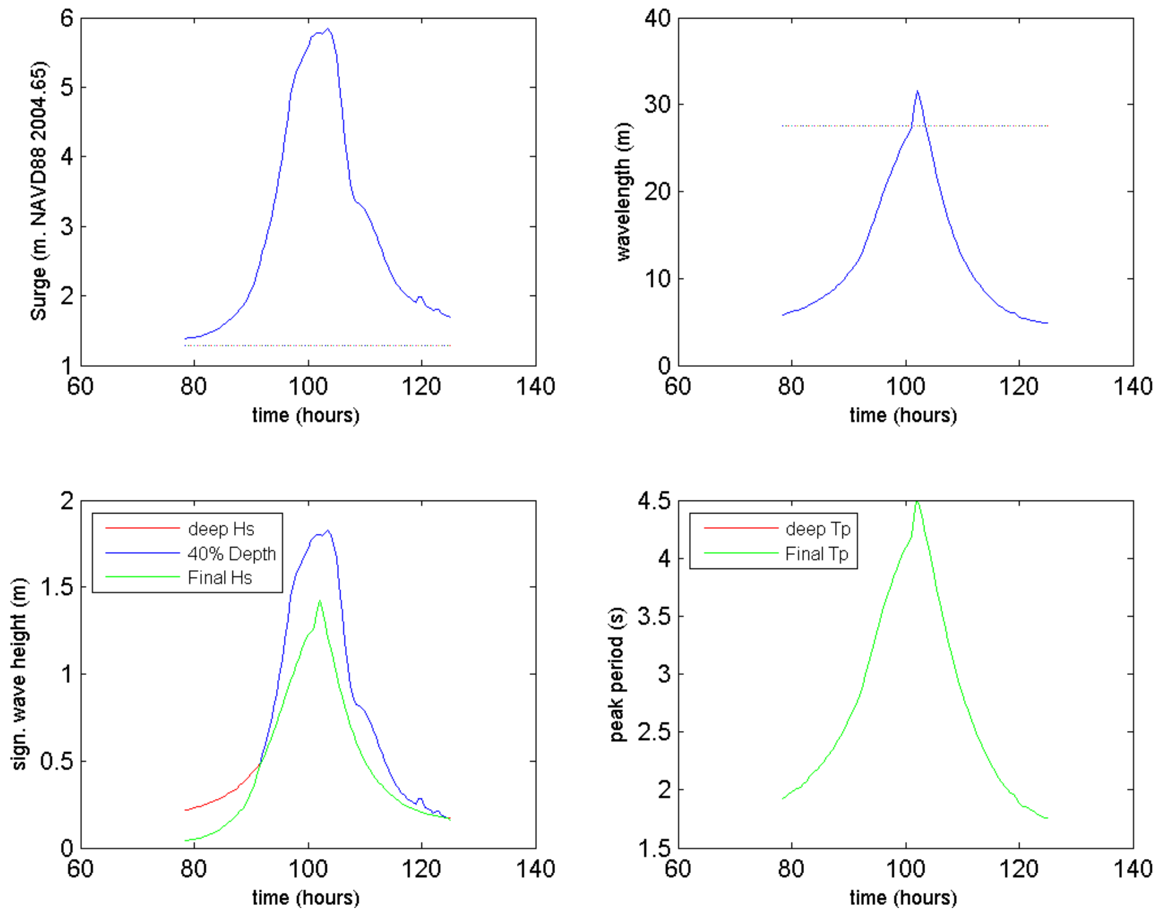


Figure 13 – Example of Final Model Output Time Series at Model Point 65W for Storm 053

Figure 13 shows an example of the surge, wave length, significant wave height, and wave period time series. The batture elevation is plotted as a thin black line on the surge time series plot. In this particular case, the surge elevation is always greater than the batture elevation. Therefore, wave time series at the levee toe are calculated for the entire storm. The batture width is plotted as a thin black line on the wavelength time series plot. In this particular case, the wavelength is greater than the batture width for a few hours at the peak of the wave height time series. The significant wave height plot has three time series: 1) The red line represents the incoming significant wave height. 2) The blue line represents 40% of the depth over the batture. 3) The green line represents the final selected wave time series. In the beginning of the storm, the incoming significant wave height is greater than 40% of the depth and the final selected significant wave height is taken as 40% of the depth. As the storm goes on, the incoming significant wave height becomes less than 40% of the depth, and the wave heights are no longer reduced due to the batture. For the entire storm, peak wave period is the same for incoming waves and waves at the levee toe. These time series plots were produced at dozens of model points for each storm to validate the model performance.

The significant wave heights and wave periods were extracted at the timing of the peak of the surge hydrograph for each storm and these are carried forward. Before using the wave heights to derive the wave statistics in Step 2, the wave heights had been modified to account for non-perpendicular incidence. This reduction for the wave heights is discussed in **Section 2.7**. Note that the wave period has not been modified.

2.7 WAVE ANGLE EFFECT

This section describes how the effect of a non-perpendicular incidence of waves to the levees has been included in the determination of the design waves for the co-located HSDRSS work of the Mississippi River. The reason for doing this is that the process of deriving the 1% levee design elevations assumes that the 1% design waves approach the levee perpendicular. In reality, however, the wave angle relatively to the levee orientation can vary substantially from the normal wave incidence depending on the particular storm and levee section along the MRL system. In the case of the waves within the river levees, it would therefore be overly conservative not to account for this effect.

To account for the wave angle effect, a reduction factor has been derived for the wave height for each individual storm and applied before the 1% design waves in Step 2 were being derived. The so-called wave angle factor to the wave height has been defined as follows:

$$f = \frac{H_1}{H_0} \tag{1}$$

where:
f = wave angle factor [-]
H_0 = wave height before reduction due to wave angle effect [m]
H_1 = wave height after reduction due to wave angle effect [m]

The angle factor f has been established by considering the overtopping rate for each particular storm at the moment of peak surge level. Following the HSDRSS guidelines, the overtopping rate over the levee is estimated using the empirical formulation of Van der Meer (TAW, 2002):

$$\frac{q}{\sqrt{gH_{m0}^{\ 3}}} = \min\left(\frac{0.067}{\sqrt{\tan\alpha}}\xi_0 \exp\left(-4.3\frac{h_k}{H_{m0}}\frac{1}{\xi_0\gamma_\beta}\right), 0.2\exp\left(-2.3\frac{h_k}{H_{m0}}\frac{1}{\gamma_\beta}\right)\right) \tag{2}$$

where:
q = overtopping rate [m^3/s/m]
g = gravitational acceleration [m^2/s]
H_{m0} = significant wave height [m]
ξ_0 = surf similarity parameter [-]
h_c = freeboard = $z_c - \zeta$ [m]
z_c = levee crest [m]
ζ = water level [m]
α = levee slope [-]
γ_β = wave angle factor [-]

Note that Equation (2) does not include several influence factors which are present in the original equation since the Mississippi River levees in the co-located reaches are smooth, and do not have wave berms and/or vertical walls on top of levees. Hence, the influence factors for these effects are all equal to 1.

The surf similarity parameter ξ0 in Eq. (2) is defined as:

$$\xi_0 = \frac{\tan\alpha}{\sqrt{s_0}} \tag{3}$$

where:
s_0 = wave steepness = 2 π Hm0 / (g Tm-102) [-]
Tm-102 = spectral wave period [s]

The influence factor γβ to account for the effect of non-perpendicular wave angles to the levees for wave overtopping with short-crested waves in Equation (2) is defined as:

$$\begin{aligned} \gamma_\beta &= 1 - 0.0033|\beta| \qquad |\beta| < 80^\circ \\ \gamma_\beta &= 1 - 0.0033 \times 80 \qquad |\beta| > 80^\circ \end{aligned} \tag{4}$$

where:
γ_β = influence factor for angled wave attack [-]
β = wave angle relative to normal incidence [º]

The wave angle factor f in Equation (1) has been set in such a way that the overtopping rate q is equal for the situation with non-perpendicular incidence (indicated with subscript 0) and the situation with perpendicular incidence (indicated with subscript 1). Or in formulas:

$$q_0(H_0, h_c, T_{m-10}, \alpha, \gamma_\beta) = q_1(H_1, h_c, T_{m-10}, \alpha)$$

To find an expression for wave angle factor f, Equation (1) and the first argument of the right-hand side of Equation (2) are combined and rewritten to:

$$H_0 \exp\left(-4.3\frac{\sqrt{2\pi}h_k}{\sqrt{g}T_0}\frac{1}{H_0^{1/2}\gamma_\beta}\frac{1}{\tan\alpha}\right) = fH_0 \exp\left(-4.3\frac{\sqrt{2\pi}h_k}{\sqrt{g}T_0}\frac{1}{f^{1/2}H_0^{1/2}}\frac{1}{\tan\alpha}\right) \tag{5}$$

This can be rewritten as:

$$\exp\left(-\frac{\alpha_0}{H_0^{1/2}\gamma_\beta}\right) = f\exp\left(-\frac{\alpha_0}{f^{1/2}H_0^{1/2}}\right) \tag{6}$$

where:

$$\alpha_0 = -4.3\frac{\sqrt{2\pi}h_c}{\sqrt{g}T_{m,-10}}\frac{1}{\tan\alpha} \tag{7}$$

Further rewriting yields:

$$f = \frac{\exp\left(-\dfrac{\alpha_0}{H_0^{1/2}\gamma_\beta}\right)}{\exp\left(-\dfrac{\alpha_0}{f^{1/2}H_0^{1/2}}\right)} = \exp\left(-\frac{\alpha_0}{H_0^{1/2}\gamma_\beta} + \frac{\alpha_0}{f^{1/2}H_0^{1/2}}\right) \tag{8}$$

$$\ln f = \frac{\alpha_0}{H_0^{1/2}}\left(\frac{1}{f^{1/2}} - \frac{1}{\gamma_\beta}\right) \tag{9}$$

An expression for angle factor f can be found similarly by combining Equation (1) and the second argument of the right-hand side of Equation (2). This results in:

$$\ln f^{3/2} = -2.3\frac{h_c}{H_0}\left(\frac{1}{f} - \frac{1}{\gamma_\beta}\right) \tag{10}$$

Equation (9) and Equation (10) can be solved iteratively to find the angle factor f. Depending on which overtopping rate equation governs according to Equation (2) from Van der Meer, the reduction factor has been established by solving either Equation (9) or Equation (10) and applying this wave angle factor to the wave height for the individual storm.

Figure 14 shows the resulting wave angle factor f for one location (RM 65 at the West Bank) for all 152 storms as a function of the wave direction at peak surge level. The orientation of this levee is north-northeast – south-southwest (246º nautical convention). Thus, waves approaching this levee perpendicular should have an orientation of 66º (nautical convention). As can be observed in **Figure 14**, the wave angle factor f at this wave direction is around 1 (i.e. no reduction in wave height) whereas the wave angle factor reduces to 0.6 – 0.7 for wave angles much less/higher than 66º (nautical convention).

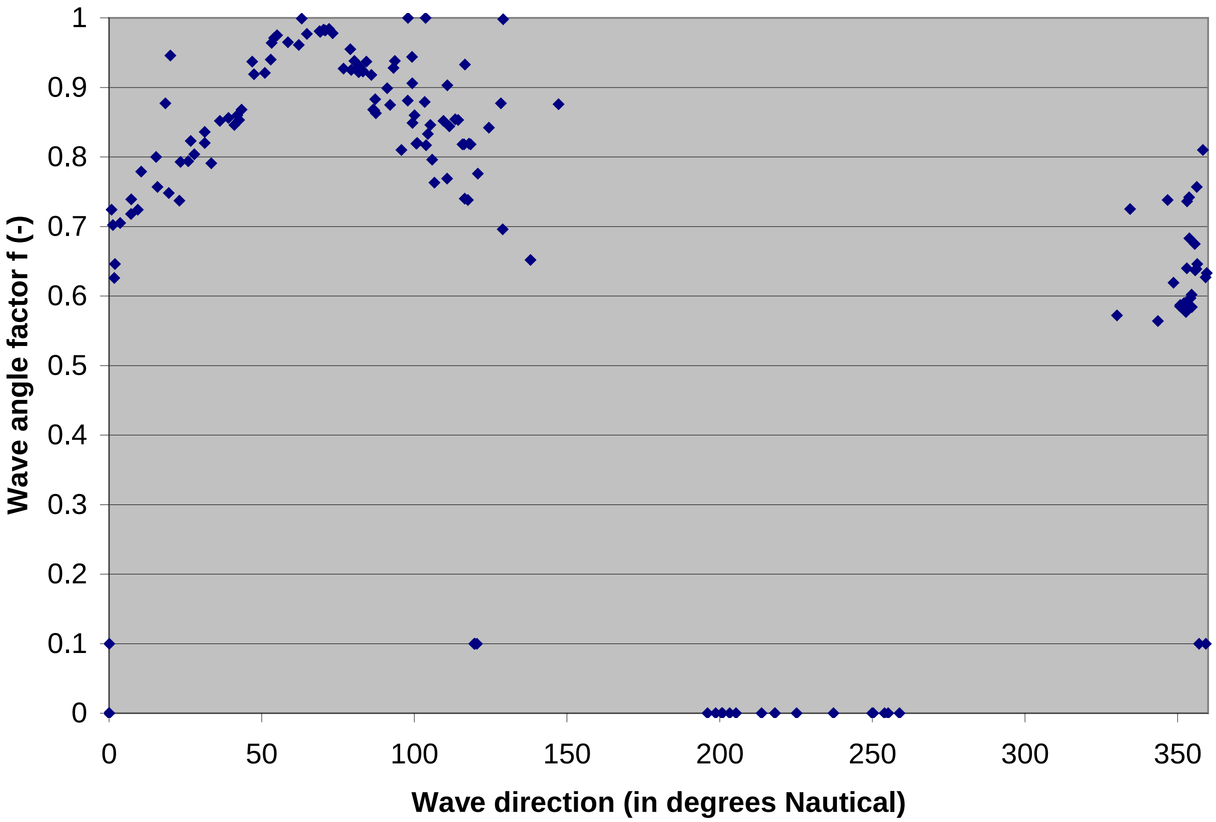


Figure 14 – Wave Angle Factor for River Mile 65 at the West Bank for All 152 Storms

Final significant wave heights were established by applying this reduction factor for all points and for all storms. These reduced wave heights (and the peak wave periods established in **Section 2.6**) are carried forward to establish the wave statistics which is the subject of **Section 3.0**.

2.8 MISSISSIPPI RIVER LEVEE WAVE MODEL VALIDATION

The wave data was quite limited for the Mississippi River to validate the modeling results presented in the preceding sections. Two sources of information were used to do a qualitative assessment of the computed waves. The first source was a video made during hurricane Katrina which is available on the internet. This video is from a tugboat in the Mississippi River. According to the person who posted this video; "The video I took is from the Marion Moran, a 127 ft ocean going tug, pushing a 575 ft hopper barge bound for Puerto Rico. This is at anchor Kenner Bend, LA." A snapshot of this video is depicted in **Figure 15**.

Figure 15 – Snapshot from Video Mississippi River During Hurricane Katrina

The exact time and location of the video is not known. However, the (limited) daylight at the video suggests that the video was made just after dawn when Hurricane Katrina made landfall. Hurricane Katrina made landfall at August 29, 6:00 A.M. (local time) at Buras and the storm crossed the Louisiana/Mississippi border at 9:45 A.M. Although the exact location is also unknown, the video talks about "anchor at Kenner Bend, LA.". This location is a designated spot for anchoring vessels. The anchor location is from RM 114.7 to 115.6 (Kenner Bend Anchorage) and RM 113.5 to 114.3 (Lower Kenner Bend Anchorage). This tugboat is probably located at the East Bank of the Mississippi River and it is looking out on the Mississippi River towards the West Bank. This is inferred from the waves predominantly traveling from left to right in the video which coincides with the predominant wind direction at that time. Looking at the video, the visible waves are estimated at 2.0 ft (significant wave height) with periods of a few seconds based on expert judgment.

The observed waves from the video have been checked against the modeled waves qualitatively. For this purpose, storms 27 and 36 have been selected because they have similar characteristics to Katrina in terms of pressure and size (900 mbar, 21 nm). These storms makes landfall at the Mississippi River downstream of New Orleans. The computed peak surge at RM 113-115 is 15.2 ft for this particular storm with the river discharge of 167,000 cfs. This computed peak surge level is in agreement with the surge observations at the Carrolton gage which peaked around 15 ft. The wave characteristics at the moment of peak surge (averaged for these two storms) are listed in **Table 2**. It can be concluded that the computed waves appear to be qualitatively in line with the observations from the video.

Table 2 – Wave characteristics at East and West Banks at River Mile 113-115

Location	**Waves at Peak Surge – Average of Storms 27/36**		
	Hs (ft)	**Tp (s)**	**Θw (º)**
113 – West Bank	2.9	3.8	7.8
113 – East Bank	0.8	1.6	7.8
114 – West Bank	3.3	3.9	6.7
114 – East Bank	0.8	1.6	6.7
115 – West Bank	3.1	4.1	5.4
115 – East Bank	1.1	1.9	5.4

A second source of information to validate the wave information is observations from the South Louisiana Flood Protection-East (SLFPA-East) along the Mississippi River levee. Debris was found close to and on top of the East Bank levee near RM 91. This debris line is a (rough) prediction of the maximum wave run-up against the levee. The levee elevation was around 20 ft. The peak storm surge at this location is estimated around 16 ft based on the ADCIRC results. The wave parameters from storm 36 are shown in **Figure 16** and used herein as representative during Katrina.

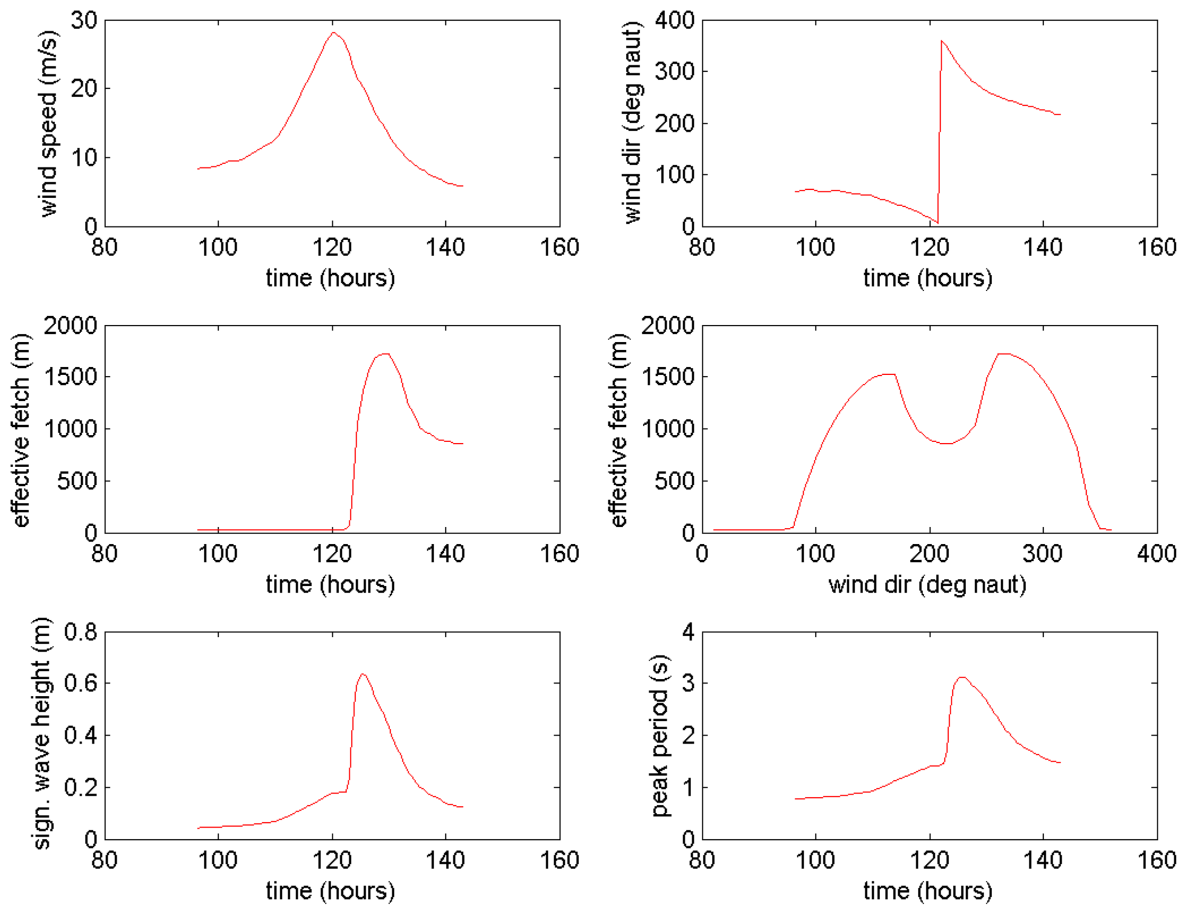


Figure 16 – Wind Speed, Wind Direction, Effective Fetch, Significant Wave Height, and Period East Bank River Mile 91 Storm 36 near Levee

Based on the surge and wave characteristics, the 2% wave run-up on top of the surge is estimated to assess the debris location; see TAW 2002 for a definition of the 2% run-up. **Table 3** lists for several moments during the storm the surge, wave characteristics, and run-up. At the peak of the storm surge the waves were relatively small because of the wind direction at that moment. However, the wave height substantially increased directly after the peak of the storm because of the change in wind direction and more perpendicular waves towards this levee. As can be observed, the 2% run-up on top of the surge, 4 hours after the peak surge level is very close to the levee crest of 20 ft.

Table 3 – Surge, Wave Characteristics, Wave Run-up – East Bank River Mile 90 Storm 36

Time	Waves at Peak Surge – Storm 36				
	Surge (ft)	**Hs (ft)**	**Tp (s)**	**2% Run-up (ft)**	**Surge + Run-up (ft)**
Peak of Storm	16	0.7	1.5	1.6	17.6
Peak of Storm +2 hr	15	1.5	2.5	3.8	18.8
Peak of Storm +4 hr	13	2.5	3.5	6.9	19.9
Peak of Storm +6 hr	12	2.0	3.0	5.3	17.3

Although field data from waves at the Mississippi River is absent, the two examples above show that the computed wave characteristics are qualitatively in line with the observations during Hurricane Katrina. It is therefore concluded that the methodology provides a good basis for the determination of the 1% design wave statistics and levee design elevations.

3.0 STEP 2: STATISTICAL DISTRIBUTION OF WAVE PARAMETERS

In order to determine the 1% wave height, the modeled wave heights for each point are related to the probability of occurrence of the storms. Subsequently, the wave heights are classifieds into bins of 0.5 ft to determine the probability of exceedence of each wave height bin. In this analysis 20 bins were used, the first bin contains waves up to 0.5 ft and the last bin brackets waves between 4.0 and 4.5 ft. Subsequently the data points are fitted with a Weibull distribution function this allows interpolation of the wave height for a return period of 100 years.

The following example illustrates the statistical approach for a point on the East Bank of the Mississippi River at RM 65. 152 wave heights have been computed for the 152 synthetic storms. **Figure 17** shows the 152 wave results at RM 65 on the East Bank. For each storm a probability of occurrence has been determined. The wave height is related to the probability of occurrence of the storm, in doing so the probability of exceedence of a wave height bin can be determined. For example, for this point a large number of wave heights fall in the bin 1.5 – 2.0 ft. The sum of the probability of occurrence of the storms (coupled to these waves) contribute to the probability of occurrence (or frequency) of this bin.

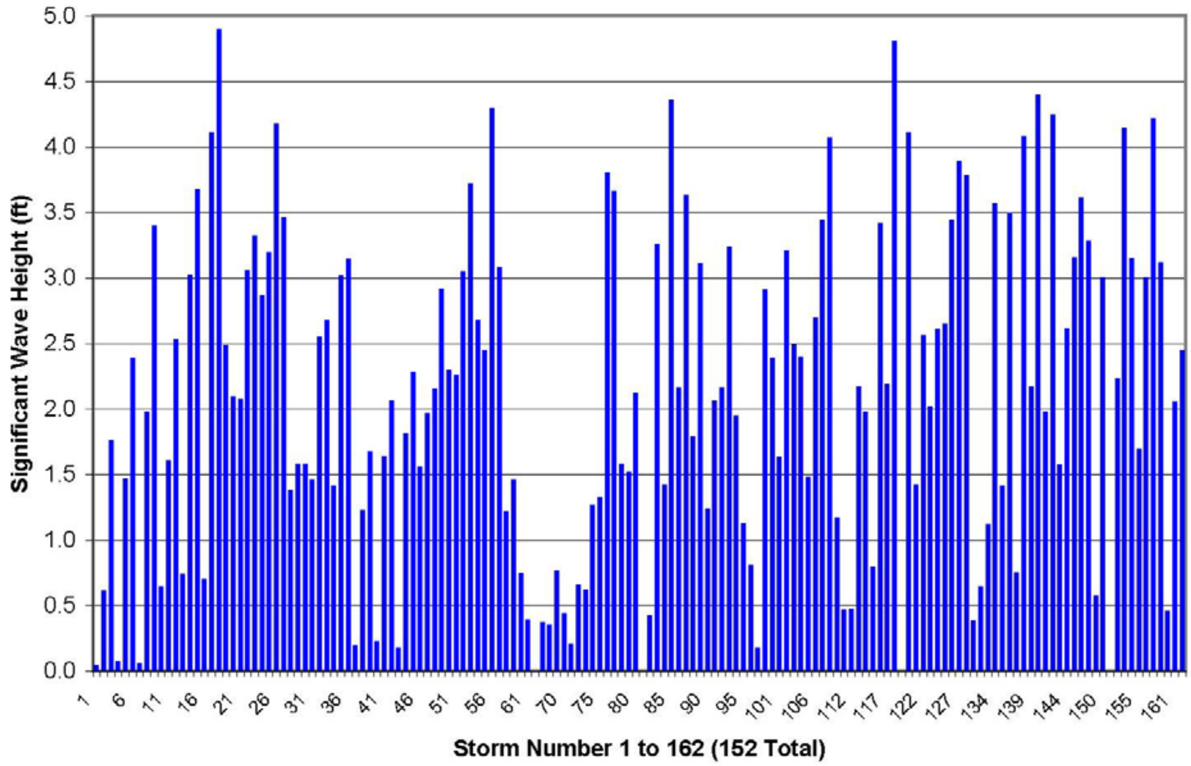


Figure 17 – Significant Wave Height for 152 Storms at River Mile 65 East Bank

Subsequently, the probability of exceedence for each wave height bin is calculated **(Table 4)**. **Figure 18** shows the relation between the wave height bin and the frequency of that bin based upon the probability of exceedence. The data points are fitted with a Weibull function in order to interpolate the 100 year (or 1% exceedence event) wave height.

Table 4 – Tabulated Return Periods for Wave Height Bins at River Mile 65 on East Bank

Significant Wave Height Bin (ft)	Average Significant Wave Height (ft)	Frequency (1/yr)	Average Return Period (year)
0.0 – 0.5	0.25	0.065637	15.2
0.5 – 1.0	0.75	0.046433	21.5
1.0 – 1.5	1.25	0.025616	39.0
1.5 – 2.0	1.75	0.006829	146.4
2.0 – 2.5	2.25	0.003882	257.6
2.5 – 3.0	2.75	0.00368	271.7
3.0 – 3.5	3.25	0.000548	1825.7
3.5 – 4.0	3.75	0.000548	1825.7
4.0 – 4.5	4.25	0.000201	4965.32
4.5 – 5.0	4.75	0.000201	∞

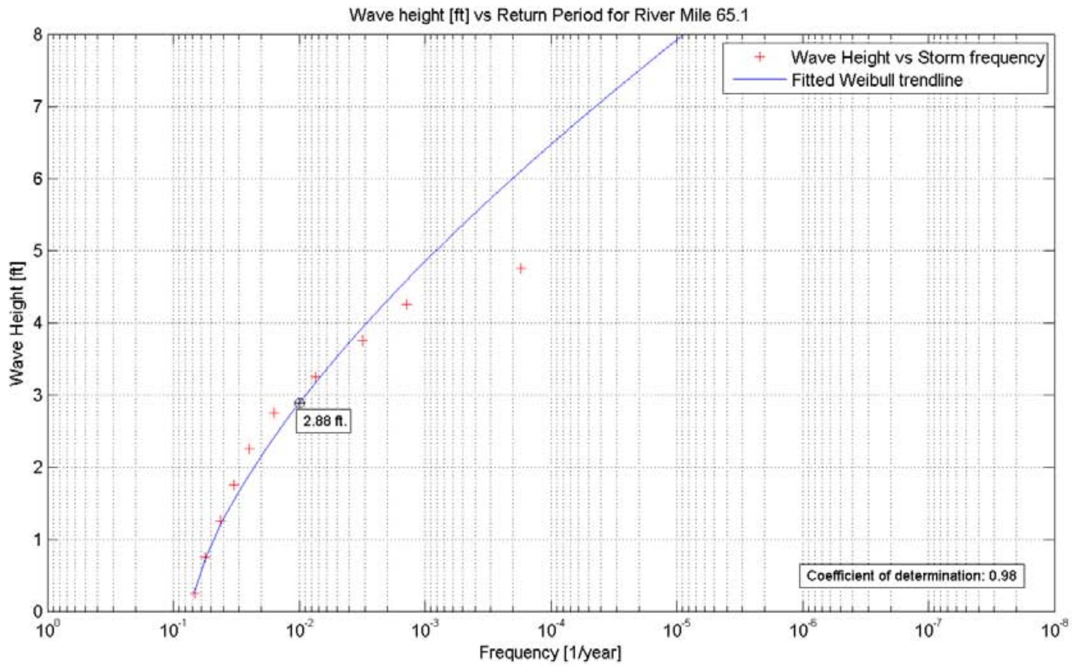


Figure 18 – Wave Height vs. Frequency and Weibull Fitted Distribution Curve at River Mile 65 on West Bank

From the fitted Weibull function, wave heights for the 50, 100, and 500 year etc. return period can be determined **(Table 5)**. For this particular point on the East Bank of the Mississippi River at RM 65 the 1% exceedence wave height is estimated at 1.87 ft.

Table 5 – Tabulated Wave Heights for Return Periods at River Mile 65 on the West Bank

Frequency (1/yr)	Average Return Period (year)	Significant Wave Height (ft) Derived from Fitted Weibull Distribution
0.02	50	1.38
0.01	100	2.88
0.002	500	3.01
0.001	1000	3.22
0.0001	10000	4.36

The analysis, as described above, has been applied for all river miles for both West and East banks

Standard deviations of the wave parameters, which represent uncertainty in the values, were established using the methodology of the original HSDRRS guidelines. The standard deviation of the 1% significant wave height was set to 10% of the 1% significant wave height value. The standard deviation of the 1% peak wave period was set to 20% of the 1% peak wave period value.

4.0 1% EXISTING CONDITION WAVE CHARACTERISTICS

The 1% existing condition wave values, developed using the statistics process described in **Section 3.0**, are provided in **Table 6**. Given the high variability of 1% wave parameters, recommended values were developed for different sections of the river. The recommended wave values are based on trends seen in the 1% wave results and enforcement of minimum design values. The minimum design significant wave height was selected to be 1.5 ft. This value is used as a minimum for coastal structures. The corresponding, minimum wave period was selected to be 2.5 sec. Although lower values are presented in this write-up, the minimum and recommended wave values are carried forward in design of structures. **Figure 19** through **22** show the existing condition modeled and recommended 1% wave values for the East and West Banks.

Table 6 – Existing Condition 1% Exceedence Significant Wave Heights and Peak Wave Periods for East and West Banks

River Mile	1% Existing Significant Wave Height Hs (ft)		1% Existing Peak Wave Period Tp (sec)	
	West Bank	East Bank	West Bank	East Bank
65	2.88	0.14	4.07	0.00
66	2.83	0.00	3.96	0.00
67	2.58	0.00	3.95	0.00
68	2.59	0.00	4.11	0.00
69	3.39	0.66	4.61	2.59
70	2.79	1.16	4.08	3.23
71	2.21	1.25	4.01	3.18
72	2.27	1.17	3.88	3.20
73	2.26	0.80	3.88	2.66
74	2.39	1.33	4.10	3.32
75	2.13	1.09	3.78	3.14
76	1.96	0.91	3.70	2.96
77	2.15	0.74	3.78	2.59
78	1.14	1.07	3.54	2.84
79	1.05	0.00	2.79	0.00
80	0.82	0.13	2.34	0.00
81	0.86	0.57	2.34	2.39
82	1.45	0.89	3.85	2.88
83	2.05	0.17	3.75	1.27
84	2.49	0.00	4.11	0.00
85	2.02	0.00	3.62	0.00
86	1.48	0.54	3.39	2.42
87	1.73	1.15	3.41	3.24
88	0.72	0.33	2.46	2.53
89	1.87	0.36	3.70	2.68
90	2.00	0.10	3.90	0.00
91	1.69	0.00	3.67	0.00
92	1.68	0.81	3.55	3.04
93	1.51	0.00	3.57	0.00
94	0.30	0.31	1.40	1.55

River Mile	1% Existing Significant Wave Height Hs (ft)		1% Existing Peak Wave Period Tp (sec)	
	West Bank	East Bank	West Bank	East Bank
95	0.81	0.00	2.40	0.00
96	0.72	0.00	2.28	0.00
97	0.99	0.30	2.51	2.06
98	0.49	0.34	1.78	1.74
99	0.60	0.85	1.99	2.74
100	0.43	0.00	1.93	0.00
101	1.44	0.00	3.27	0.00
102	0.69	0.00	3.41	0.00
103	1.44	0.24	3.40	1.65
104	1.49	0.20	3.24	0.00
105	0.42	0.60	1.84	1.97
106	0.63	0.52	2.08	2.12
107	0.18	0.00	1.85	0.00
108	0.00	0.31	0.00	1.55
109	1.74	0.28	3.46	1.58
110	1.66	0.52	3.38	2.06
111	0.96	0.04	3.24	0.87
112	0.70	0.00	3.55	0.00
113	1.42	0.46	3.77	2.89
114	1.68	0.20	3.59	2.22
115	0.53	0.25	2.29	1.70
116	0.53	0.32	2.21	1.62
117	0.53	0.84	2.30	2.80
118	0.48	0.62	1.98	2.15
119	0.22	0.46	1.46	1.53
120	1.23	0.00	3.21	0.00
121	1.01	0.22	3.41	1.97
122	0.99	0.00	3.80	0.00
123	1.17	0.43	3.37	1.92
124	0.66	0.32	3.79	2.85
125	0.00	0.00	0.00	0.00
126	0.85	0.31	3.11	1.91
127	0.62	0.90	3.41	3.77
128	0.62	1.58	3.61	3.91
129	0.74	0.00	2.98	0.00
130	0.45	0.00	3.25	0.00

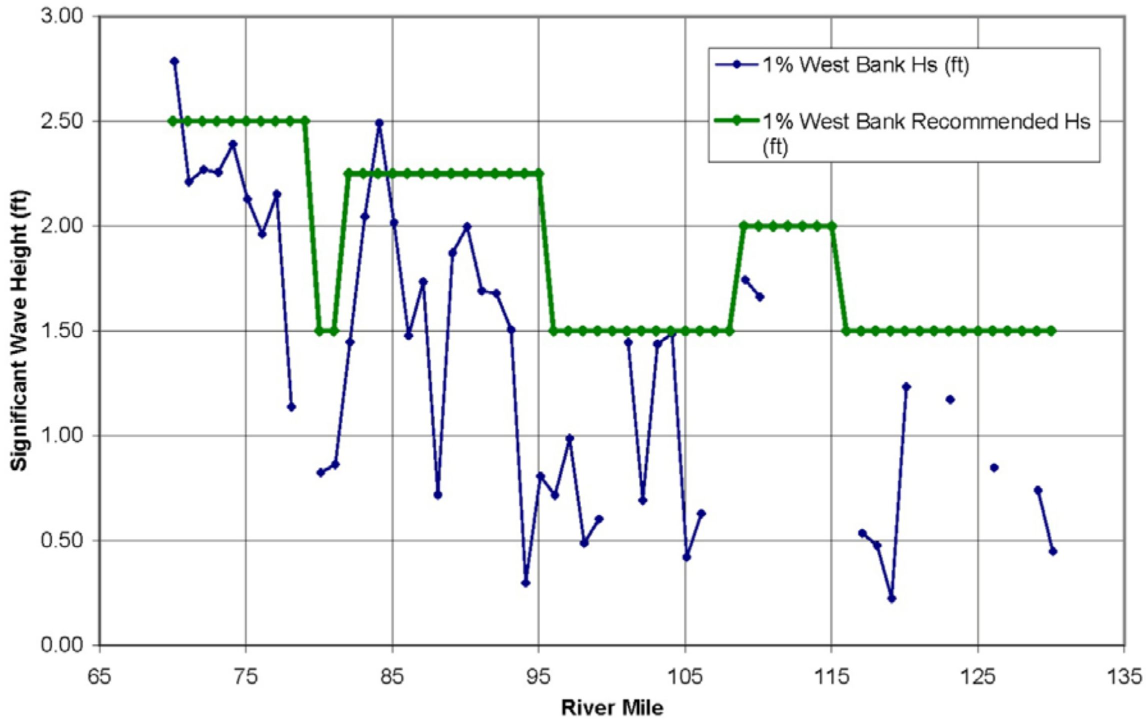


Figure 19 – Existing Condition – Modeled and Recommended 1% Significant Wave Height at West Bank

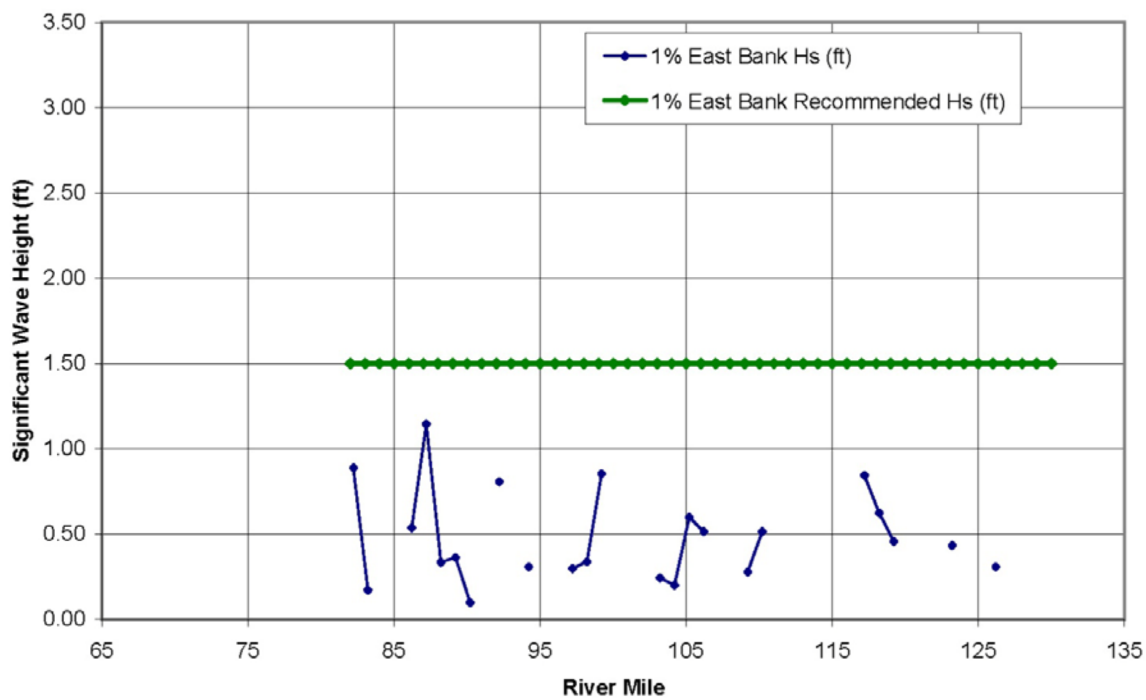


Figure 20 – Existing Condition – Modeled and Recommended 1% Significant Wave Height at East Bank

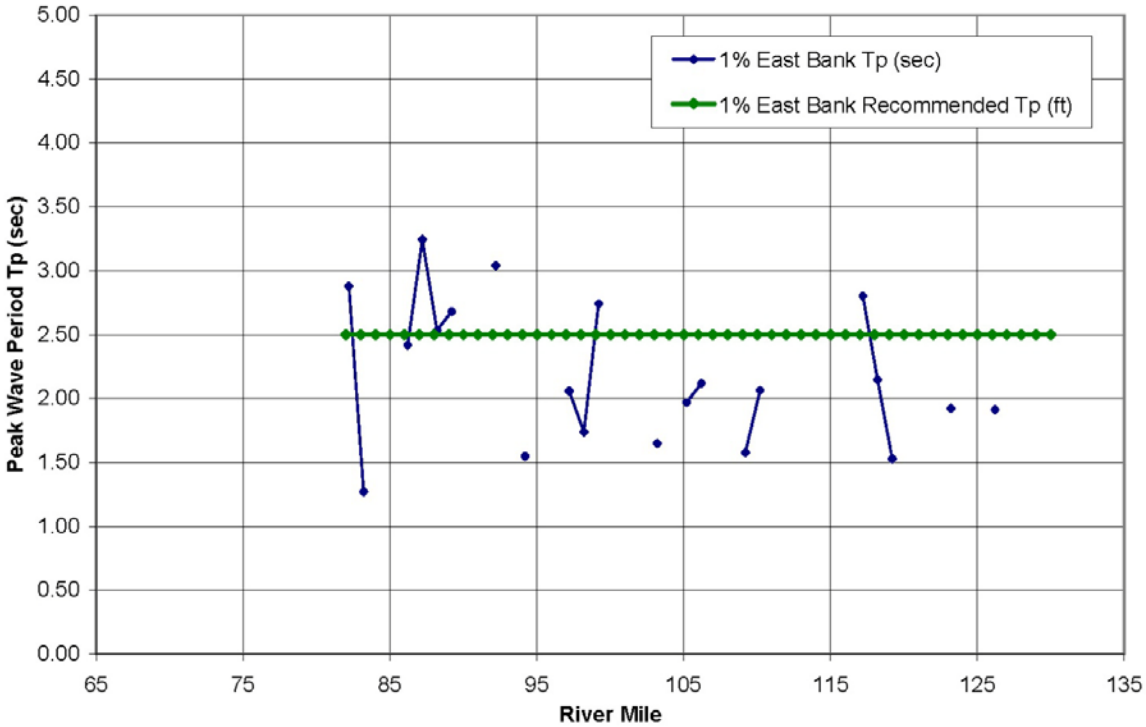


Figure 21 – Existing Condition – Modeled and Recommended 1% Peak Wave Period at East Bank

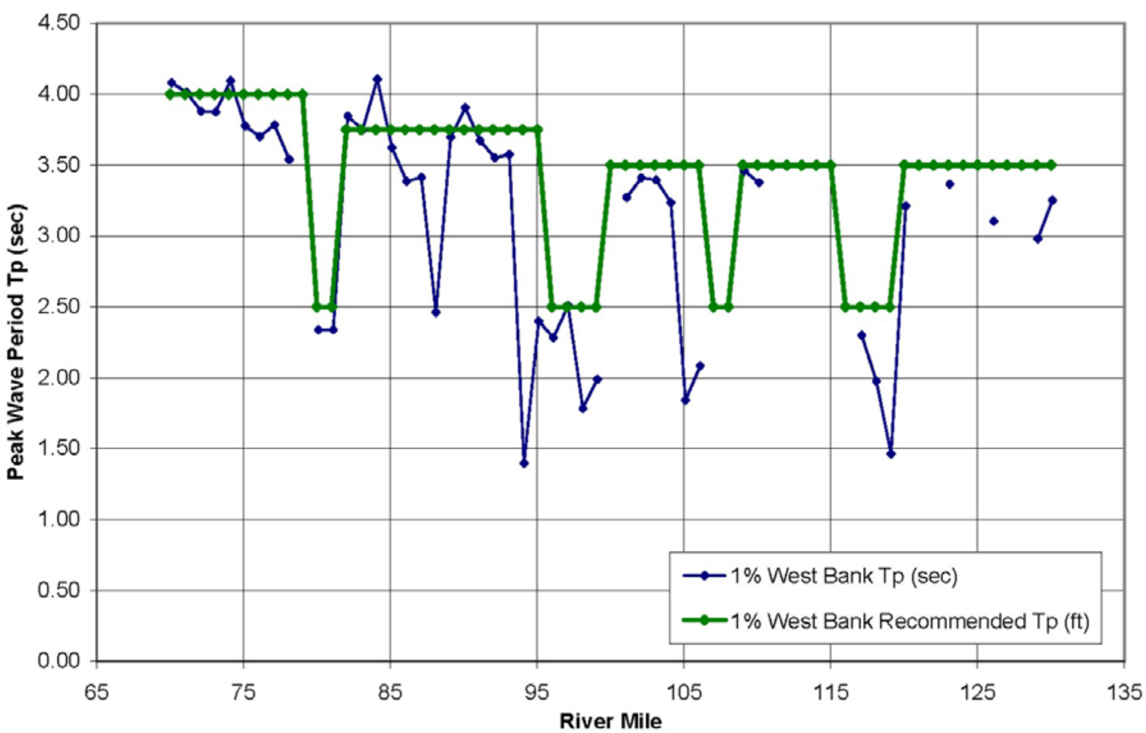


Figure 22 – Existing Condition – Modeled and Recommended 1% Peak Wave Period at West Bank

5.0 1% FUTURE CONDITION WAVE CHARACTERISTICS

Future wave characteristics were developed by modifying the model input to account for surge level increase due to relative sea level rise (RSLR). It was assumed that wind-speed, wind-direction, fetch length, batture width, and batture elevation would not change for future conditions. The only model input adjusted for future conditions was surge elevation. Higher surge creates a larger depth over the batture and thus larger incoming waves can propagate unobstructed to the levee.

Future surge hydrographs were developed by adjusting the existing condition hydrographs to account for 1.0 ft of RSLR. A RSLR value of 1.0 ft was used in the design of the HSDRRS system. The latest ADCIRC runs conducted for the MRL project included several scenarios with RSLR. Scenario 3 included 1.0 ft of RSLR with a river discharge of 400,000 cfs. Scenario 1 included no RSLR with a river discharge of 400,000 cfs. 17 storms were evaluated for each MRL scenario. A full set of 152 peak surge values were developed for each scenario by regressing the peak surge values from the 17 MRL storms against the same 17 storms from the 2007 set. The regression process is discussed in greater detail in the overall MRL report. The MRL condition 1 peak surge matrix was subtracted from the MRL condition 3 matrix to determine the increase of peak surge due to 1.0 ft of RSLR. A unique increase was calculated for all 152 storms from RM 1 to 130. The following visualization in **Figure 23** shows an example of how the surge increase was added to the existing condition hydrograph to determine the future condition hydrograph:

MRL Scenario 3 Matrix - Peak Surge (ft. NAVD88 2004.65)				
	Storm 001	Storm 002	...	Storm 162
River Mile 1.0	9.1	11.3	...	5.1
River Mile 2.0	9.7	11.7	...	5.2
...	...	...	...	...
River Mile 130	8.2	11.7	...	5.2

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MRL Scenario 1 Matrix - Peak Surge (ft. NAVD88 2004.65)				
	Storm 001	Storm 002	...	Storm 162
River Mile 1.0	8	10.1	...	4
River Mile 2.0	8.6	10.5	...	4.3
...	...	...	...	...
River Mile 130	7	10.4	...	4.2

=

Increase in Peak Surge due to Sea Level Rise of 1.0 ft				
	Storm 001	Storm 002	...	Storm 162
River Mile 1.0	1.1	1.2	...	1.1
River Mile 2.0	1.1	1.2	...	0.9
...	...	...	...	...
River Mile 130	1.2	1.3	...	1

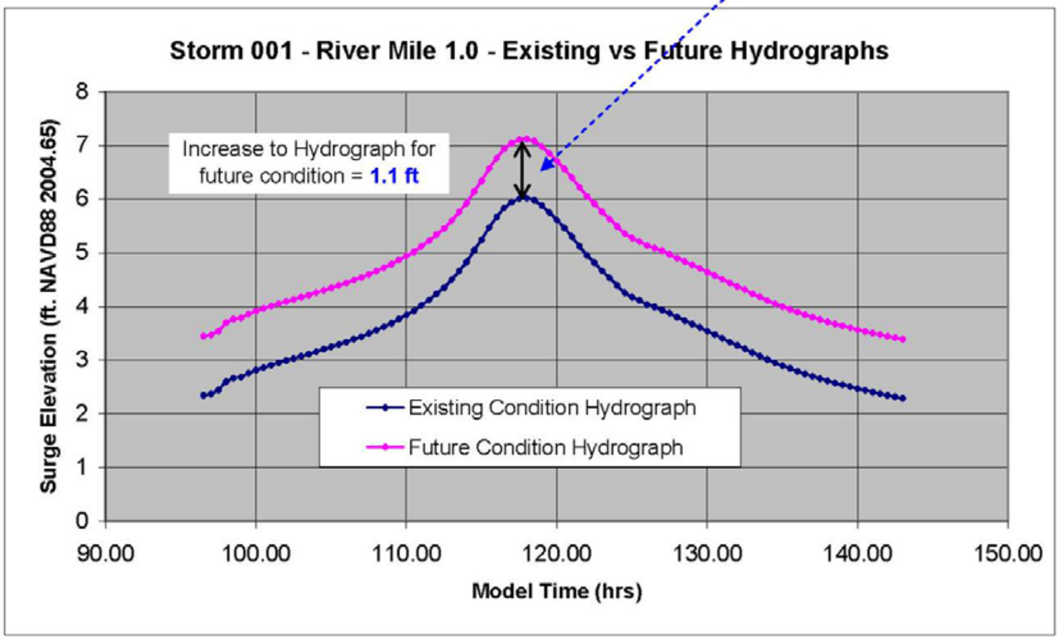

Figure 23 – Visualization of Future Condition Hydrograph Creation

(Note: values are for example purpose only and do not represent actual model input)

The model was re-run for future conditions with the adjusted hydrographs. Statistics were re-computed using the methodology described in **Section 3.0**. **Table 7** presents the Future 1% wave characteristics. Given the high variability of 1% wave parameters, recommended values were developed. The recommended wave values are based on trends seen in the 1% wave results and enforcement of minimum design values. The minimum design significant wave height was selected to be 1.5 ft and the corresponding minimum peak wave period was selected to be 2.5 sec. **Figures 24** through **27** shows the future condition modeled and recommended 1% wave values for the East and West Banks.

Table 7 – Future Condition 1% Exceedence Significant Wave Heights and Peak Wave Periods for East and West Banks

River Mile	1% Future Significant Wave Height Hs (ft)		1% Future Peak Wave Period Tp (sec)	
	West Bank	East Bank	West Bank	East Bank
65	3.59	0.15	4.15	0.00
66	3.40	0.00	4.08	0.00
67	3.26	0.00	4.09	0.00
68	3.32	0.00	4.05	0.00
69	4.67	0.79	4.86	2.52
70	3.19	1.47	4.11	3.36
71	2.90	1.50	3.81	3.23
72	2.61	1.37	3.94	3.19
73	3.34	0.93	4.12	2.65
74	3.14	1.60	4.21	3.21
75	2.57	1.44	3.81	3.14
76	2.72	1.16	3.79	2.99
77	3.01	0.99	3.98	2.89
78	2.10	1.42	3.85	2.94
79	2.07	0.29	3.17	2.07
80	1.43	0.38	2.83	1.25
81	1.44	1.02	2.68	2.60
82	2.58	0.99	4.24	2.93
83	2.90	0.31	4.05	1.90
84	3.10	0.00	4.00	0.00
85	2.85	0.62	3.85	2.60
86	2.26	1.00	3.56	2.60
87	2.45	2.06	3.88	3.61
88	1.14	0.90	2.67	3.37
89	2.86	1.05	4.20	3.27
90	2.89	0.68	4.16	2.49
91	2.44	0.00	3.91	0.00
92	2.51	1.33	3.75	3.11
93	2.25	0.00	3.71	0.00
94	0.67	0.61	2.43	2.17
95	1.23	0.00	2.71	0.00
96	1.02	0.00	2.44	0.00
97	1.57	0.78	2.86	2.47
98	0.78	0.94	2.23	2.42
99	0.87	1.21	2.37	2.49
100	0.80	0.28	2.86	2.16
101	2.01	0.00	3.83	0.00
102	1.39	0.11	3.61	1.23

River Mile	1% Future Significant Wave Height Hs (ft)		1% Future Peak Wave Period Tp (sec)	
	West Bank	East Bank	West Bank	East Bank
103	2.16	0.59	3.75	2.13
104	1.97	0.52	3.43	1.48
105	0.63	0.83	2.09	2.07
106	0.99	0.89	2.48	2.60
107	0.63	0.00	2.61	0.00
108	0.29	0.92	1.30	3.06
109	2.39	0.55	3.90	2.23
110	2.33	0.70	3.90	2.22
111	1.59	0.71	3.95	2.77
112	1.33	0.00	4.15	0.00
113	2.08	0.95	4.15	3.94
114	2.41	0.71	3.86	3.26
115	0.96	0.61	2.70	2.10
116	0.83	0.62	2.48	2.65
117	1.00	1.23	2.51	2.72
118	0.73	0.85	2.42	2.20
119	0.67	0.72	2.51	2.10
120	1.69	0.42	3.45	2.52
121	1.38	0.74	3.80	2.98
122	1.31	0.39	4.38	2.45
123	1.45	0.85	3.82	2.71
124	0.99	0.67	4.27	3.16
125	0.24	0.14	2.44	0.00
126	1.23	0.78	3.33	3.20
127	1.13	1.56	3.66	3.80
128	1.17	2.04	3.90	4.24

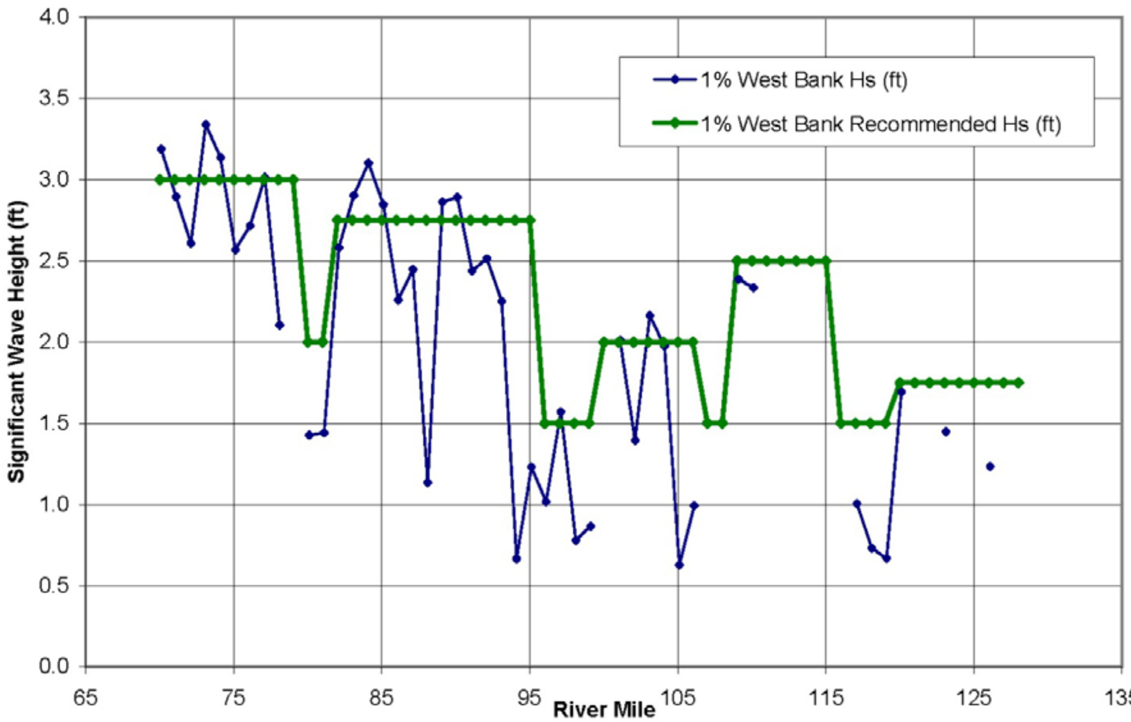

Figure 24 – Future Condition - Modeled and Recommended 1% Significant Wave Height at West Bank

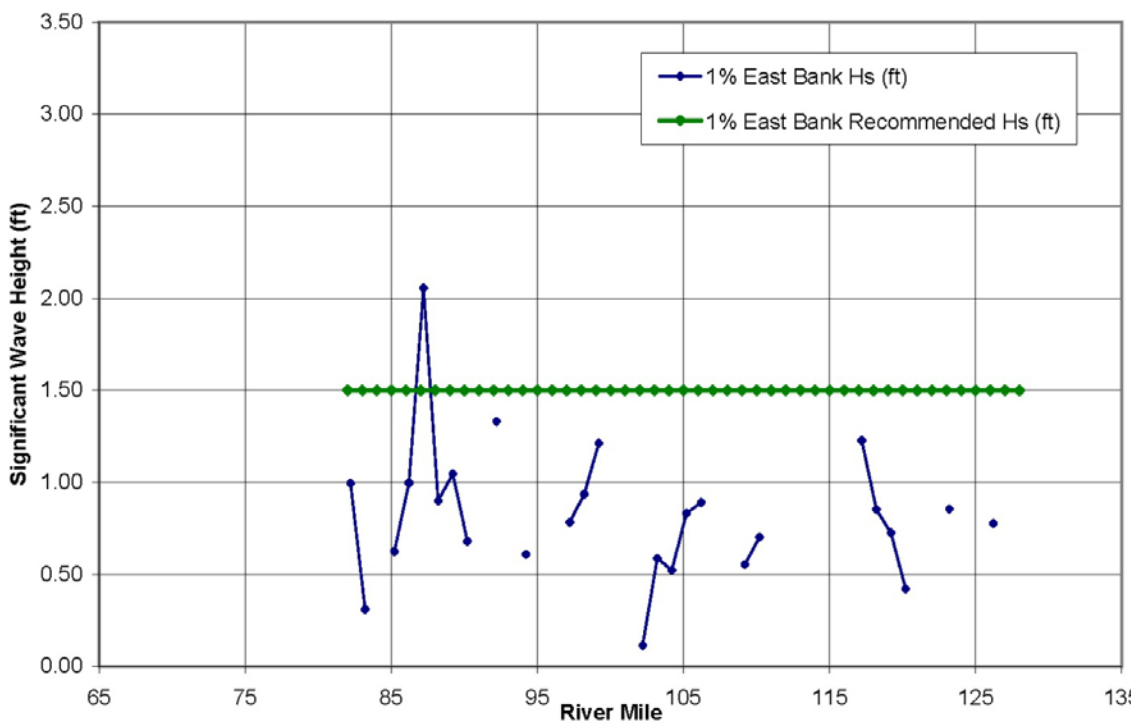

Figure 25 – Future Condition - Modeled and Recommended 1% Significant Wave Height at East Bank

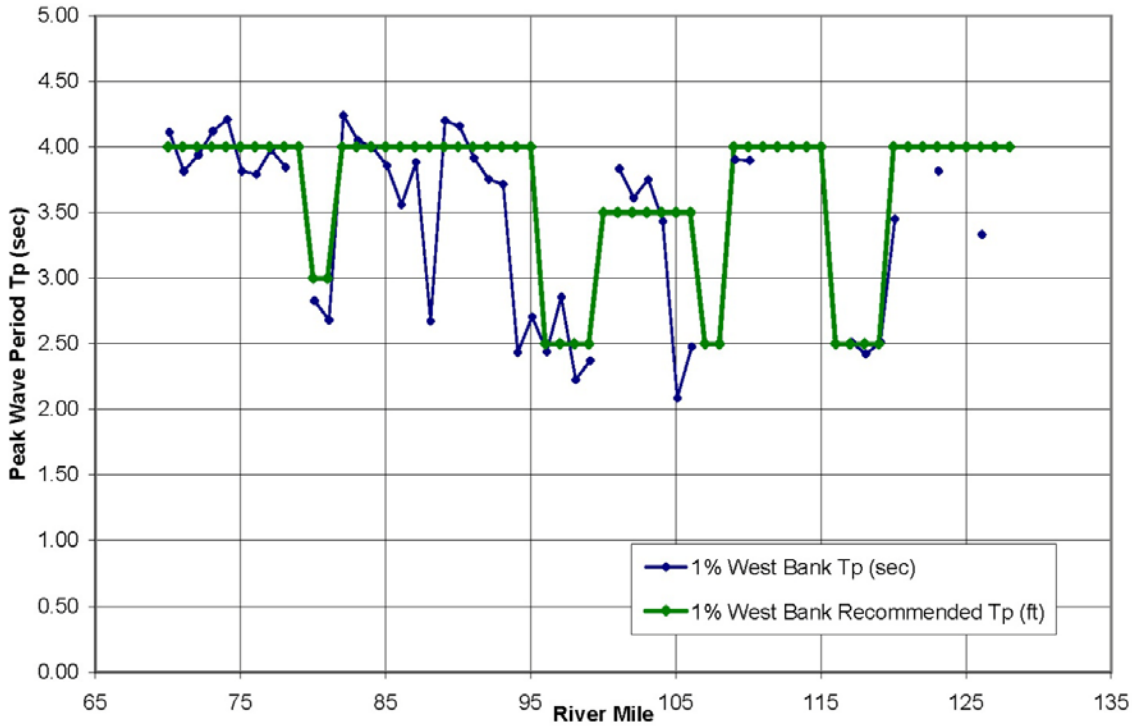

Figure 26 – Future Condition - Modeled and Recommended 1% Peak Wave Period at West Bank

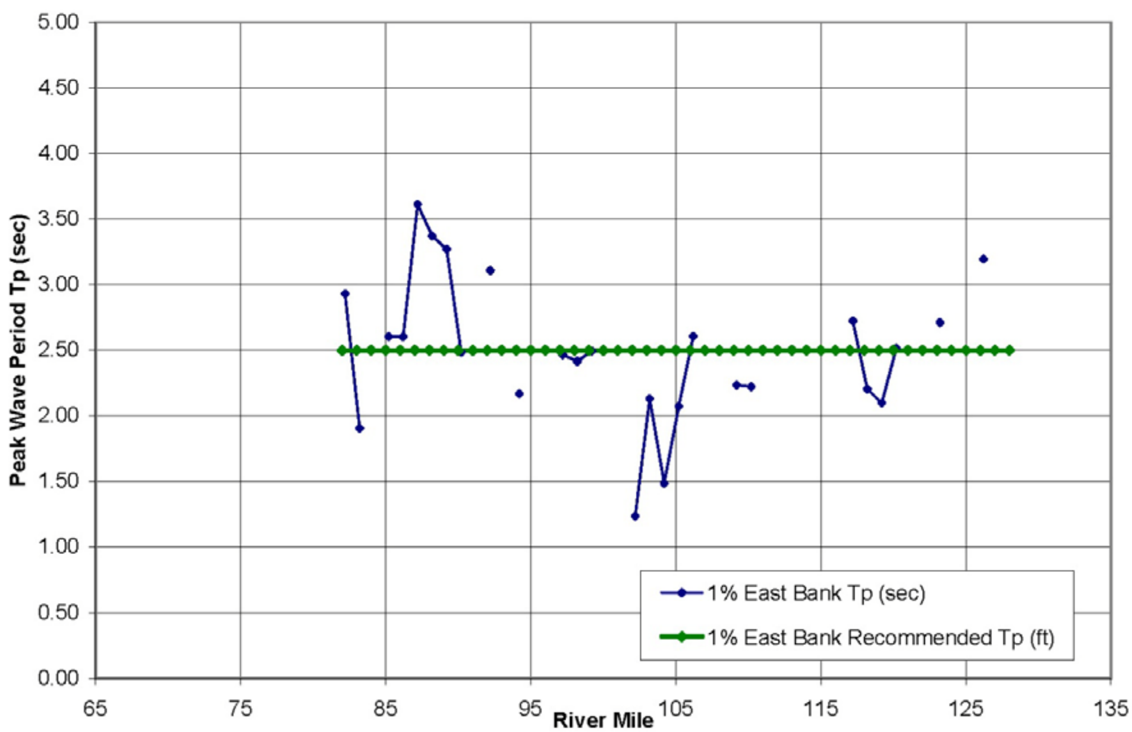

Figure 27 – Future Condition - Modeled and Recommended 1% Peak Wave Period at East Bank

6.0 SUMMARY OF RECOMMENDED 1% WAVE CHARACTERISTICS

Tables 8 and **9** summarize the recommended 1% existing and future wave values for the co-located MRL-HSDRRS system. **Figure 28** through **31** show the recommended 1% wave characteristics for existing and future conditions in relation to the HSDRRS system.

Table 8 – Summary of Recommended 1% Existing and Future Wave Values – West Bank

RM Start	RM End	Existing Conditions		Future Conditions	
		Significant Wave Height (ft)	Peak Period (s)	Significant Wave Height (ft)	Peak Period (s)
70	79	2.5	4.0	3.0	4.0
80	81	1.5	2.5	2.0	3.0
82	95	2.25	3.75	2.75	4.0
96	99	1.5	2.5	1.5	2.5
100	106	1.5	3.5	2.0	3.5
107	108	1.5	2.5	1.5	2.5
109	115	2.0	3.5	2.5	4.0
116	119	1.5	2.5	1.5	2.5
120	130	1.5	3.5	1.75	4.0

Table 9 – Summary of Recommended 1% Existing and Future Wave Values – East Bank

RM Start	RM End	Existing Conditions		Future Conditions	
		Significant Wave Height (ft)	Peak Period (s)	Significant Wave Height (ft)	Peak Period (s)
82	130	1.5	2.5	1.5	2.5

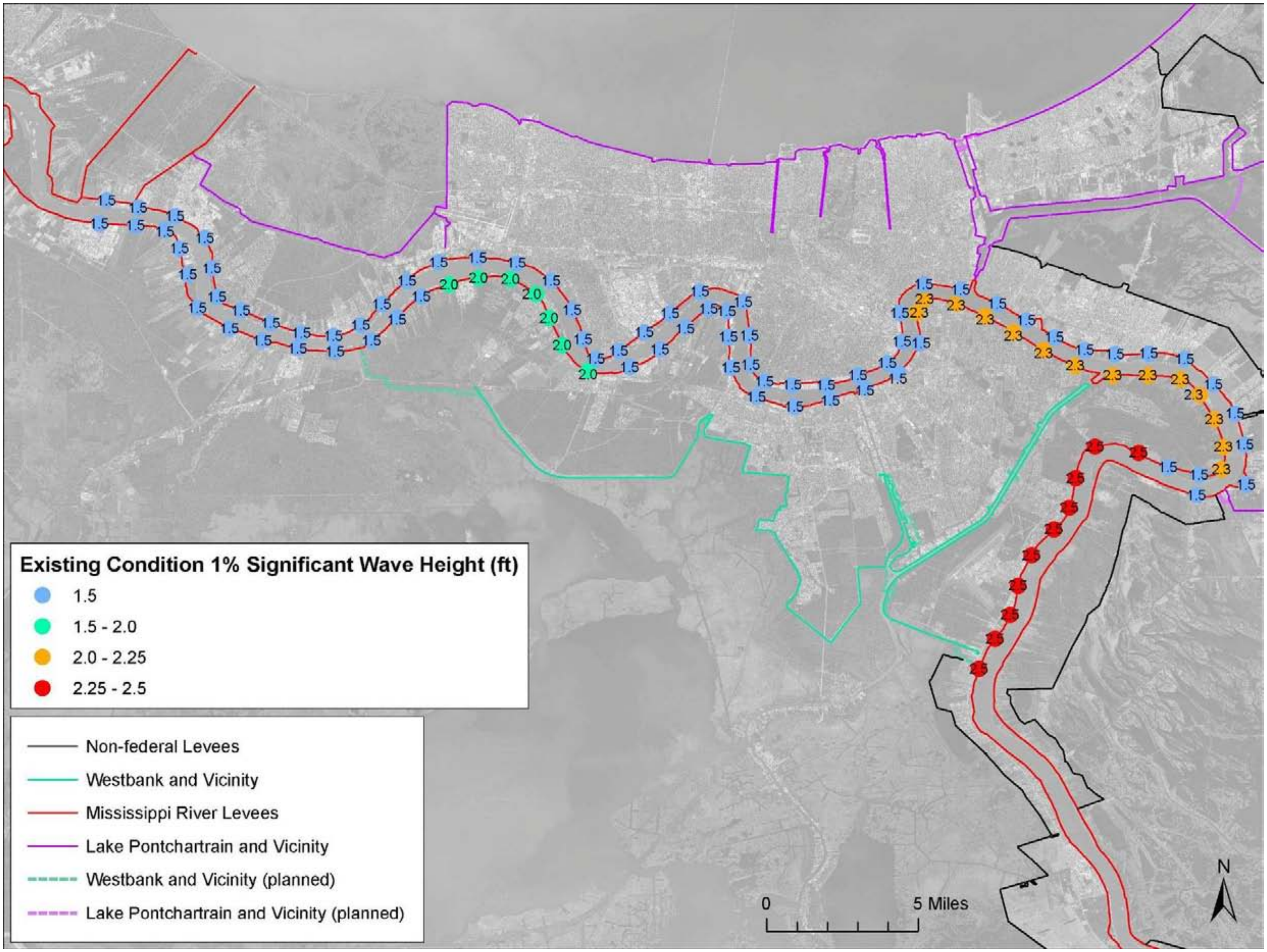

Figure 28 – Existing Condition - Recommended 1% Significant Wave Height 2D Plot

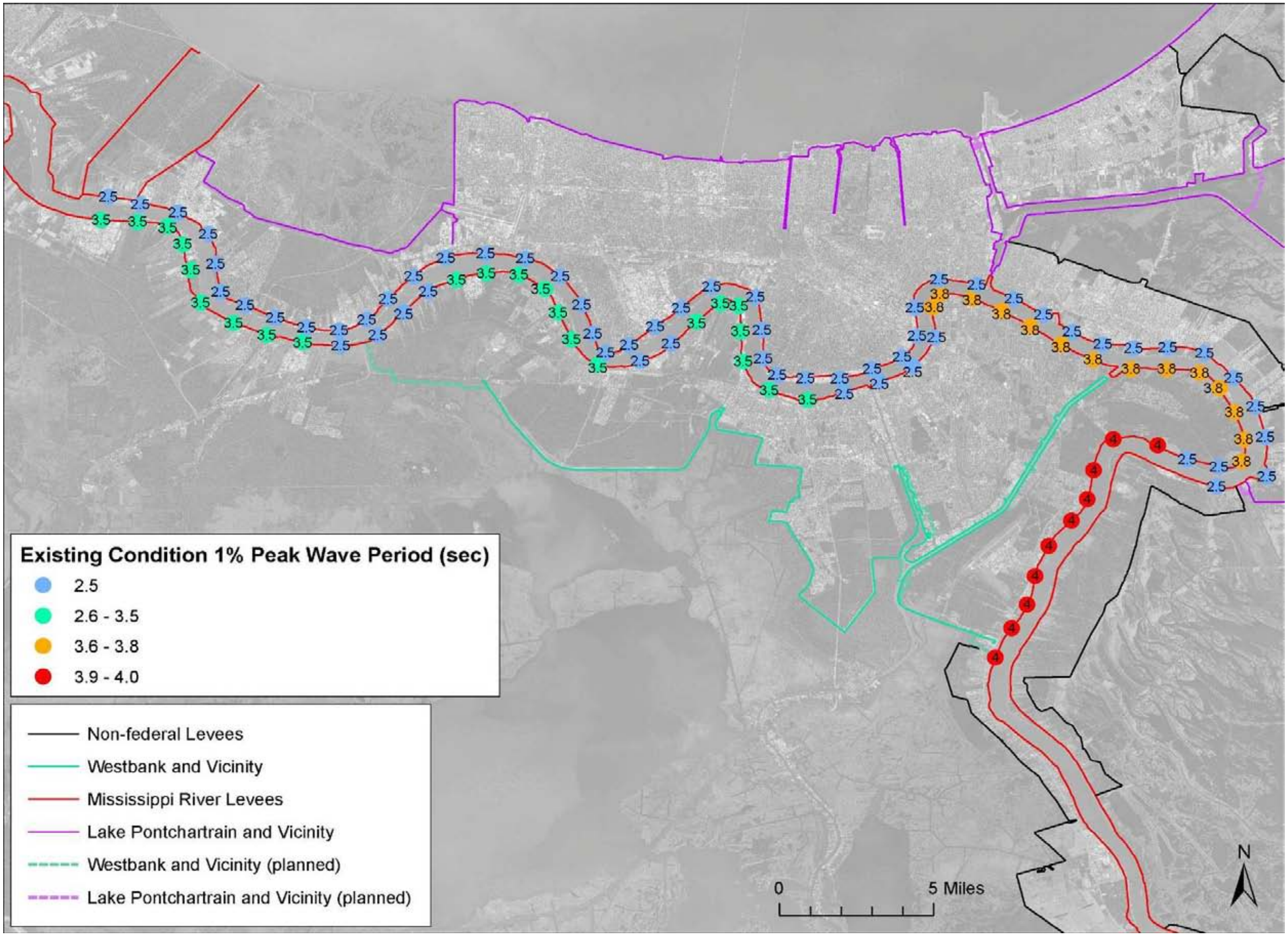

Figure 29 – Existing Condition - Recommended 1% Peak Wave Period 2D Plot

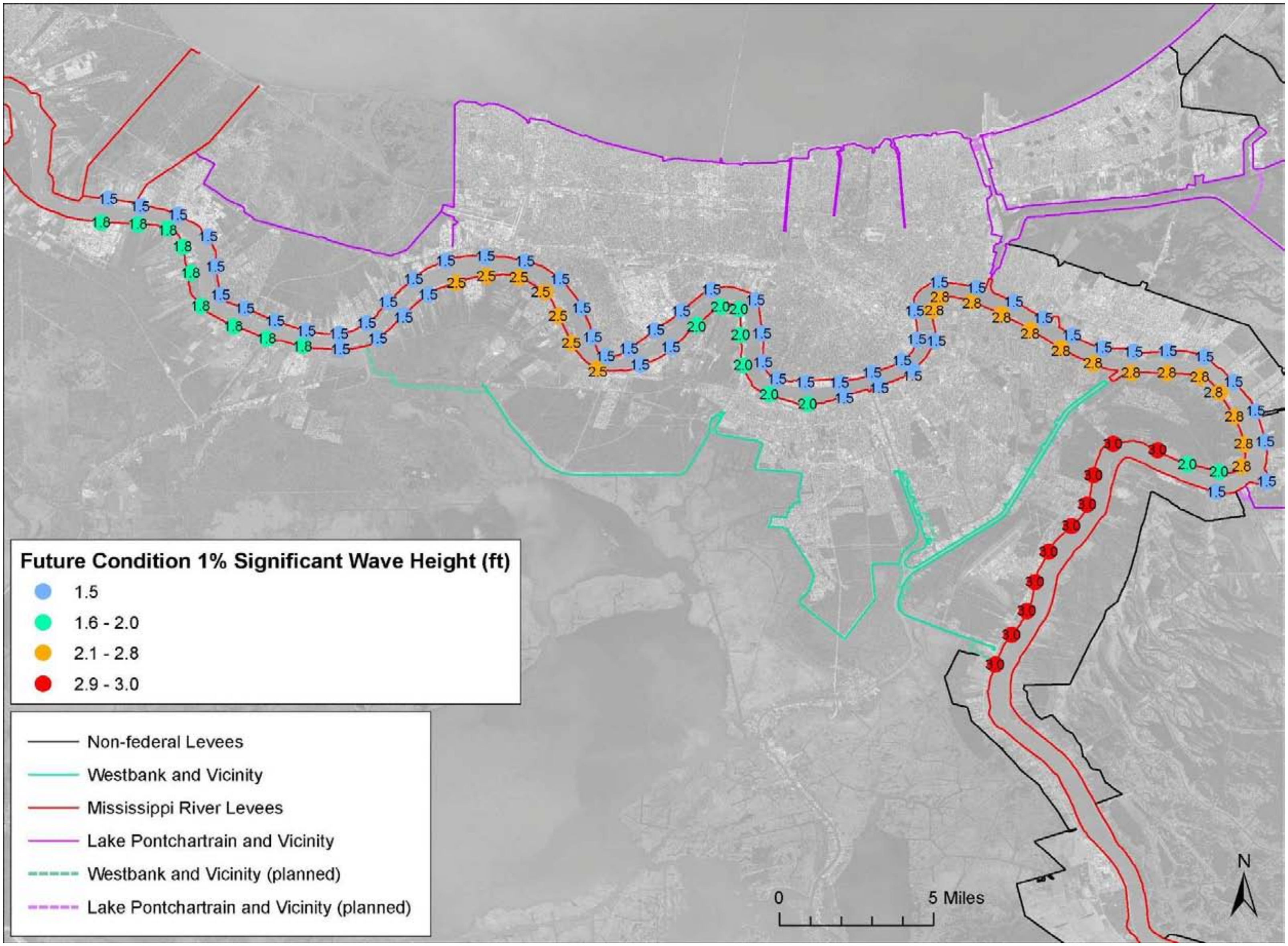

Figure 30 – Future Condition - Recommended 1% Significant Wave Height 2D Plot

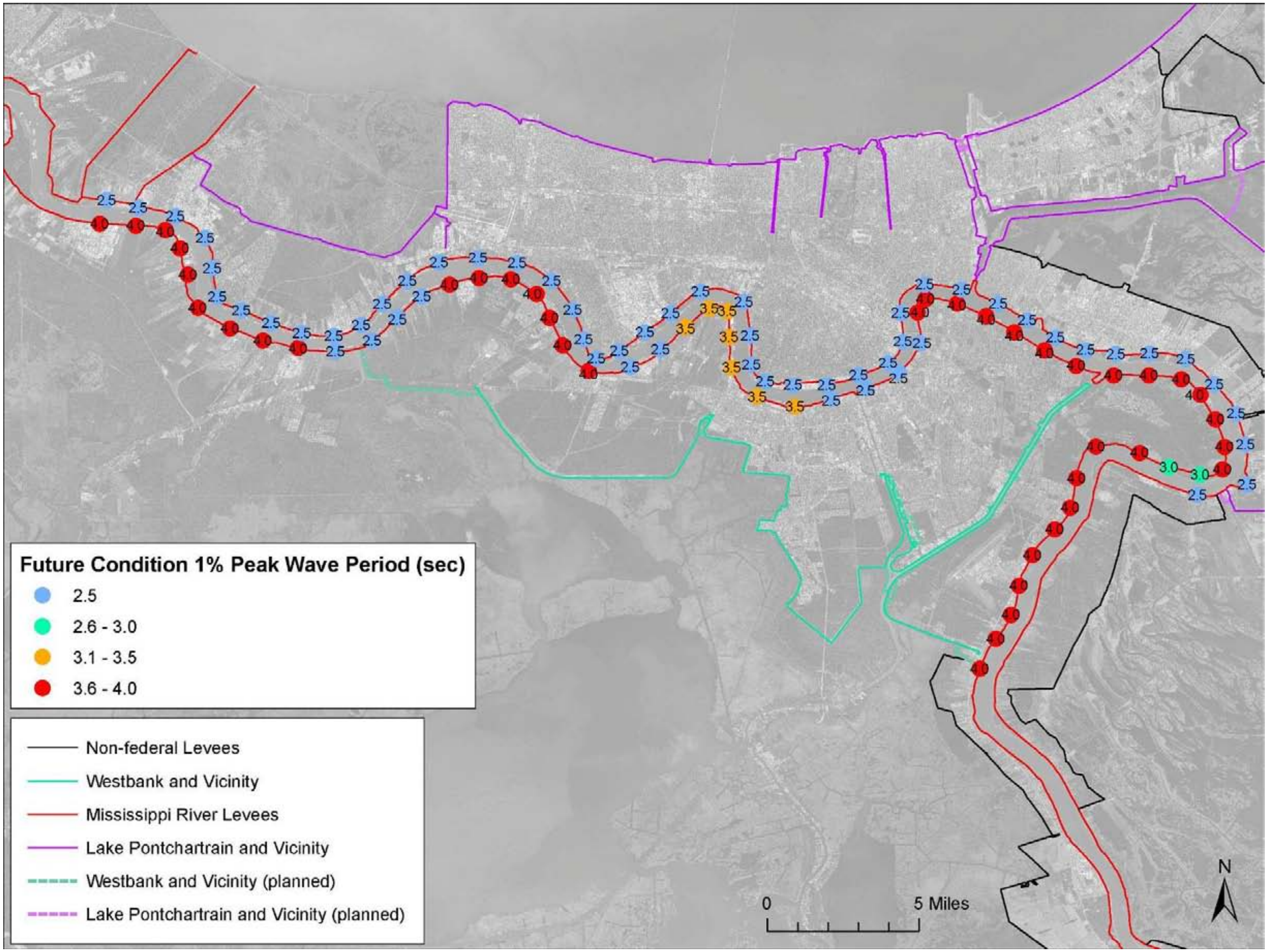

Figure 31 – Future Condition - Recommended 1% Peak Wave Period 2D Plot

Appendix J

Numerical Modeling Study of Western Closure Complex Project

(Letter Report Version 4, Authors: ERDC)

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APPENDIX J – NUMERICAL MODELLING STUDY OF WESTERN CLOSURE COMPLEX PROJECT
(Letter report version 4)

Authors: ERDC

Summary

The New Orleans District (MVN) contracted the Coastal and Hydraulics Laboratory (CHL) to examine the effects of the proposed Western Closure Complex (WCC) project on surge levels and wave heights with the numerical models ADCIRC and STWAVE. The WCC project will be constructed to reduce the risk of flooding and consists of a floodgate south of the Harvey and Algiers canals. The purpose of the WCC floodgate is to reduce flooding north of the gate location during storm events. A detailed description of the floodgate specifications is provided by MVN. This section focuses on the difference in surge and waves with and without the WCC floodgate in place.

Storm water levels and wave heights were computed with the proposed Western Closure Complex project in place and compared with the base condition (the 2010 grid previously developed for LACPR/FEMA). A suite of 10 storms was selected for simulation by CHL in consultation with MVN from the existing Louisiana storm suite database developed for previous FEMA and LACPR studies. The performance of the project was evaluated by conducting a sensitivity analysis, i.e. by comparing the WCC simulated storm water levels and wave heights to the previously run base condition simulated results.

In general, the changes in maximum surge as a result of the WCC project are small for all storms simulated, on the order of 0.2 ft or less for areas south of the project floodgate location. Likewise, the changes in maximum waves as a result of the WCC project are small for all storms simulated, on the order of 0.5 ft or less for areas south of the project floodgate location. For areas north of the WCC floodgate, the maximum storm surge is reduced by 2-11.5 ft in the Harvey Canal and Intracoastal Waterway, depending on the storm characteristics (such as track) and statistical surge level (return frequency). Changes in maximum wave heights were not computed for areas north of the WCC floodgate due to the resolution of the STWAVE model domain.

Study Area

The WCC project is located on the west bank of the Mississippi River, south of New Orleans and includes a floodgate south of the intersection of the Harvey Canal and Intracoastal Waterway. The bathymetry and topography are shown in Figure 1.

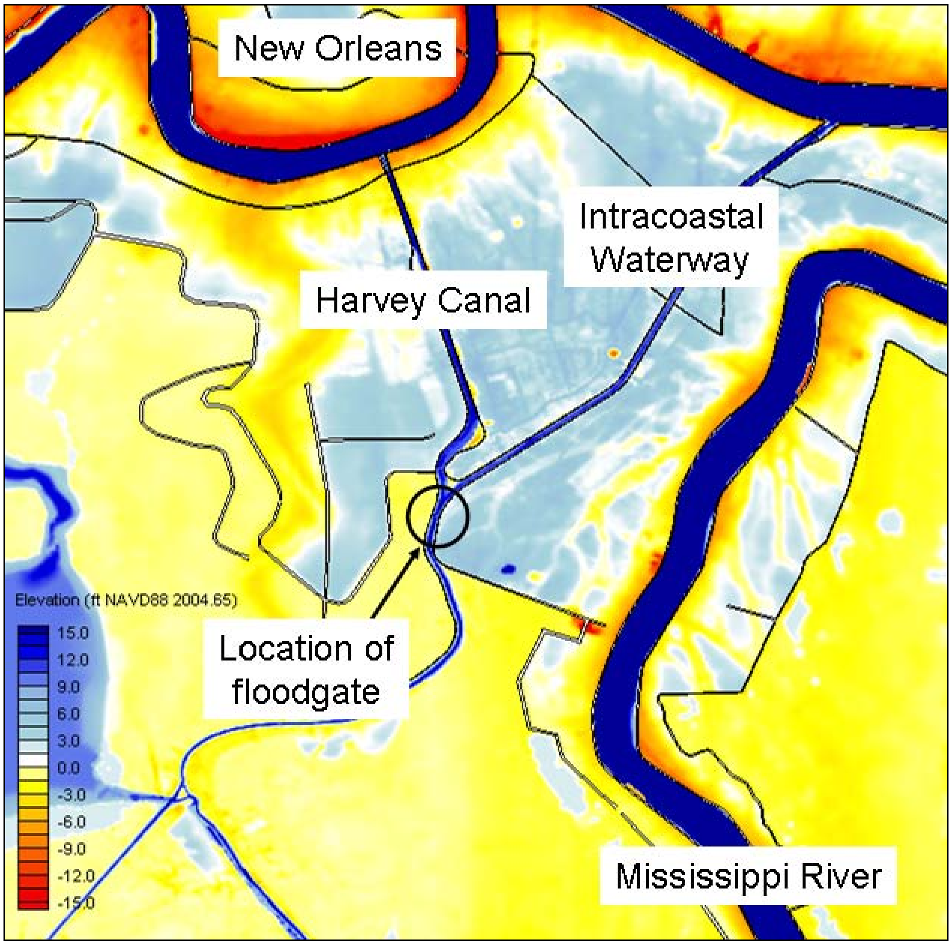


Figure 1: Bathymetry/topography (ft NAVD88 2004.65) for study area.

Grid Modifications

MVN provided CHL with geo-referenced data files containing the location of the proposed project and the base grid levee alignment was modified accordingly. The base grid levee heights in this area are based upon the authorized 2010 levee elevations and are generally 30 ft high. The floodgate was also modeled with a height of 30 ft. Figure 2 shows the base grid levee alignment and the grid modifications for the with-project WCC alignment. Figure 3 shows the bathymetry and topography in the immediate vicinity of the WCC floodgate for the base grid as well as the modified with-project WCC alignment.

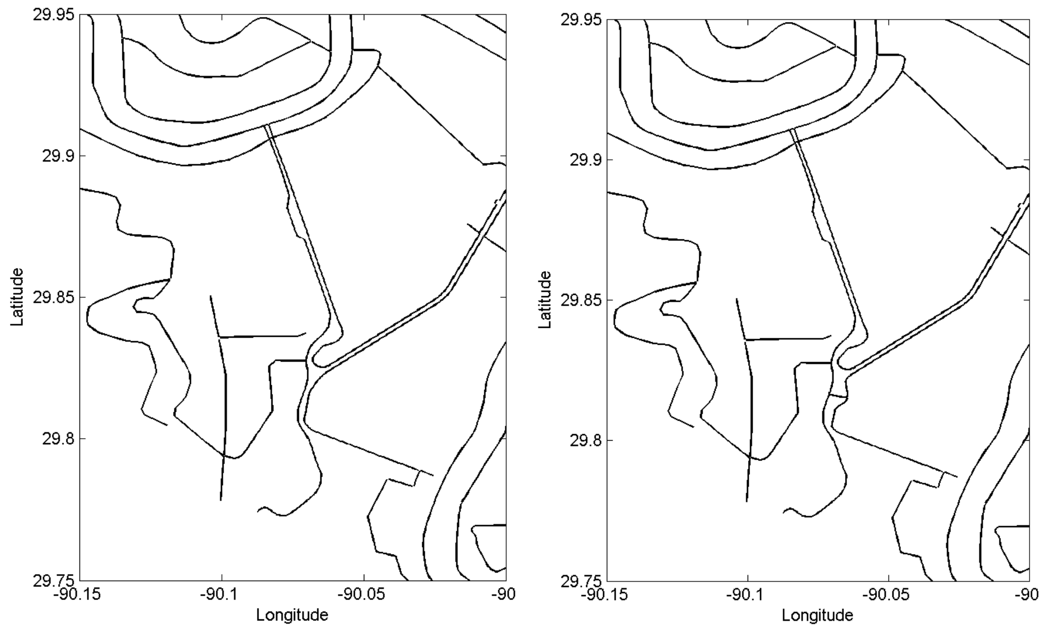


Figure 2: Base grid (2010) levee alignment (left panel) and with-project grid (WCC) levee alignment (right panel).

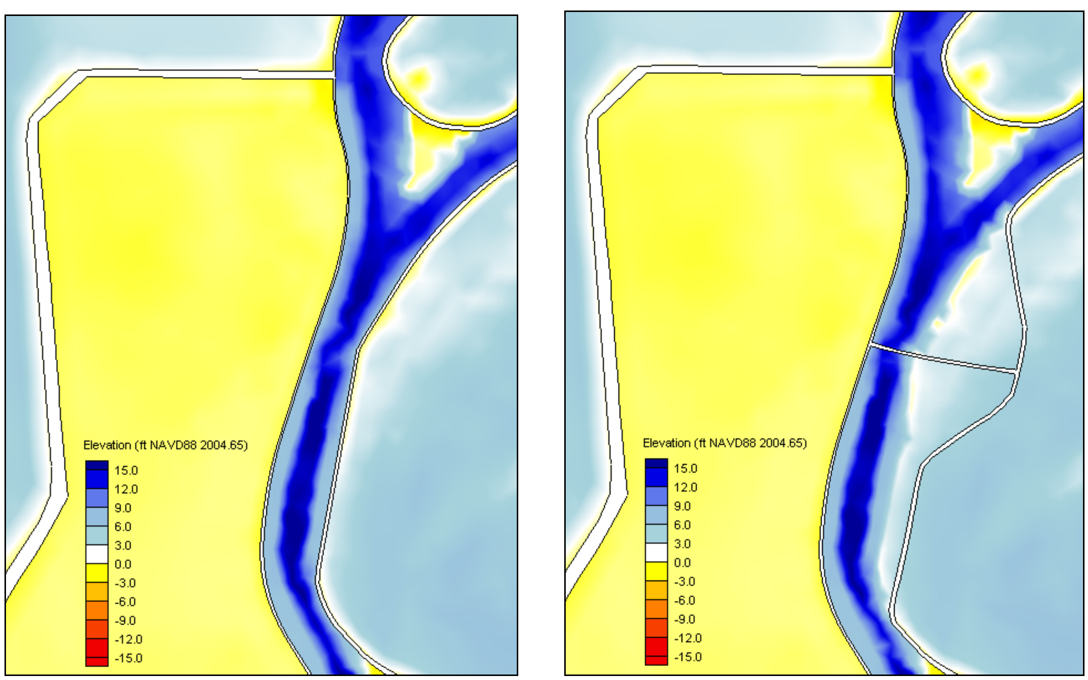


Figure 3: Bathymetry/topography (ft NAVD88 2004.65) in the immediate vicinity of the WCC floodgate for the base grid 2010 alignment (left panel) and the modified with-project WCC alignment (right panel).

Storm Selection

As requested by MVN, ten storms were to be selected for simulation according to the following criteria: 1) three storms having a surge level corresponding to a 50-year water level in the vicinity of the WCC within +/- 0.5 ft; 2) three storms having a surge level corresponding to a

100-year water level in the vicinity of the WCC within +/- 0.5 ft; 3) three storms having a surge level corresponding to a 500-year water level in the vicinity of the WCC within +/- 0.5 ft; and lastly, 4) Storm 050 because the characteristics of that synthetic storm were most similar to recently occurring Hurricane Gustav (2008). The trajectories for the WCC storm suite are shown in Figure 4.

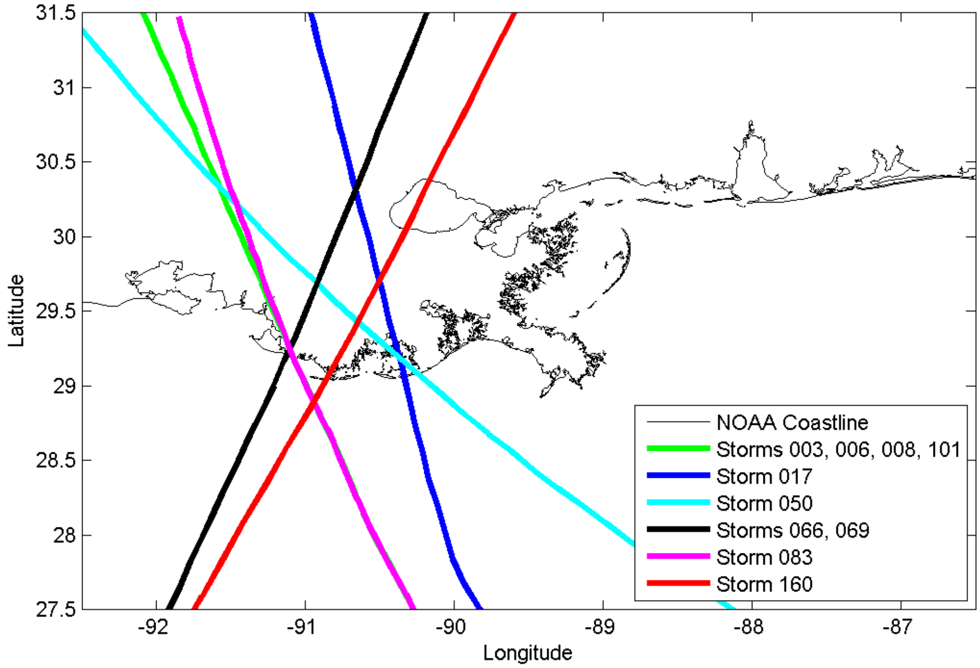


Figure 4: Storm tracks for the WCC storm suite.

Five save locations in the vicinity of the WCC from the L274 save point set were analyzed to aid in the selection of storms that produce the various water levels (50-yr, 100-yr, 500-yr) in the project area (Figure 5). Statistics for these and other save locations were computed in a prior study (LACPR/FEMA) and the statistical surge levels at the five selected save locations are given in Table 1. Knowing the statistical 50-yr, 100-yr, and 500-yr water levels at these five save locations, the previously simulated storms were examined to see which storms produced the statistical levels at the particular locations of interest within +/- 0.5 ft. That is, the storm responses for all 152 LAEAST storms were examined at each of the five points to determine which storms produce the statistical 50-, 100-, and 500-yr water levels. For example, at Point 10, the 50-yr level was determined to be 6.0 ft. Storms 11, 12, 66, 94, 137, and 153 all produced water levels around the 6.0 ft mark. For Point 37, the 50-yr level was determined to be 4.6 ft. Storms 3, 11, 66, 82, 101, and 112 all produced water levels around the 4.6 ft mark. This was repeated for the other three save locations and storms that produced the 50-yr water levels at those locations are given in Table 2. The storms that came closest to producing the 50-yr water level at most/all of the five points are Storms 003, 066, and 101. These storms were therefore selected for simulation. This procedure was repeated for the 100-yr and 500-yr return periods and the resulting selected storms are also given in Table 2.

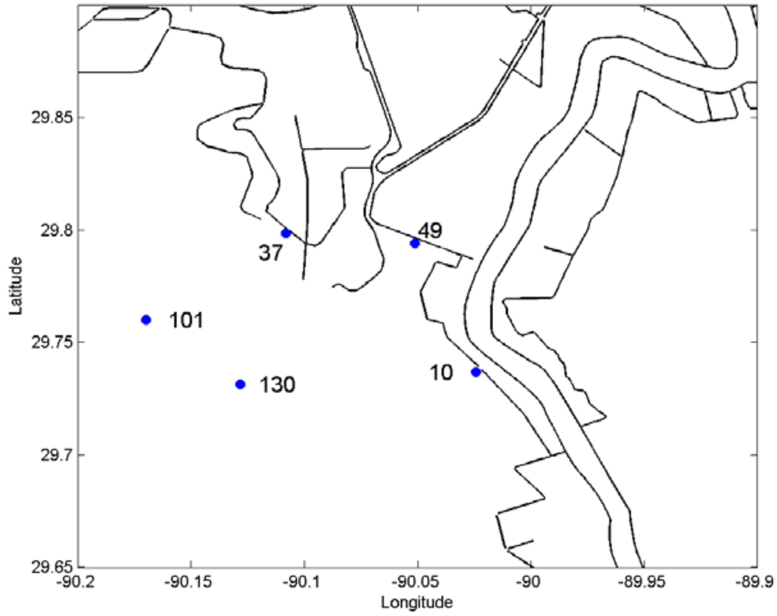


Figure 5: Location of the five L274 save points analyzed as part of the storm selection procedure. The background image is the base condition levee alignment.

Table 1: Surge Levels from 2010 Statistics.

L274 Save Point	50-yr surge (ft NAVD88 2004.65)	100-yr surge (ft NAVD88 2004.65)	500-yr surge (ft NAVD88 2004.65)
10	6.0	8.1	12.3
37	4.6	6.3	9.4
49	5.4	7.5	11.6
101	4.7	6.1	8.4
130	5.3	7.1	10.2

Table 2. Storms that Produced Statistical Surge Levels in Table 1.

L274 Point	50-yr surge	100-yr surge	500-yr surge
10	011 012 066 094 137 153	008 093 102 111	017 069
37	003 011 066 082 101 112	006 008 087 111 118 127 145 160	017 069 083 097
49	003 011 012 066 082 101 152 153	006 008 072 102 111 160	017 069 149
101	003 066 067 072 082 101 118 131	006 008 014 015 053 093 126 145 160	018 069 083 097 140
130	003 011 012 066 067 068 082 101 112 131 137 153	006 008 072 102 126 127 145 160	017 069
Selected Storms	**003 066 101**	**006 008 160**	**017 069 083**

Surge Response

In general, the changes in maximum surge as a result of the WCC project are small for all storms simulated for areas south of the project floodgate, on the order of 0.2 ft or less. For the with project condition, surge is prevented from propagating north of the floodgate into the Harvey Canal and Intracoastal Waterway. Instead, this volume of water is distributed over a much larger area south of the floodgate. Hence, the changes in maximum surge are small for areas south of the floodgate.

Maximum surge and difference results are shown in Figures 6-8 for Storm 160 (a storm that produced the 100-year water level in the vicinity of the project for the base condition). For areas north of the WCC floodgate, the maximum storm surge is reduced by 2-11.5 ft in the Harvey Canal and Intracoastal Waterway, depending on the storm characteristics (such as track) and statistical surge level (return frequency). In general, the maximum storm surge is reduced by 4-6 ft in the Harvey Canal and Intracoastal Waterway for those storms which produce the 50-year water level (Storm 003, Storm 066, and Storm 101), 4.5-7 ft for those storms which produce the 100-year water level (Storm 006, Storm 008, and Storm 160), and 7.5-11.5 ft for those storms which produce the 500-year water level (Storm 017, Storm 069, and Storm 083). For Storm 050 (Gustav-like storm), the maximum storm surge is reduced by 2-4 ft in the Harvey Canal and Intracoastal Waterway. Maximum surge and difference maps for the base condition (2010) and with-project condition (WCC) for the entire storm suite are provided in Appendices A through C.

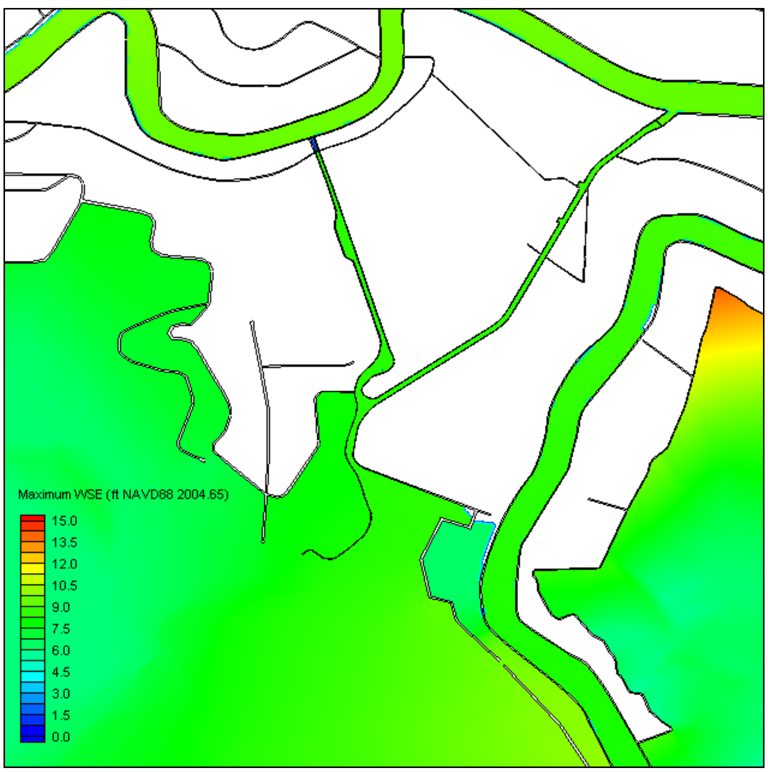


Figure 6: Maximum Surge (ft NAVD 88 2004.65) for Storm 160 for the base (2010) condition.

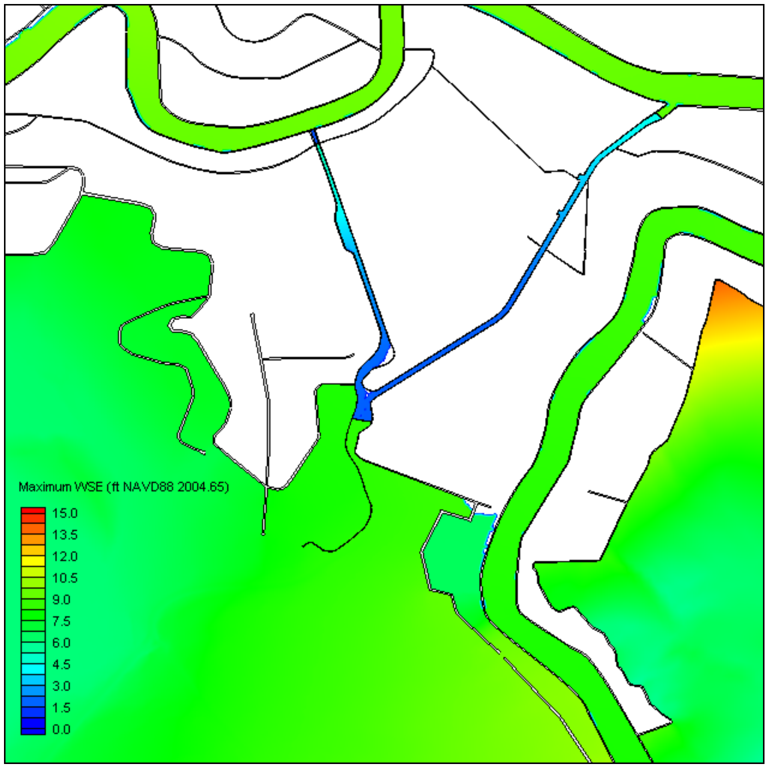


Figure 7: Maximum Surge (ft NAVD 88 2004.65) for Storm 160 for the with-project (WCC) condition.

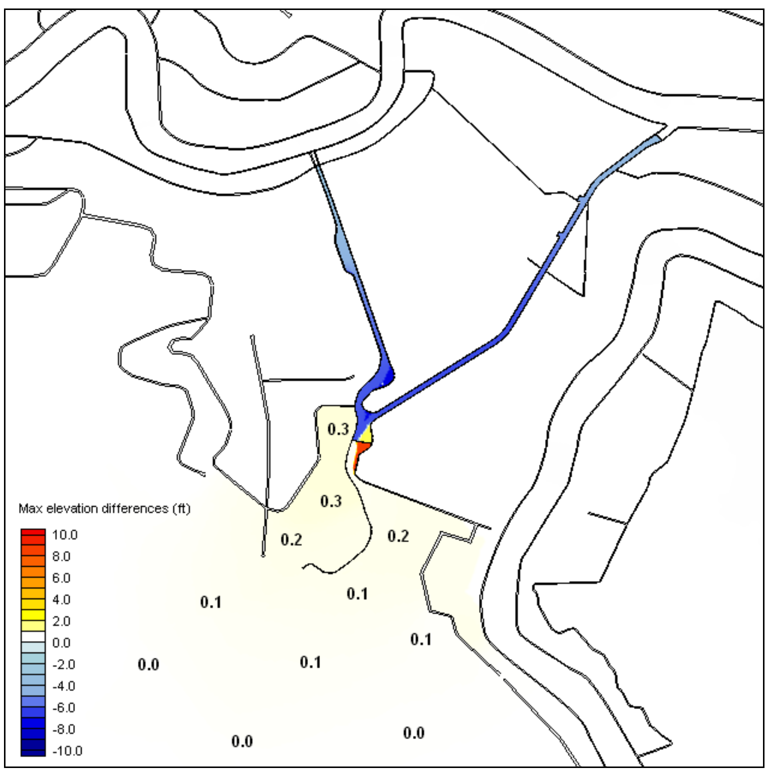


Figure 8: Differences in Maximum Surge (ft) for Storm 160: WCC minus Base condition.

Waves Response

The resolution of the STWAVE grid is fixed at 200 meters throughout the project site. Therefore, it is not possible to simulate the wave behavior in the Harvey Canal and Intracoastal Waterway immediately north of the WCC floodgate structure with the pre-existing STWAVE grid. However, for areas south of the WCC floodgate, the changes in maximum waves as a result of the WCC project are small for all storms simulated, on the order of 0.5 ft or less. Because the surge differences are small south of the floodgate, the changes in maximum wave height are likewise small.

Maximum waves and difference results are shown in Figures 9-11 for Storm 160 (a storm which produced the 100-year water level in the vicinity of the project for the base condition). Maximum wave heights and difference maps for the base condition (2010) and with-project condition (WCC) for the entire storm suite are provided in Appendices D through F. Note that the effects of bottom friction on waves are not included in this report.

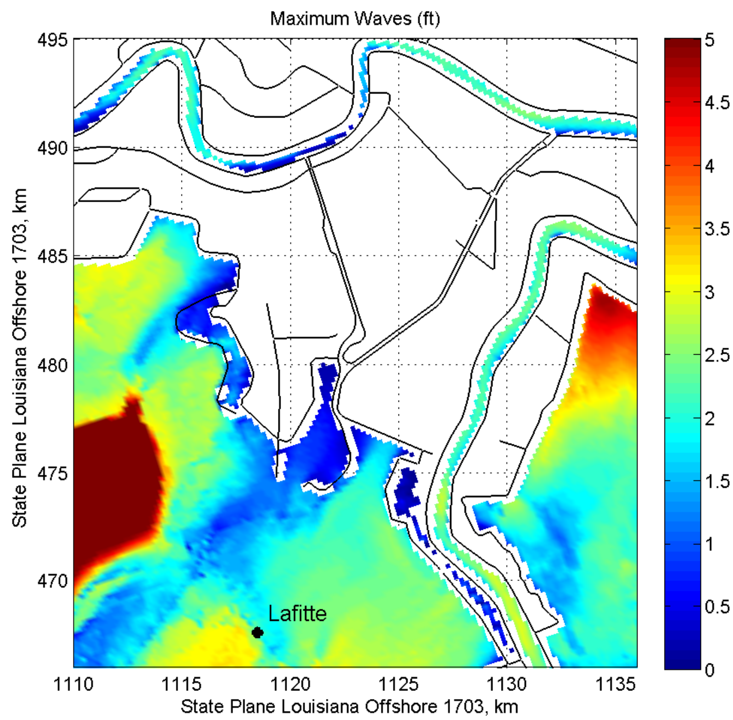


Figure 9: Maximum Waves (ft) for Storm 160 for the base condition.

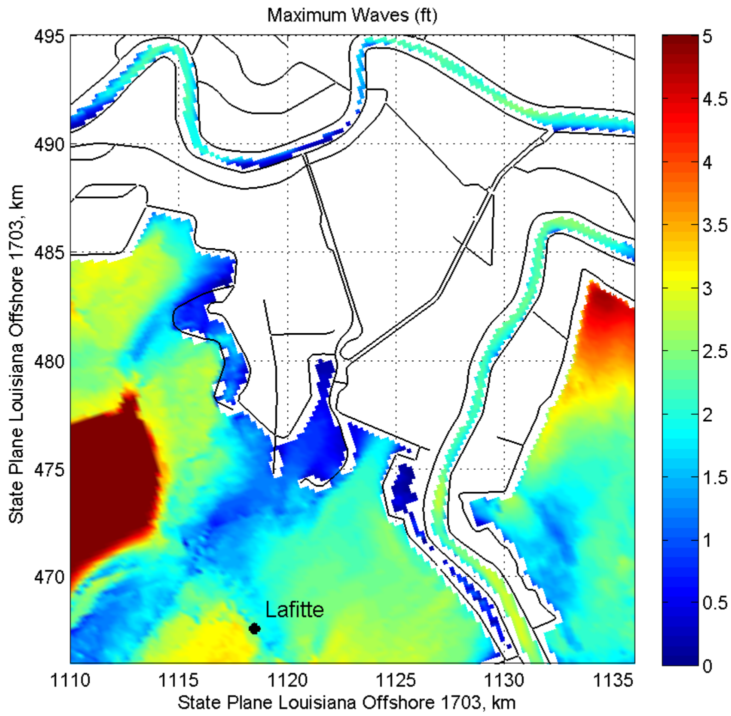


Figure 10: Maximum waves (ft) for Storm 160 for the with-project condition.

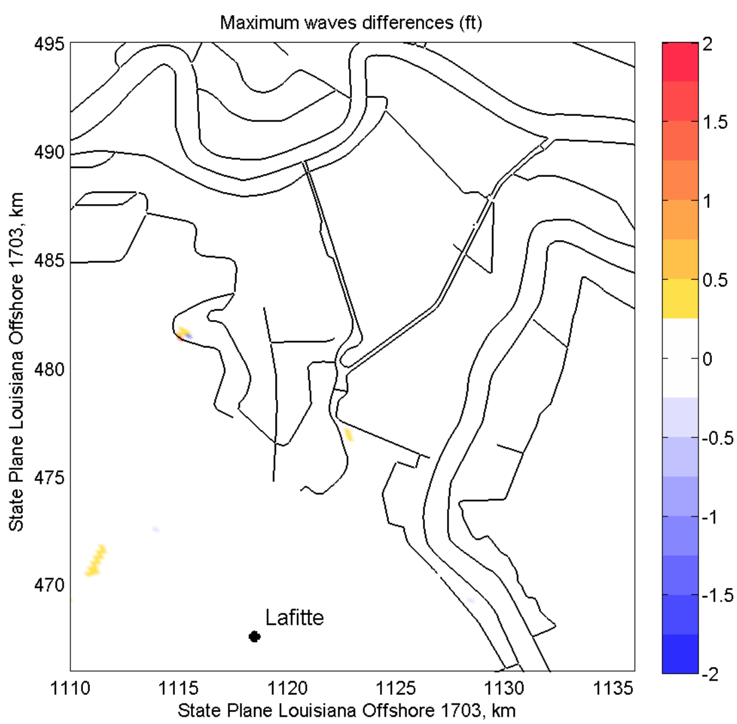


Figure 11: Differences in Maximum Wave Heights (ft) for Storm 160: WCC minus 2010.

Save Locations and Time Series

Six save locations were selected by MVN and CHL for further examination of water level time series. The save locations sites are shown in Figure 12 and listed in Table 3 below.

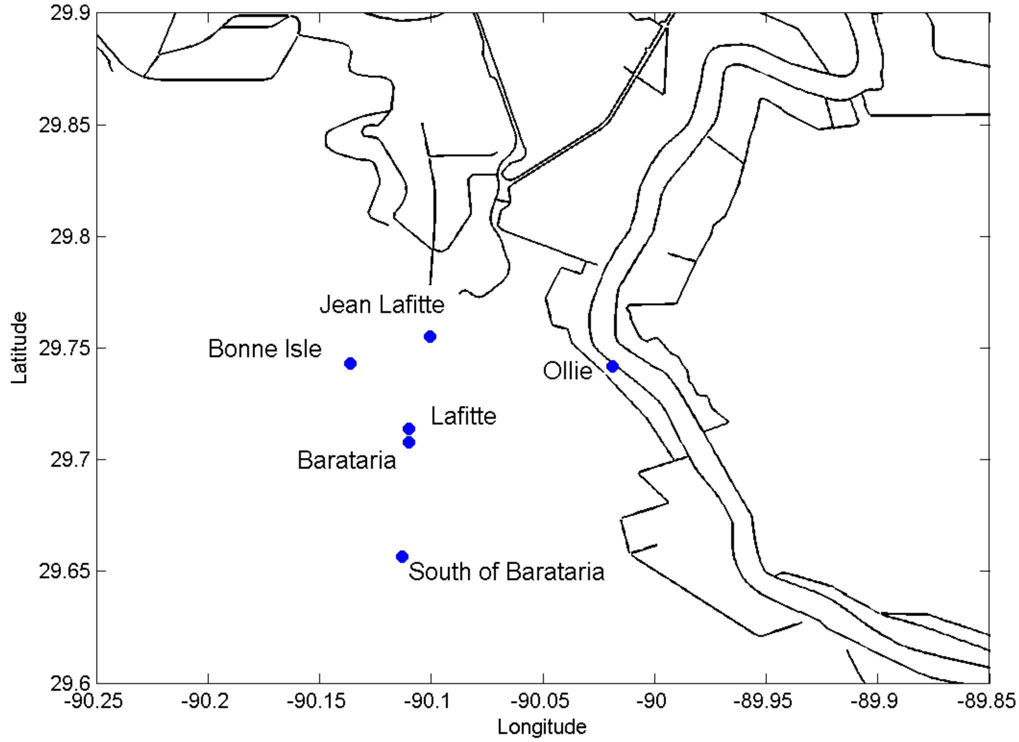


Figure 12: Location map of the six save points.

Table 4 gives the maximum surge values for each of the ten storms at each of the six save locations for the base (2010) and with-project (WCC) conditions. Note that data marked "Dry" indicates that the particular save location did not inundate for a given storm event. For all of the six save locations, the difference in maximum surge is small, on the order of 0.20 ft or less for all storms simulated. The average difference in maximum surge is 0.03 ft.

Table 5 gives the maximum wave height values for each of the ten storms at each of the six save locations for the base (2010) and with-project (WCC) conditions. The six save locations are located to the west of the Mississippi River; therefore the values given in Table 5 are based on the South STWAVE domain. For all of the six save locations, the difference in maximum wave height values is small, on the order of 0.03 ft or less for all storms simulated. Note that the effects of bottom friction on waves are not included in this report.

Surge time series results are shown in Figure 13 for Storm 160 (a storm which produced the 100-year water level in the vicinity of the project for the base condition) at Lafitte. The difference in simulated storm surge between the base and with-project scenarios is negligible. The full set of time series for each save location and storm are located in Appendix G.

Table 3: Coordinates for the six save points.

Save Point	Name	Longitude	Latitude
1	South of Barataria	-90.112850000	29.656869444
2	Jean Lafitte	-90.100408333	29.755002778
3	Bonne Isle	-90.136361111	29.743013889
4	Lafitte	-90.110091667	29.713788889
5	Barataria	-90.110086111	29.708047222
6	Ollie	-90.018895800	29.741990400

Table 4: Maximum surge values (ft NAVD88 2004.65) for each of the ten storms at each of the six save locations for the base (2010) and with-project (WCC) conditions.

	South of Barataria		Jean Lafitte		Bonne Isle		Lafitte		Barataria		Ollie	
	2010	WCC	2010	WCC	2010	WCC	2010	WCC	2010	WCC	2010	WCC
003	5.79	5.79	5.12	5.23	4.68	4.72	5.54	5.55	5.70	5.70	4.98	5.00
006	7.64	7.64	6.78	6.94	6.11	6.16	7.30	7.30	7.45	7.44	6.37	6.40
008	7.84	7.85	6.88	7.03	6.29	6.34	7.44	7.45	7.61	7.61	6.44	6.47
017	11.24	11.24	10.81	10.85	8.40	8.56	11.15	11.17	11.28	11.28	12.57	12.57
050	4.65	4.65	4.08	4.09	3.58	3.60	4.35	4.35	4.55	4.55	Dry	Dry
066	5.30	5.30	5.34	5.40	4.71	4.76	5.39	5.40	5.52	5.52	5.26	5.29
069	10.53	10.54	10.99	11.07	9.73	9.81	10.83	10.86	10.85	10.87	12.30	12.33
083	10.17	10.17	10.06	10.15	9.76	9.79	9.97	10.00	10.06	10.07	10.08	10.17
101	6.04	6.04	5.20	5.25	4.74	4.77	5.66	5.67	5.83	5.84	4.99	5.01
160	7.90	7.90	7.83	7.89	6.26	6.36	7.88	7.92	7.93	7.96	9.12	9.16

Table 5: Maximum wave heights (ft NAVD88 2004.65) for each of the ten storms at each of the six save locations for the base (2010) and with-project (WCC) conditions.

	South of Barataria		Jean Lafitte		Bonne Isle		Lafitte		Barataria		Ollie	
	2010	WCC	2010	WCC	2010	WCC	2010	WCC	2010	WCC	2010	WCC
003	1.05	1.05	1.15	1.15	0.52	0.52	0.52	0.52	1.12	1.12	0.00	0.00
006	1.87	1.87	1.84	1.84	1.15	1.15	1.15	1.15	1.71	1.71	0.00	0.00
008	2.03	2.03	1.94	1.94	1.25	1.25	1.25	1.25	1.80	1.80	0.00	0.00
017	3.94	3.94	3.54	3.54	2.76	2.76	2.76	2.76	3.41	3.41	0.00	0.00
050	0.69	0.69	0.82	0.82	0.10	0.10	0.10	0.10	0.85	0.85	0.00	0.00
066	1.02	1.02	1.31	1.31	0.66	0.66	0.66	0.66	1.38	1.38	0.00	0.00
069	3.25	3.25	3.15	3.15	2.56	2.56	3.15	3.12	3.08	3.08	0.00	0.00
083	3.15	3.15	2.99	2.99	2.59	2.62	2.72	2.72	2.59	2.59	0.00	0.00
101	1.18	1.18	1.15	1.15	0.56	0.56	1.21	1.21	1.15	1.15	0.00	0.00
160	2.20	2.20	1.80	1.80	1.57	1.57	2.20	2.20	2.36	2.36	0.00	0.00

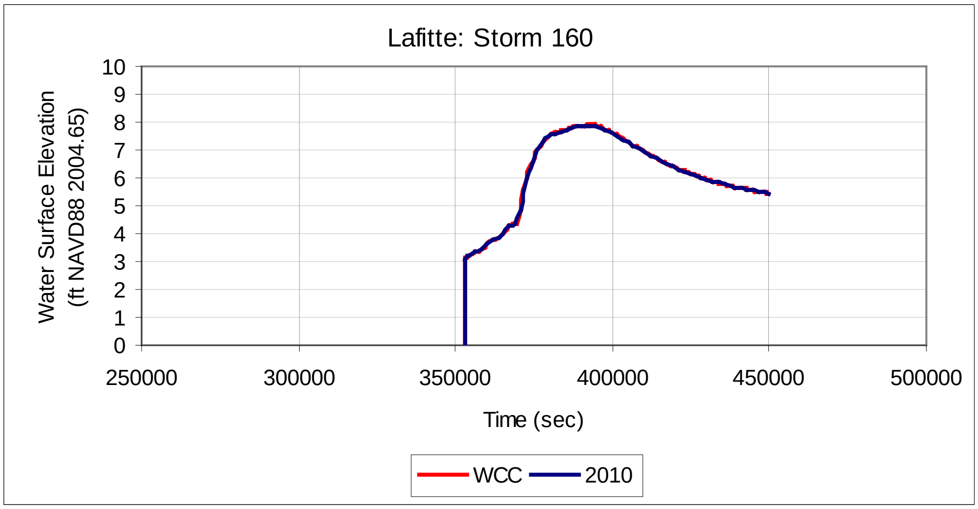


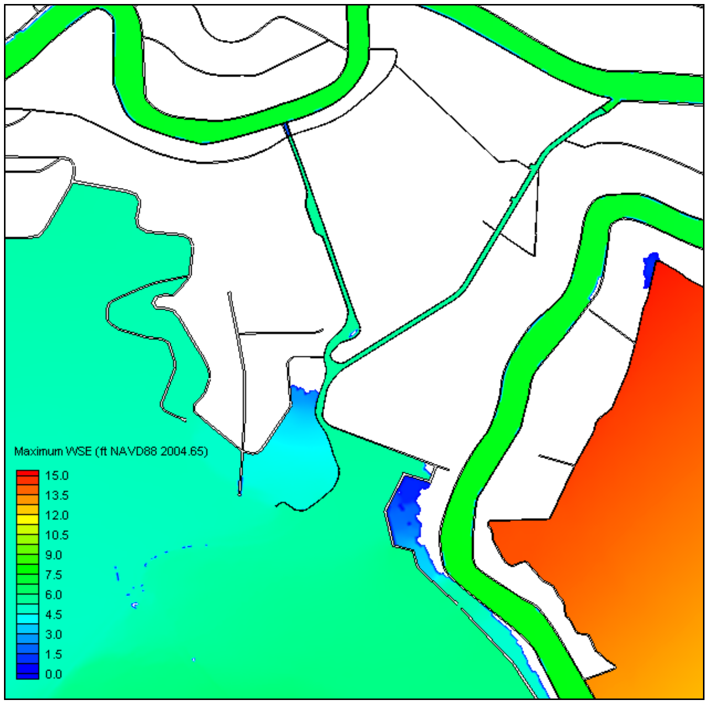
Figure 13: Time series of surge at Lafitte for Storm 160 for the base (2010) and with-project (WCC) conditions.

Conclusions

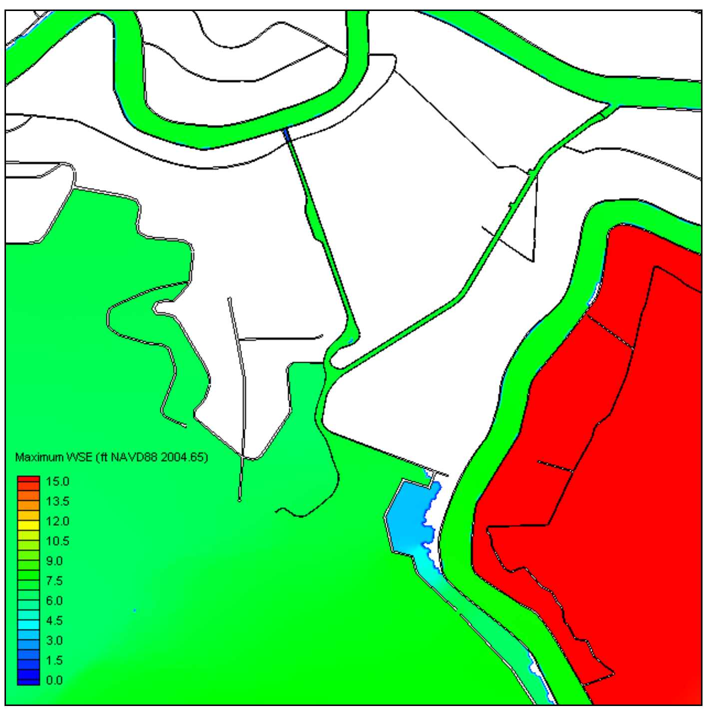
In general, the changes in maximum surge as a result of the WCC project are small for all storms simulated, on the order of 0.2 ft or less for areas south of the project floodgate location. Likewise, the changes in maximum waves as a result of the WCC project are small for all storms simulated, on the order of 0.25 ft or less for areas south of the project floodgate location. For areas north of the WCC floodgate, the maximum storm surge is reduced by 2-11.5 ft in the Harvey Canal and Intracoastal Waterway, depending on the storm characteristics (such as track) and statistical surge level (return frequency). Changes in maximum wave heights were not computed for areas north of the WCC floodgate due to the resolution of the STWAVE model domain.

Appendices to Letter Report WCC Modeling

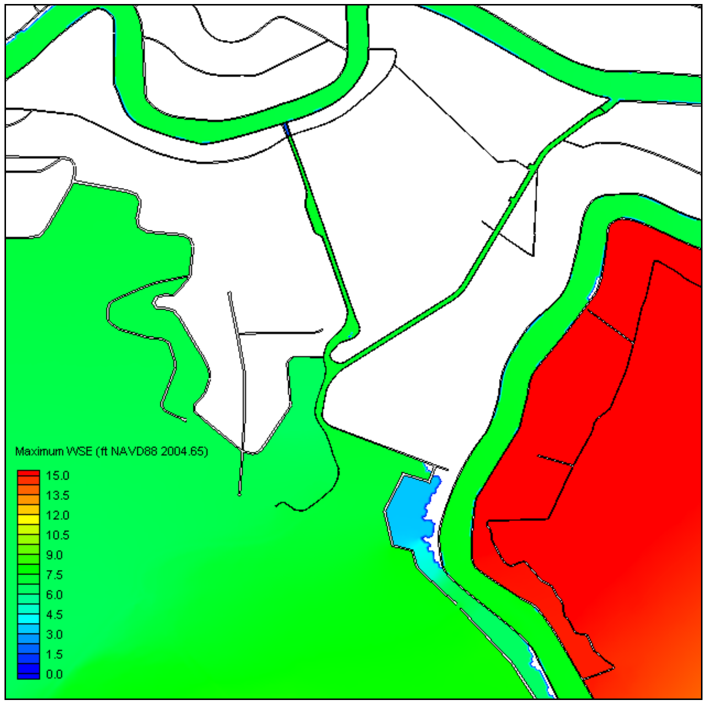
Appendix A: Maximum Surge Figures for the Base Condition (2010)



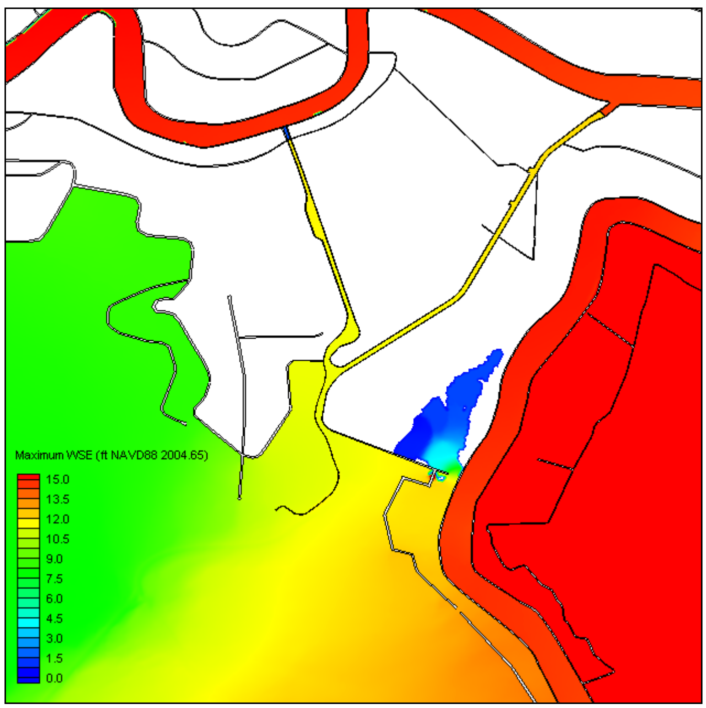
Maximum Surge (ft NAVD88 2004.65) for Storm 003



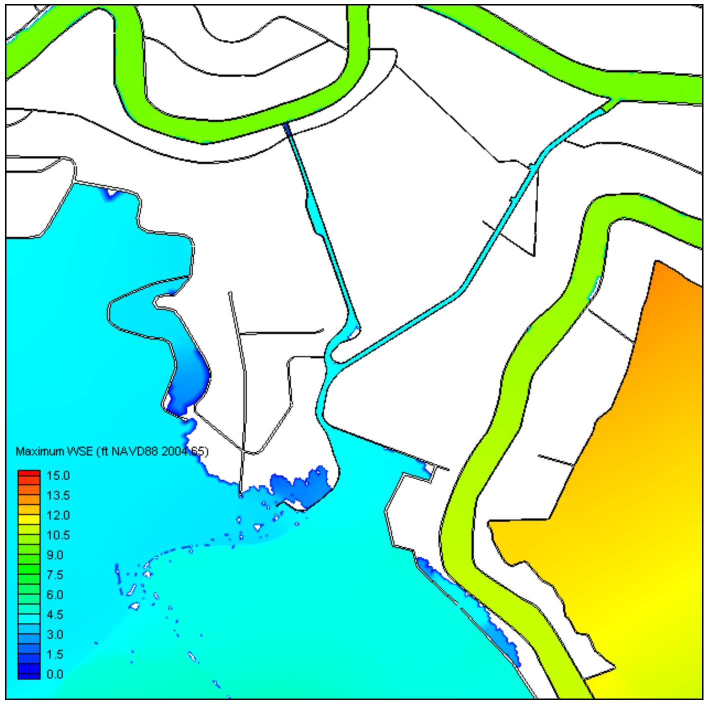
Maximum Surge (ft NAVD88 2004.65) for Storm 006



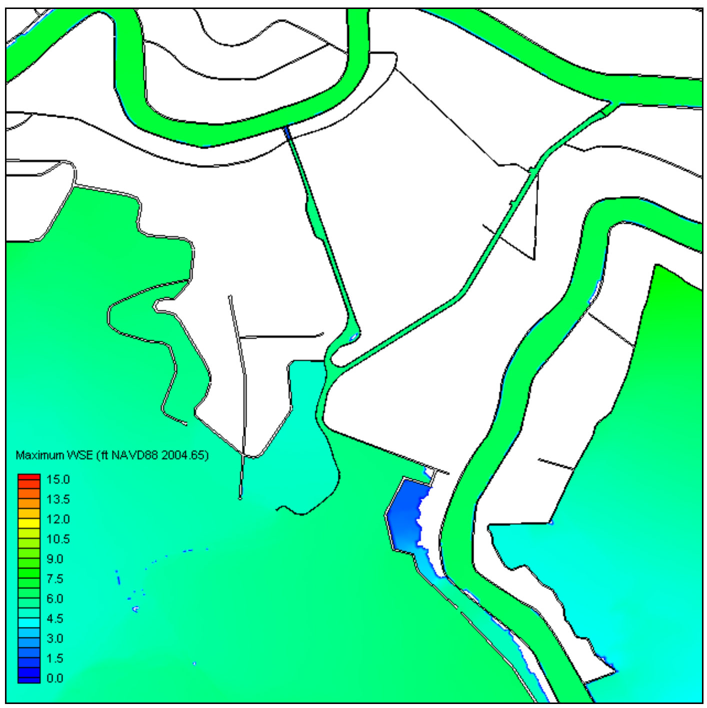
Maximum Surge (ft NAVD88 2004.65) for Storm 008



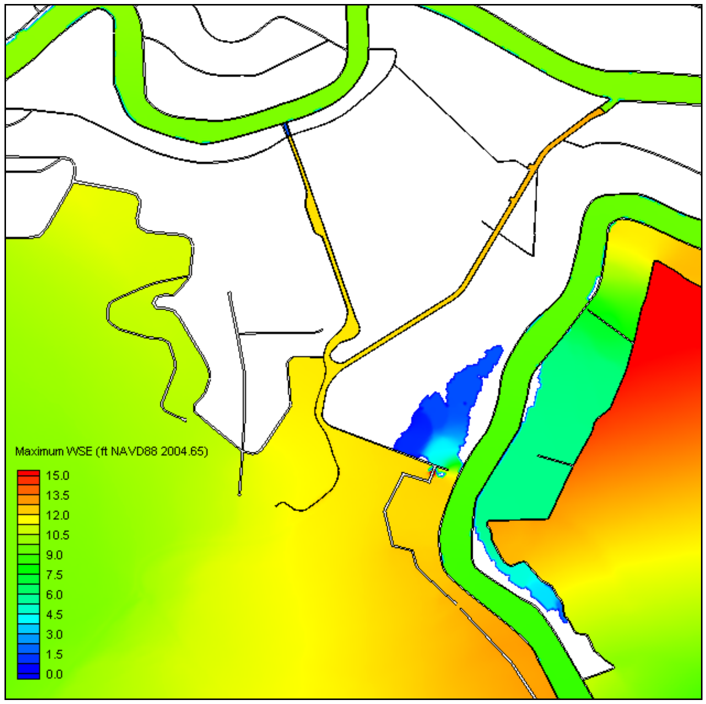
Maximum Surge (ft NAVD88 2004.65) for Storm 017



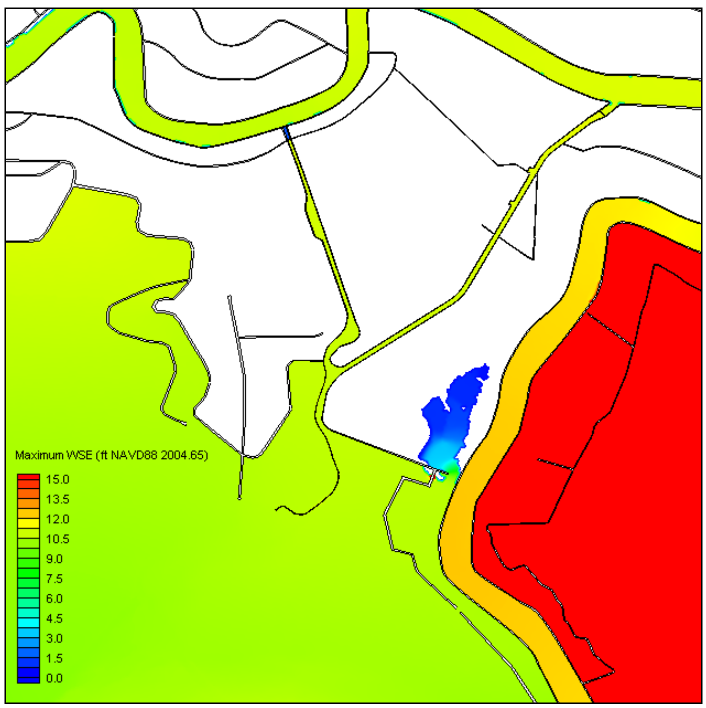
Maximum Surge (ft NAVD88 2004.65) for Storm 050



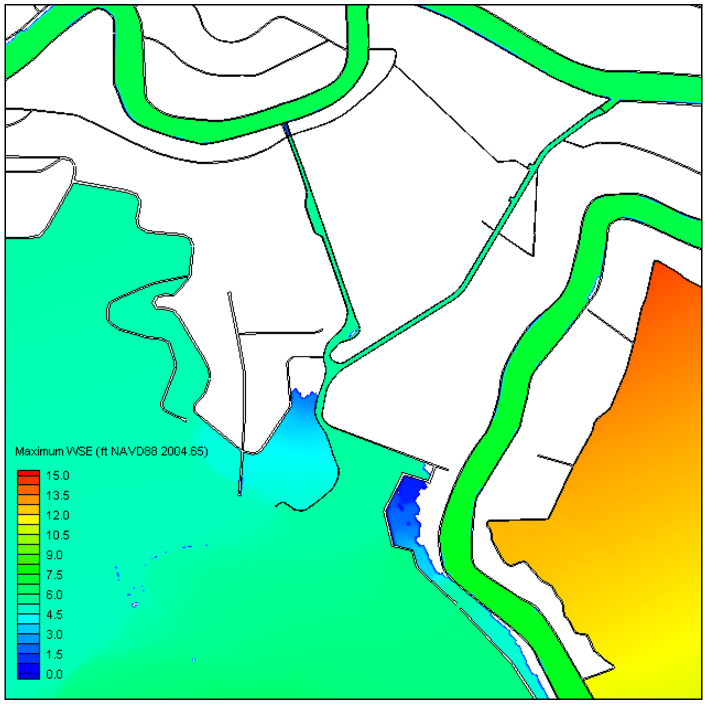
Maximum Surge (ft NAVD88 2004.65) for Storm 066



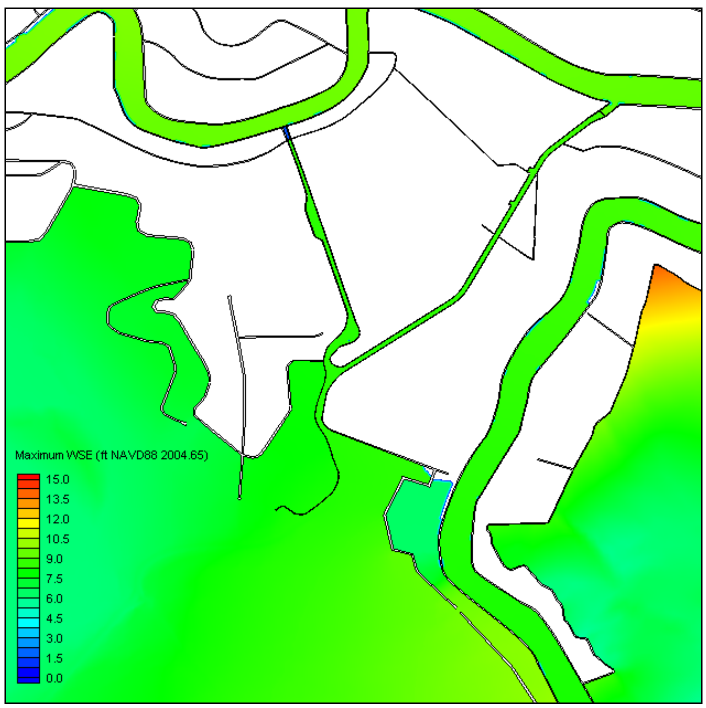
Maximum Surge (ft NAVD88 2004.65) for Storm 069



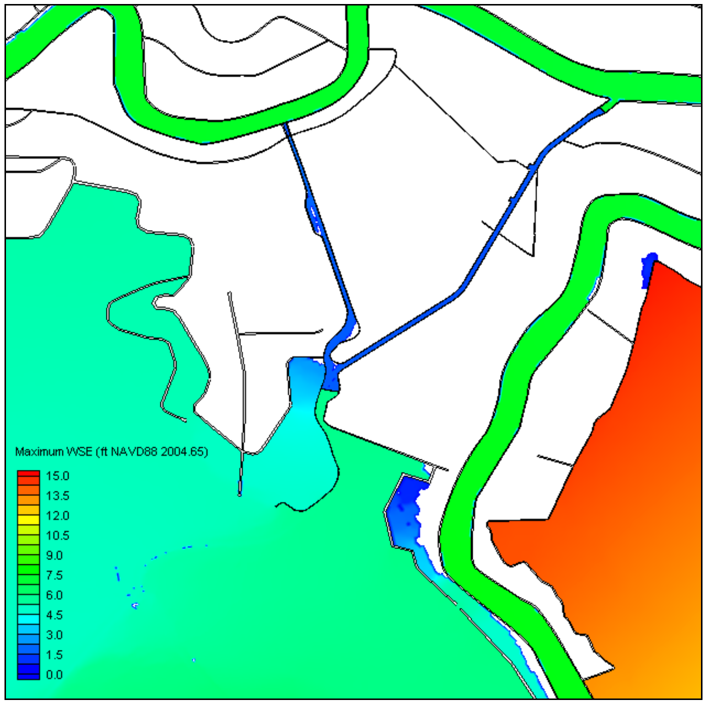
Maximum Surge (ft NAVD88 2004.65) for Storm 083



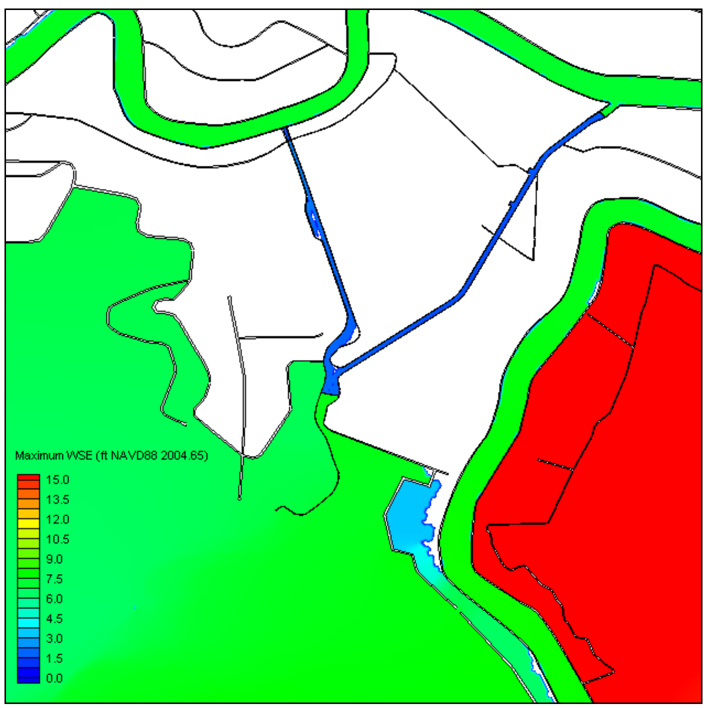
Maximum Surge (ft NAVD88 2004.65) for Storm 101



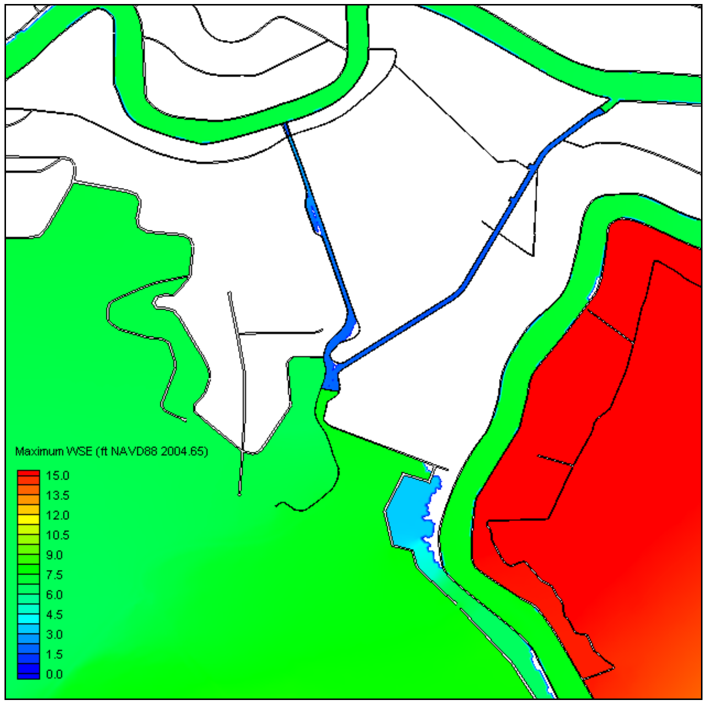
Maximum Surge (ft NAVD88 2004.65) for Storm 160



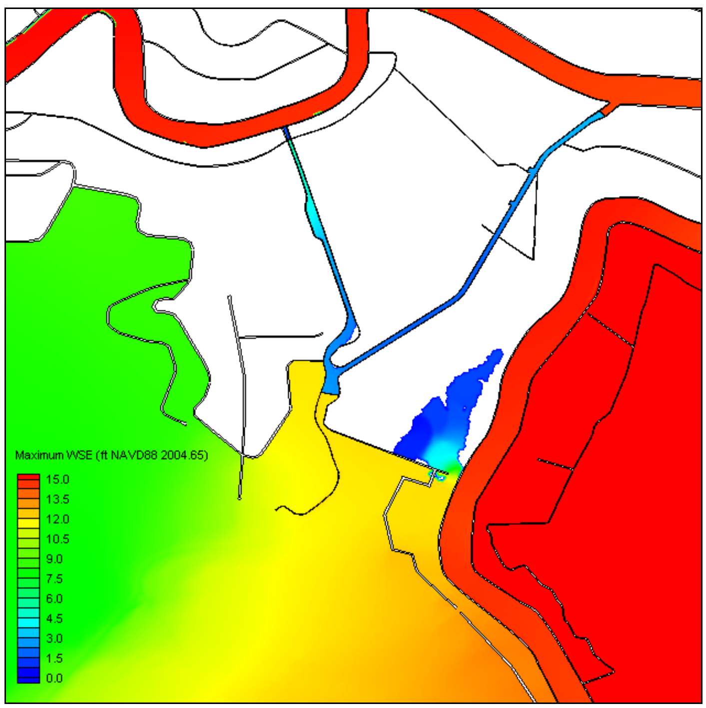
Maximum Surge (ft NAVD88 2004.65) for Storm 003



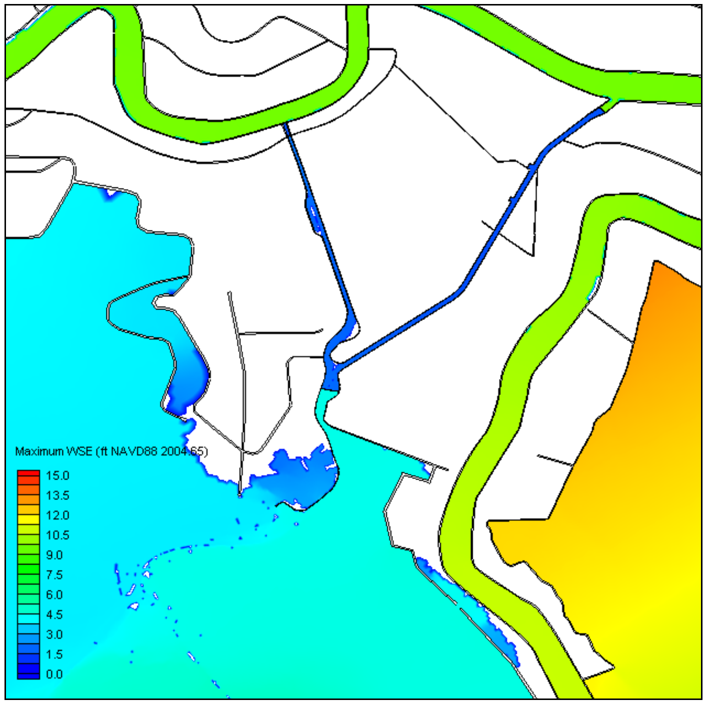
Maximum Surge (ft NAVD88 2004.65) for Storm 006



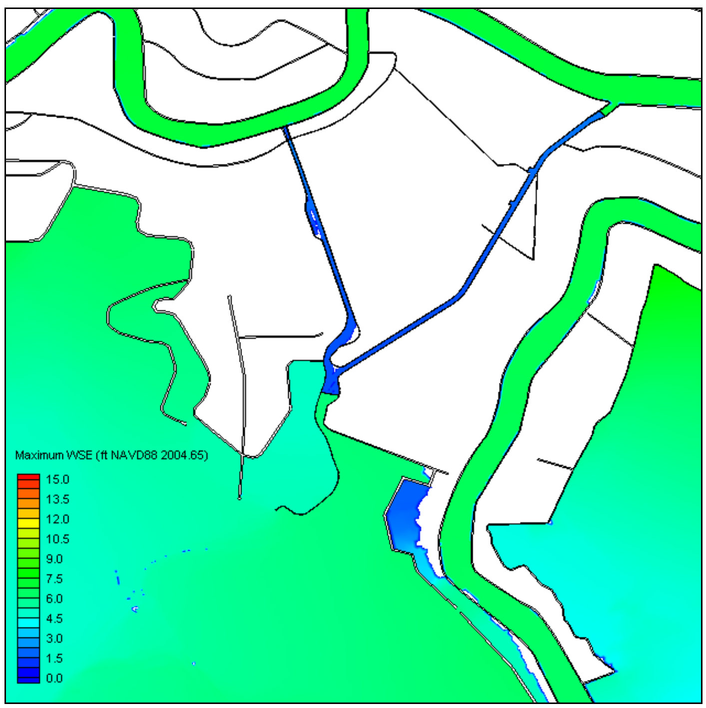
Maximum Surge (ft NAVD88 2004.65) for Storm 008



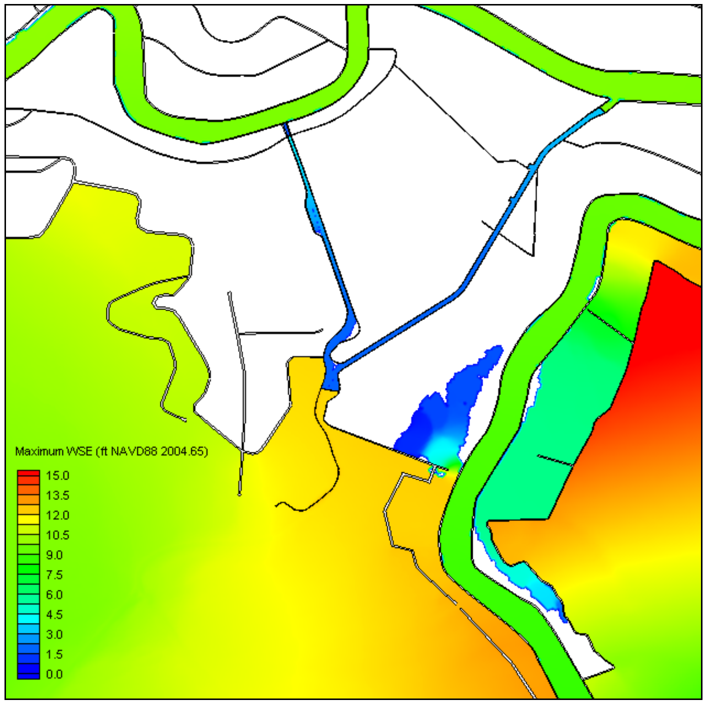
Maximum Surge (ft NAVD88 2004.65) for Storm 017



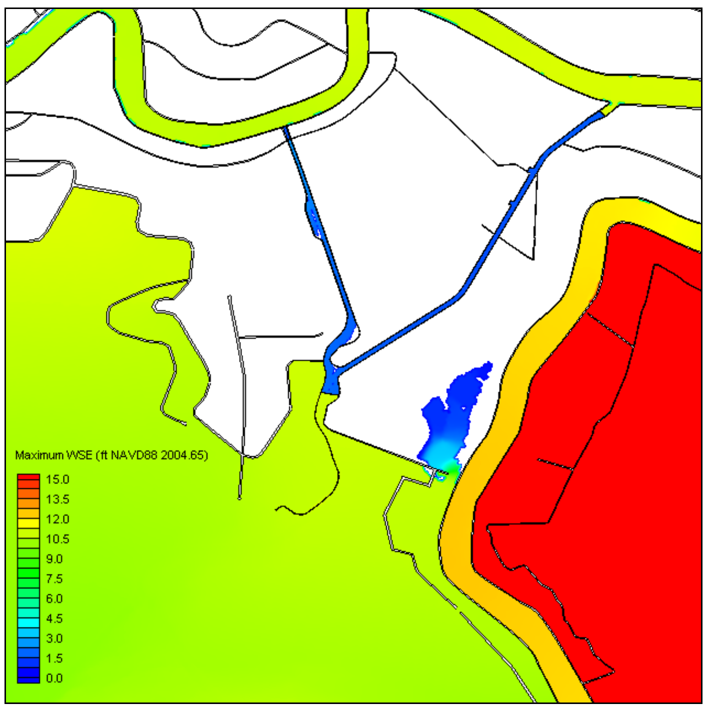
Maximum Surge (ft NAVD88 2004.65) for Storm 050



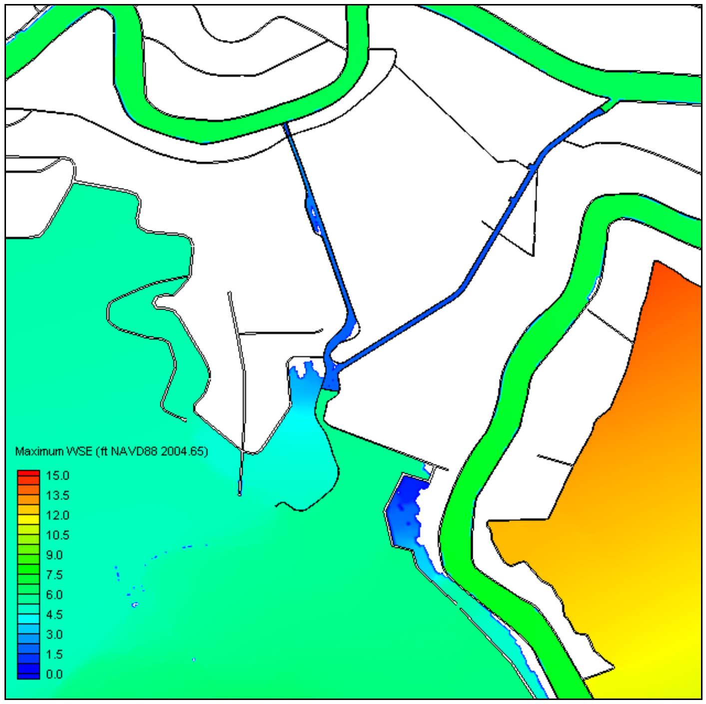
Maximum Surge (ft NAVD88 2004.65) for Storm 066



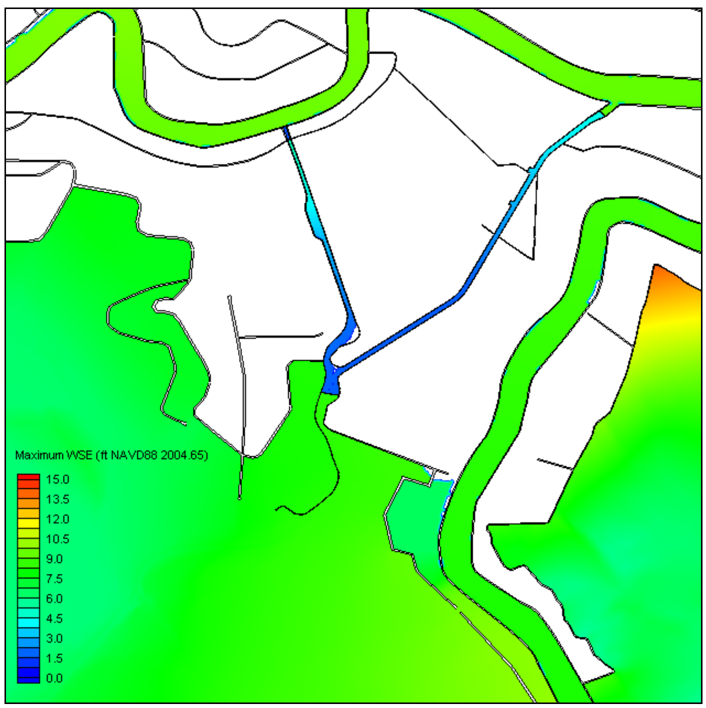
Maximum Surge (ft NAVD88 2004.65) for Storm 069



Maximum Surge (ft NAVD88 2004.65) for Storm 083

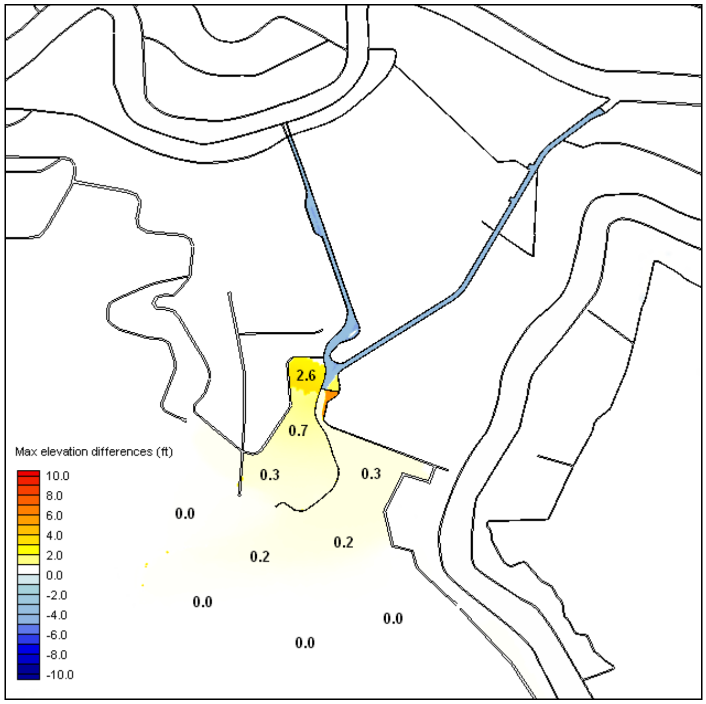


Maximum Surge (ft NAVD88 2004.65) for Storm 101

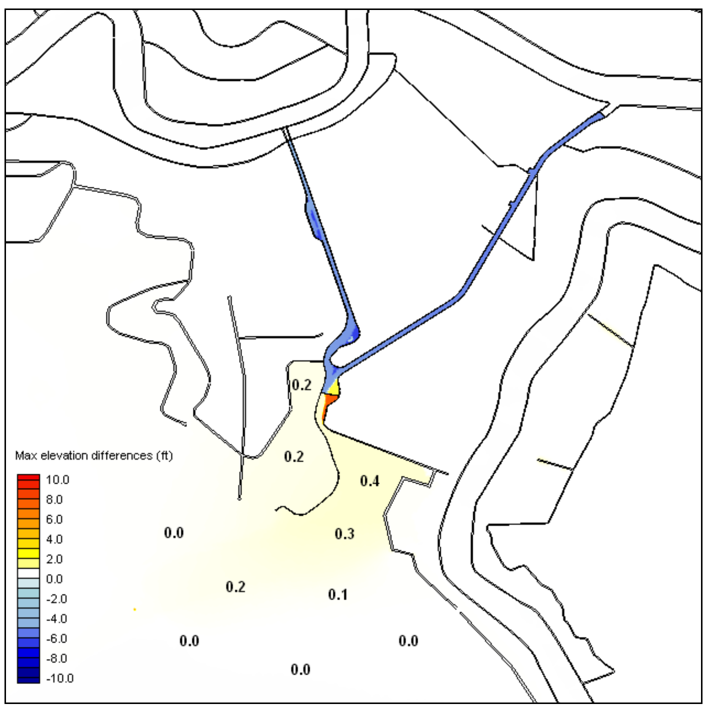


Maximum Surge (ft NAVD88 2004.65) for Storm 160

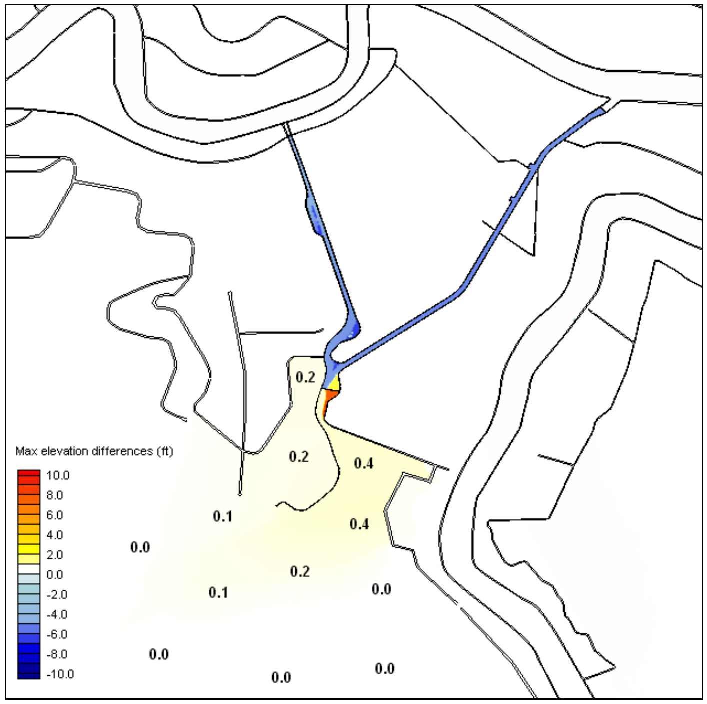
Appendix C: Maximum Surge Difference Figures for WCC minus 2010



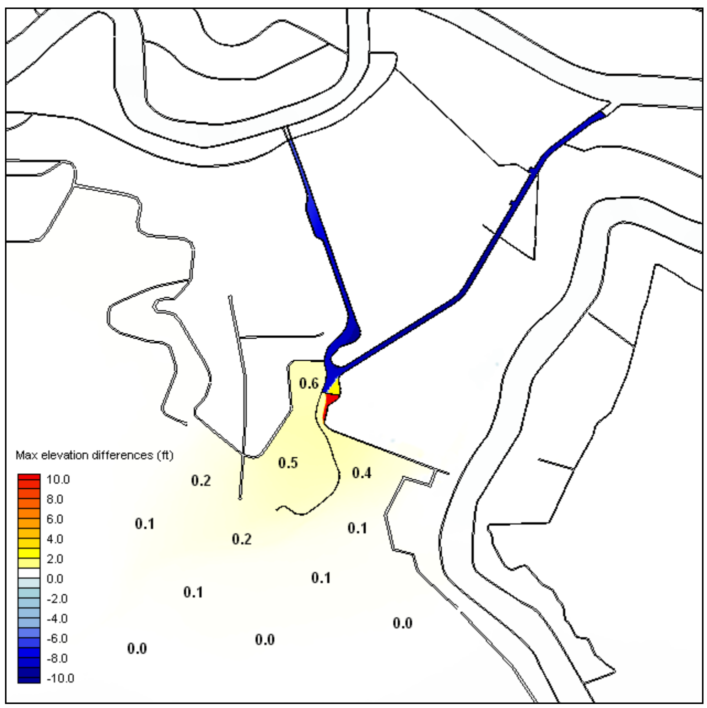
Maximum Surge Differences (ft) for Storm 003



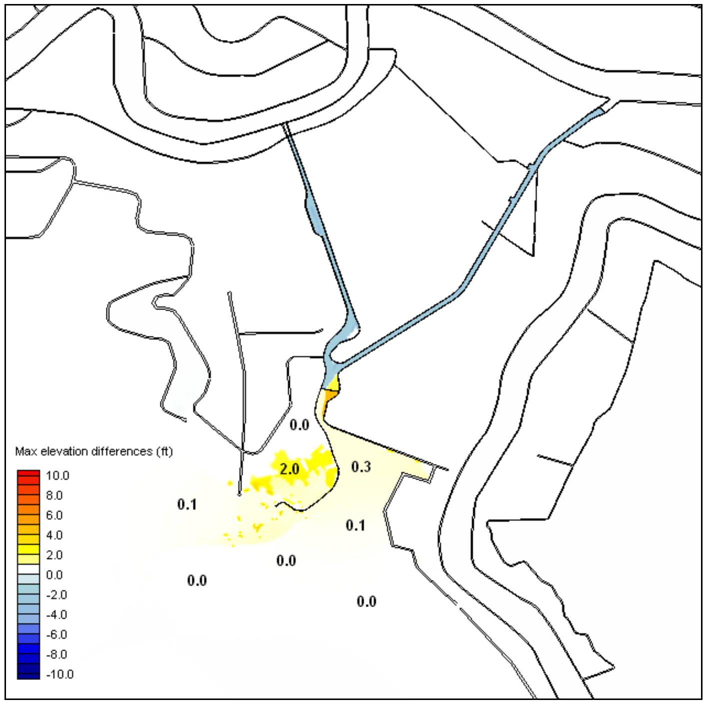
Maximum Surge Differences (ft) for Storm 006



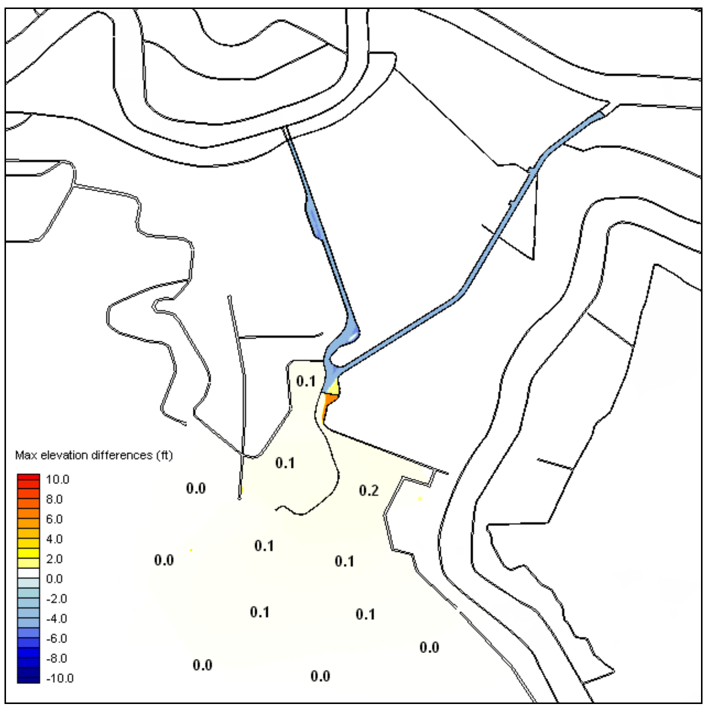
Maximum Surge Differences (ft) for Storm 008



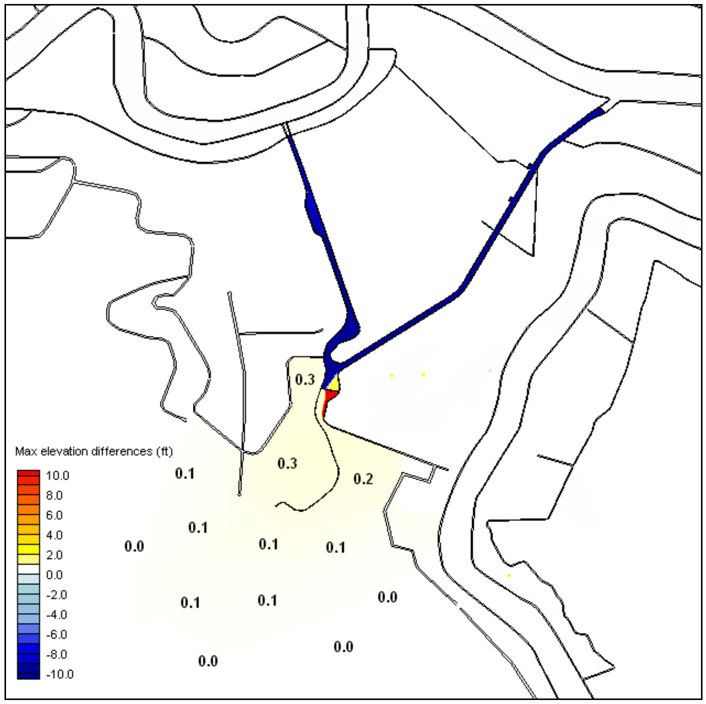
Maximum Surge Differences (ft) for Storm 017



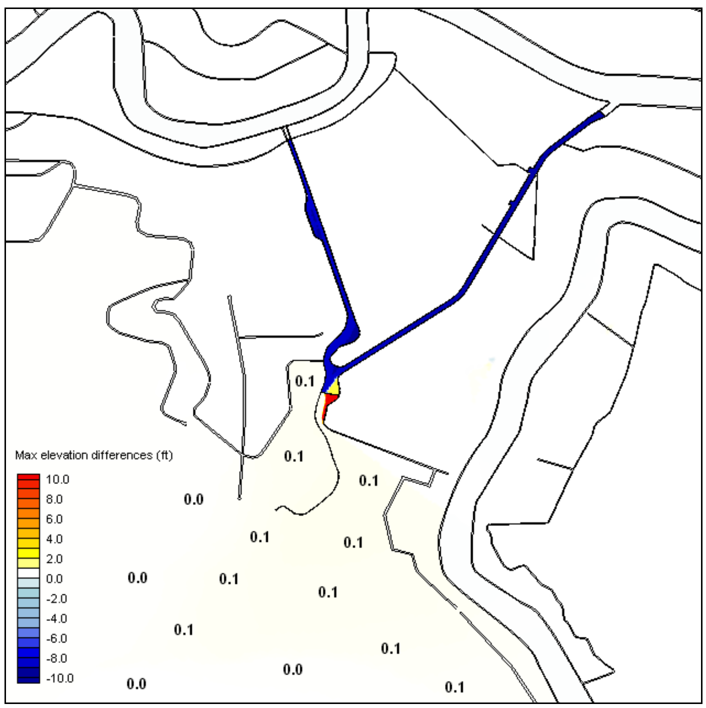
Maximum Surge Differences (ft) for Storm 050



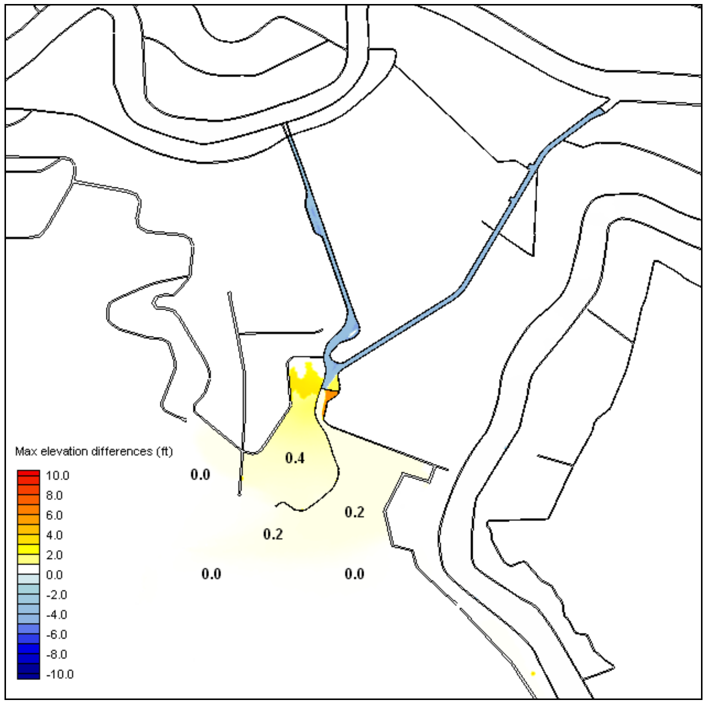
Maximum Surge Differences (ft) for Storm 066



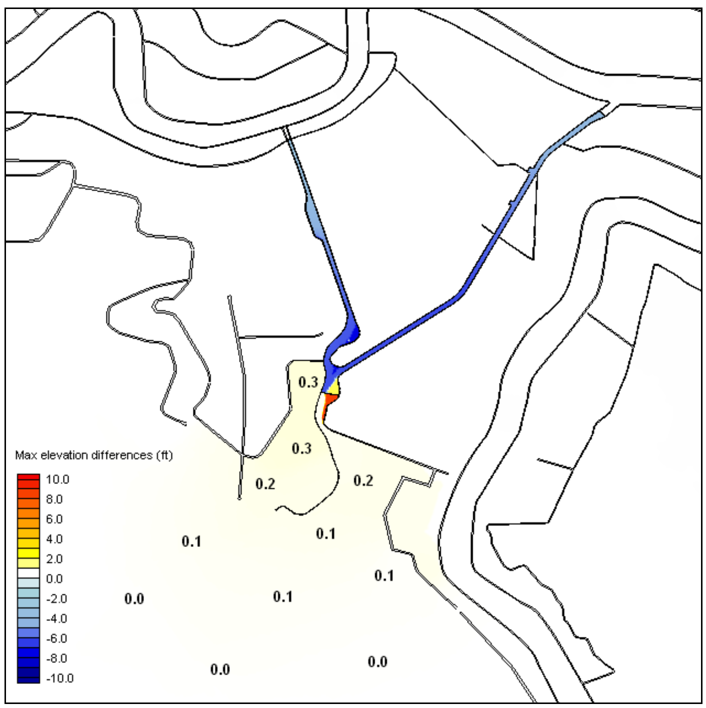
Maximum Surge Differences (ft) for Storm 069



Maximum Surge Differences (ft) for Storm 083

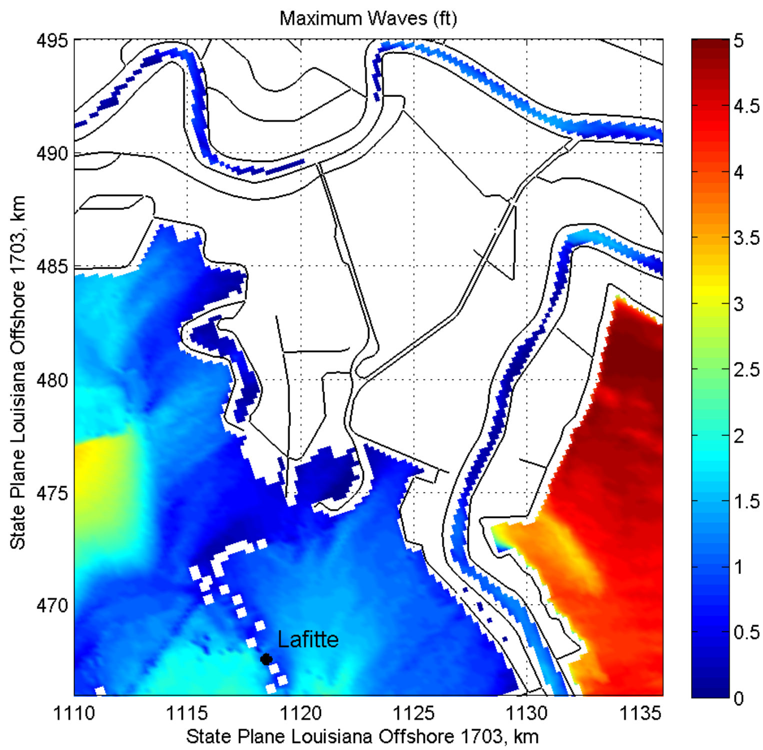


Maximum Surge Differences (ft) for Storm 101

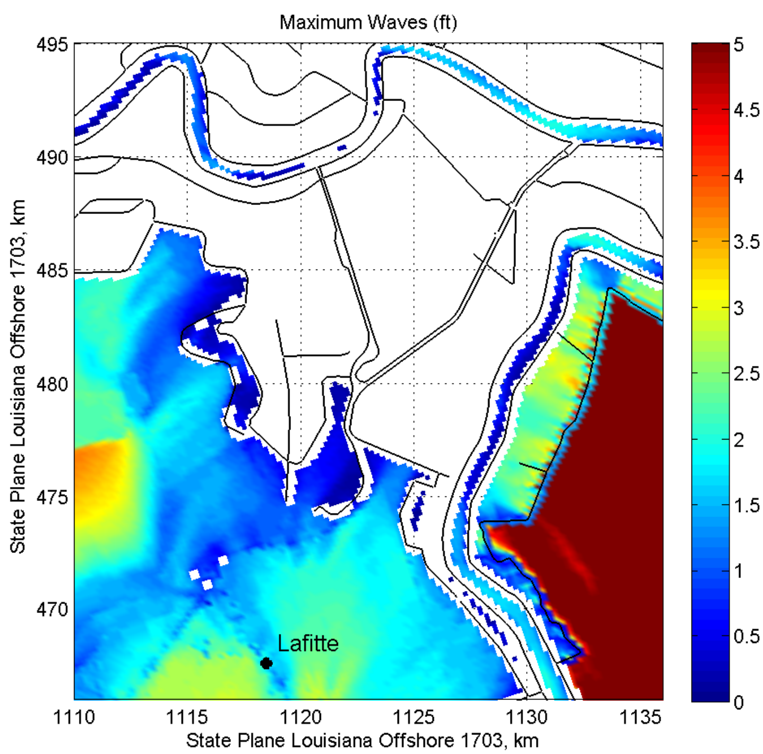


Maximum Surge Differences (ft) for Storm 160

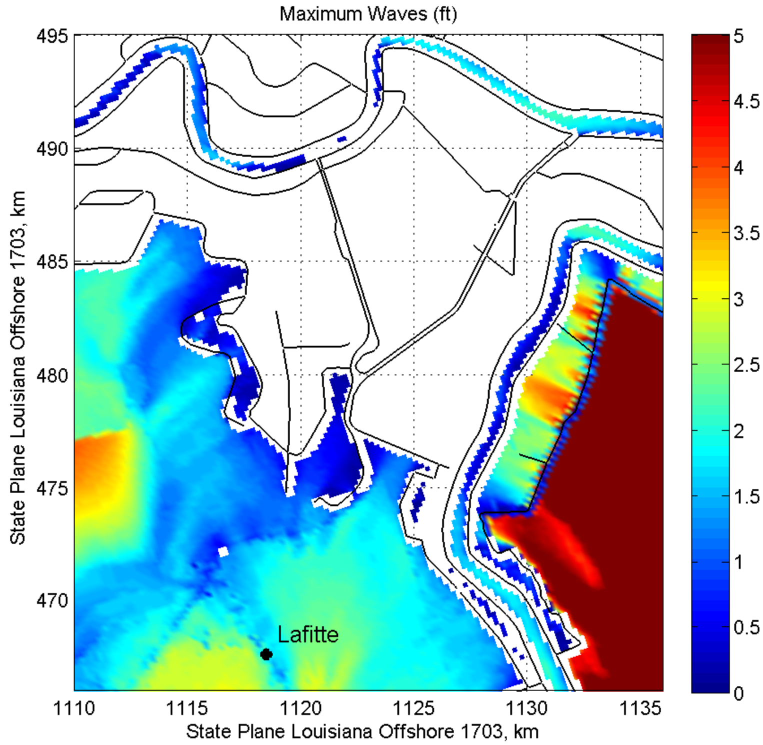
Appendix D: Maximum Waves Figures for the Base Condition (2010)



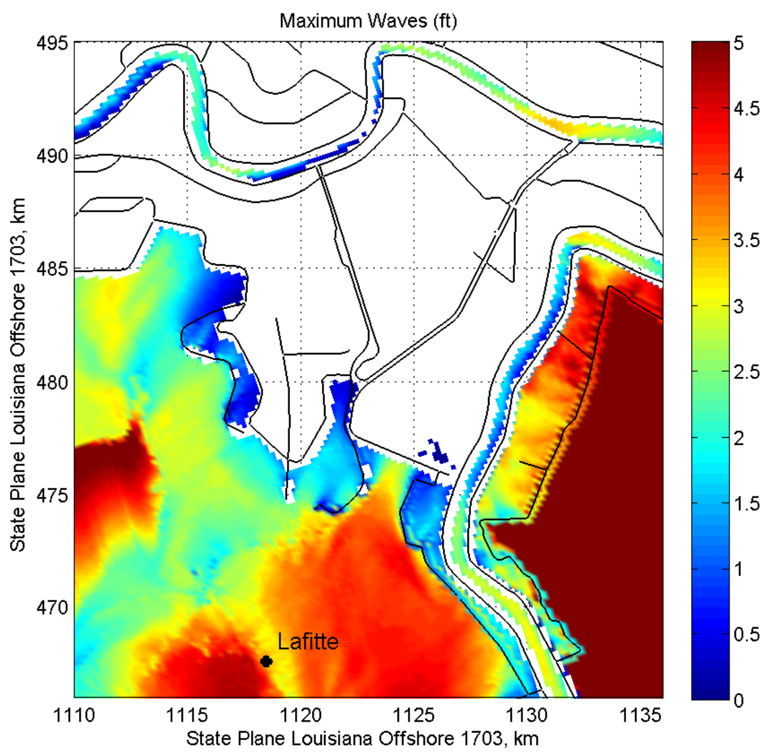
Maximum Waves (ft NAVD88 2004.65) for Storm 003



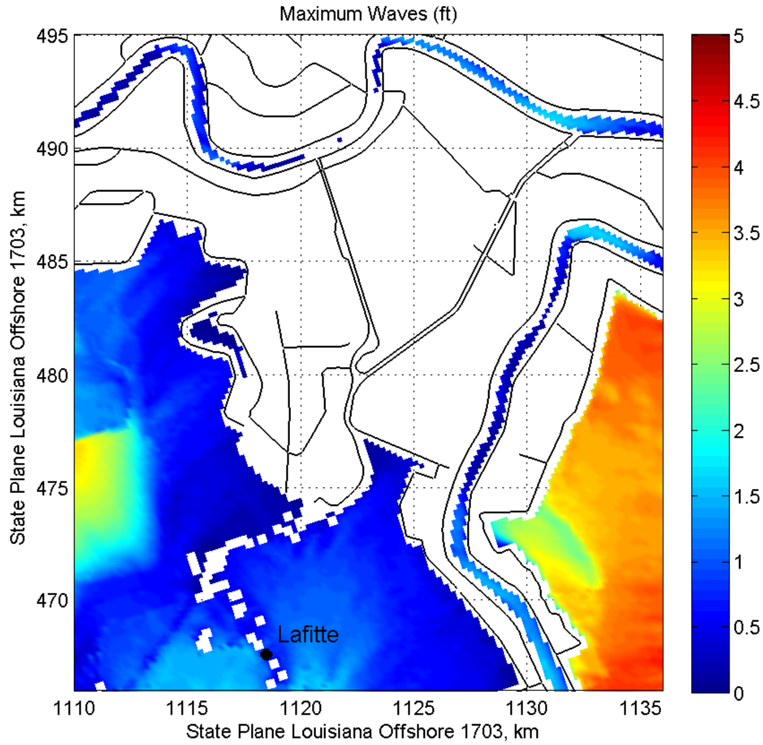
Maximum Waves (ft NAVD88 2004.65) for Storm 006



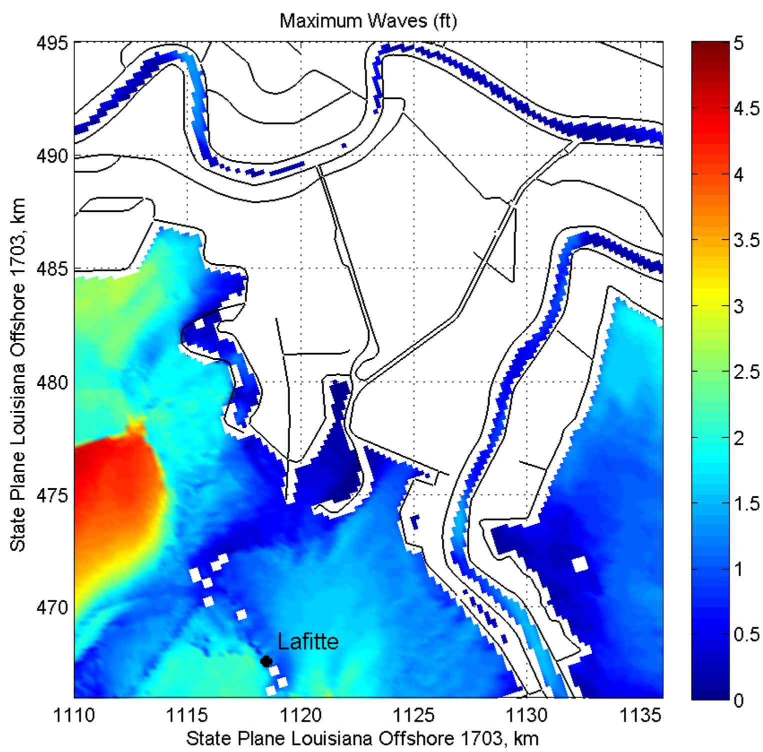
Maximum Waves (ft NAVD88 2004.65) for Storm 008



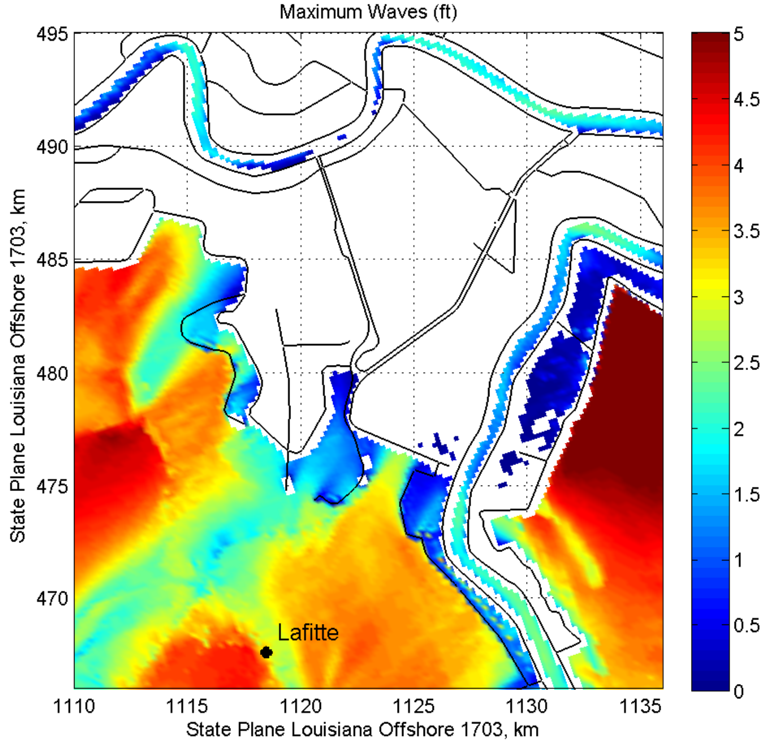
Maximum Waves (ft NAVD88 2004.65) for Storm 017



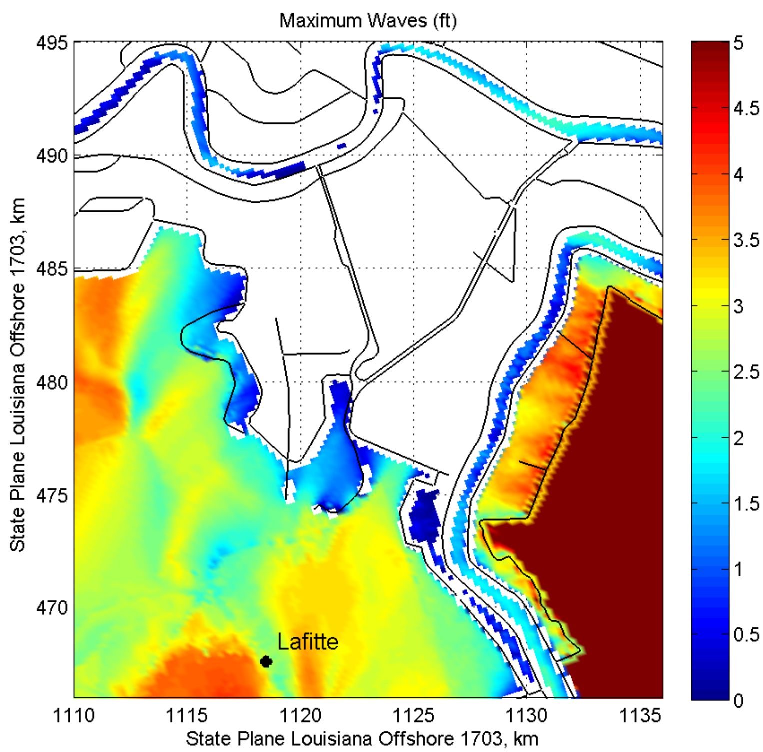
Maximum Waves (ft NAVD88 2004.65) for Storm 050



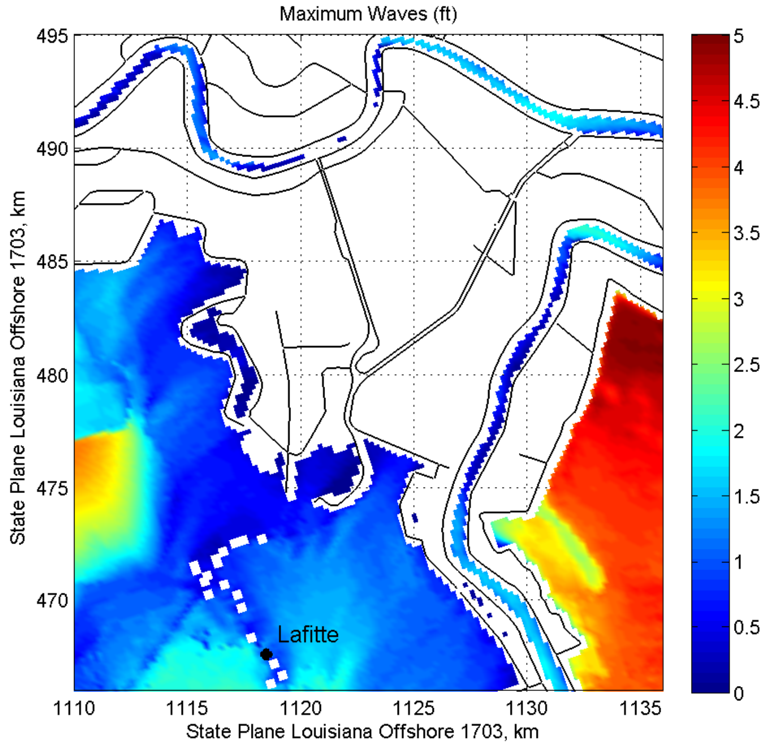
Maximum Waves (ft NAVD88 2004.65) for Storm 066



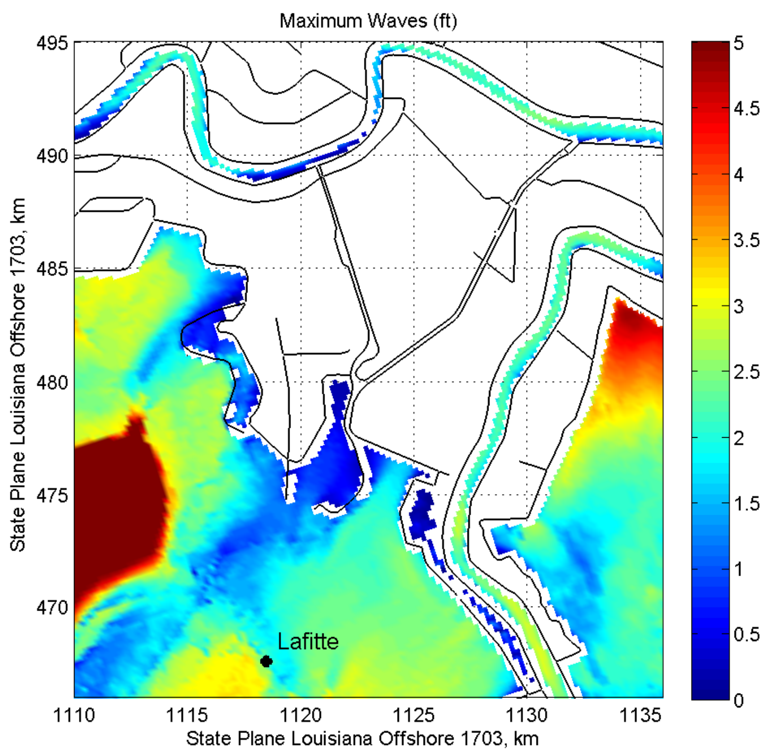
Maximum Waves (ft NAVD88 2004.65) for Storm 069



Maximum Waves (ft NAVD88 2004.65) for Storm 083

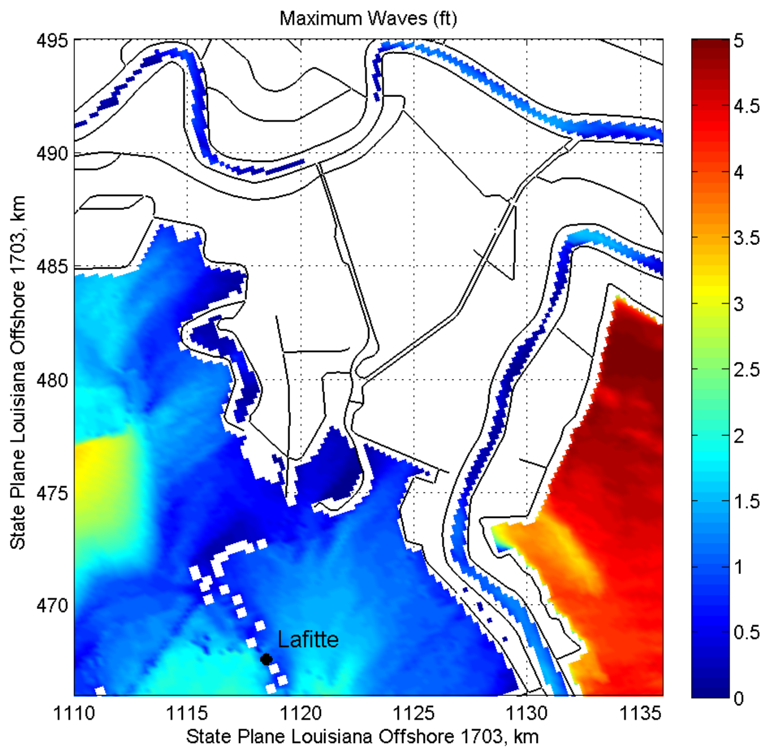


Maximum Waves (ft NAVD88 2004.65) for Storm 101

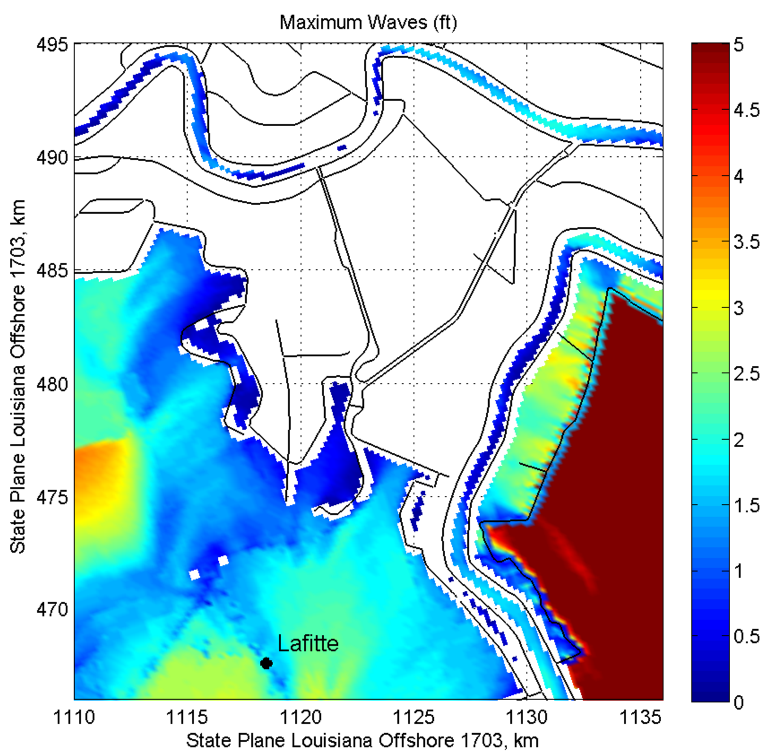


Maximum Waves (ft NAVD88 2004.65) for Storm 160

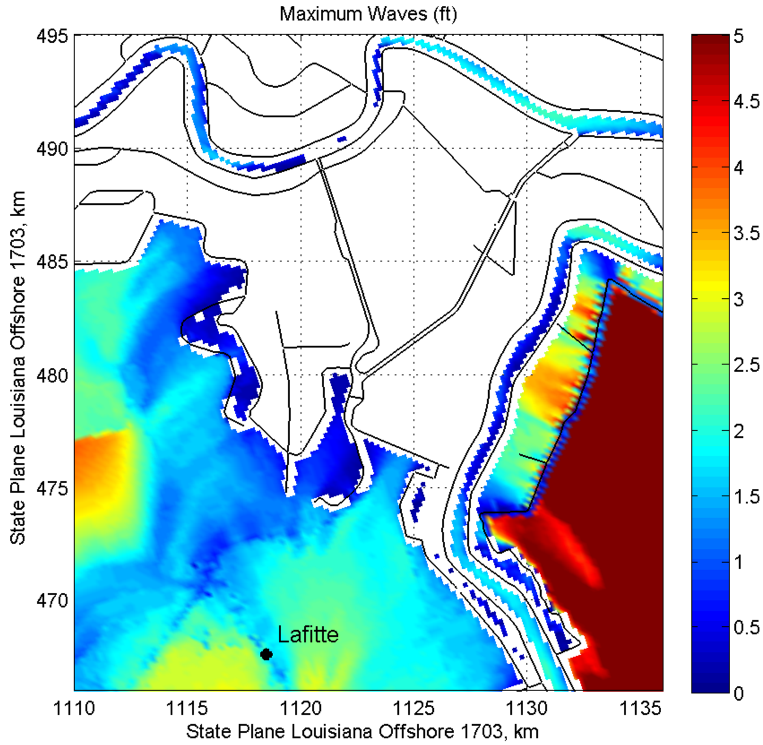
Appendix E: Maximum Waves Figures for the With-Project Condition (WCC)



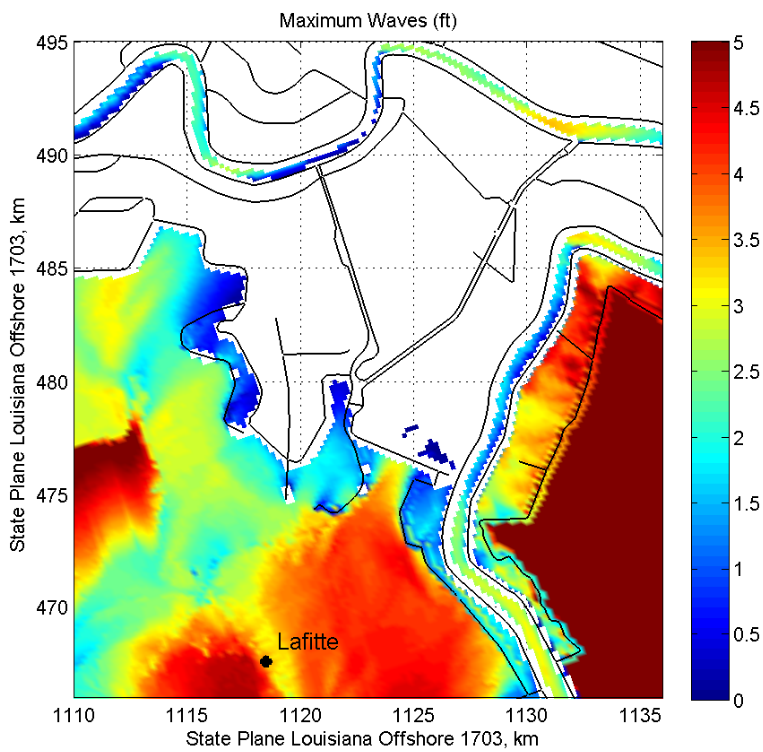
Maximum Waves (ft NAVD88 2004.65) for Storm 003



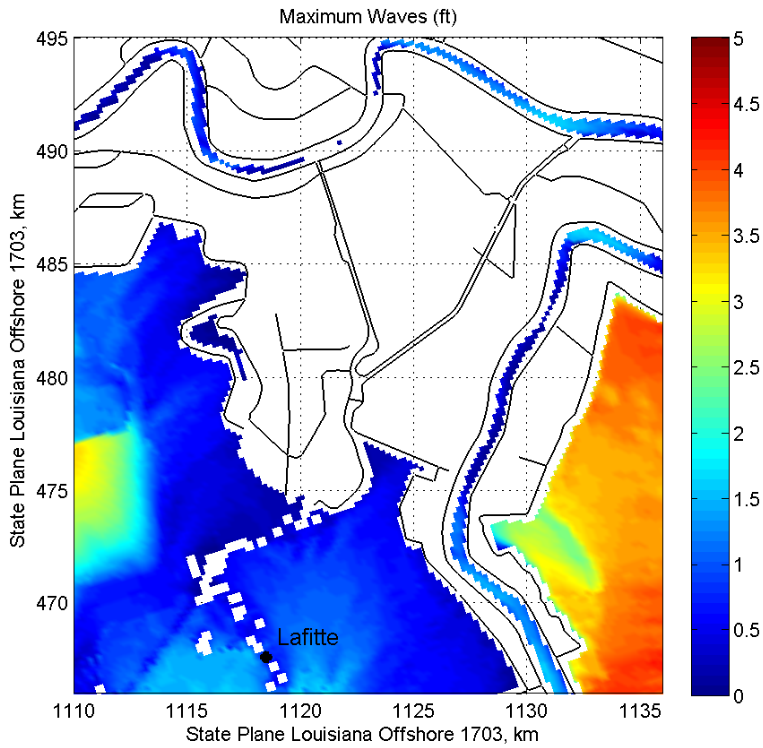
Maximum Waves (ft NAVD88 2004.65) for Storm 006



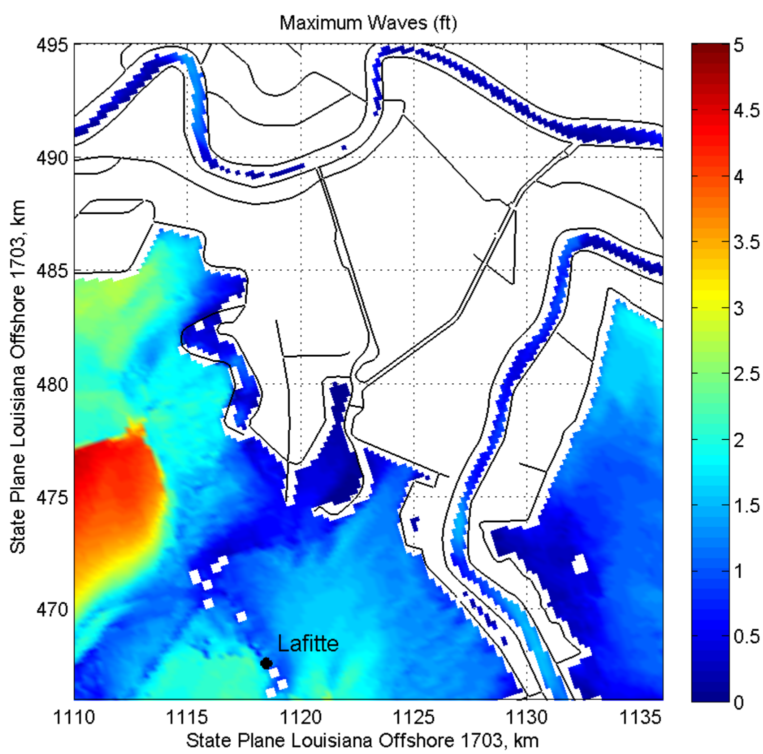
Maximum Waves (ft NAVD88 2004.65) for Storm 008



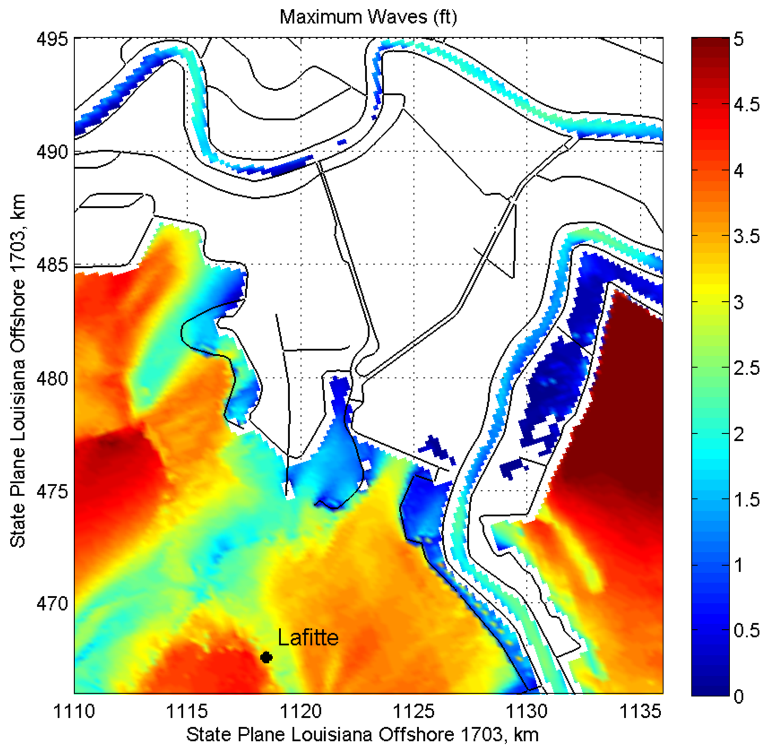
Maximum Waves (ft NAVD88 2004.65) for Storm 017



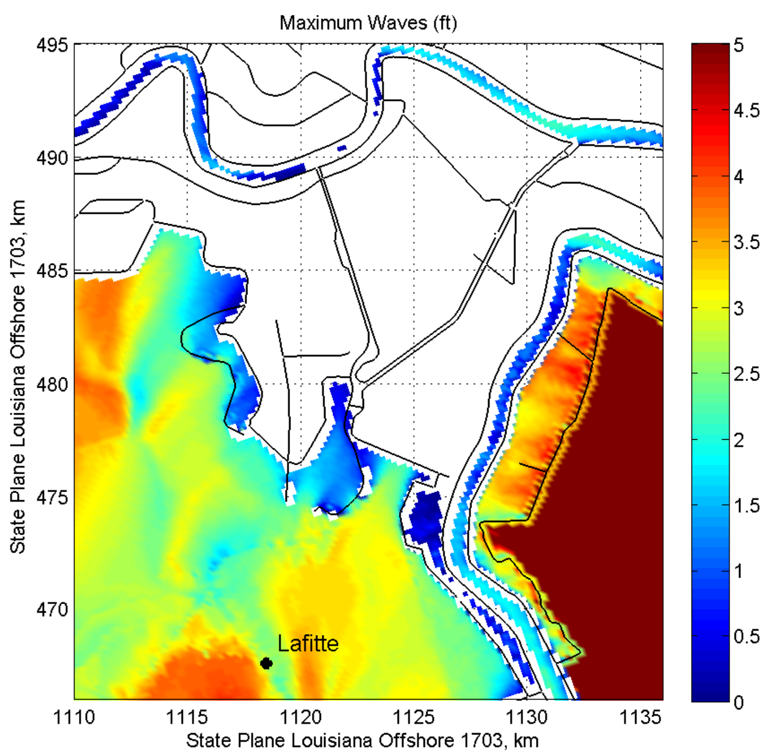
Maximum Waves (ft NAVD88 2004.65) for Storm 050



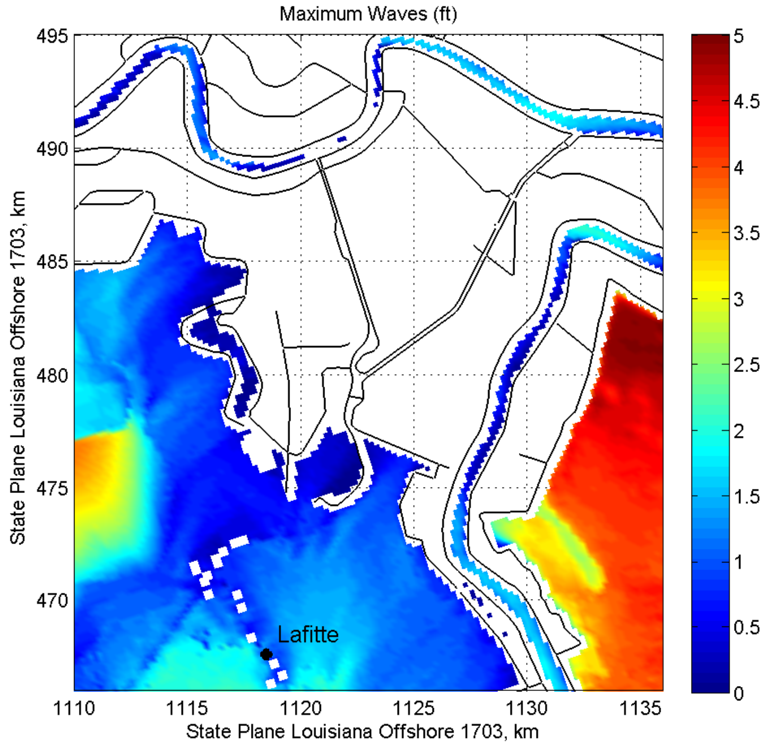
Maximum Waves (ft NAVD88 2004.65) for Storm 066



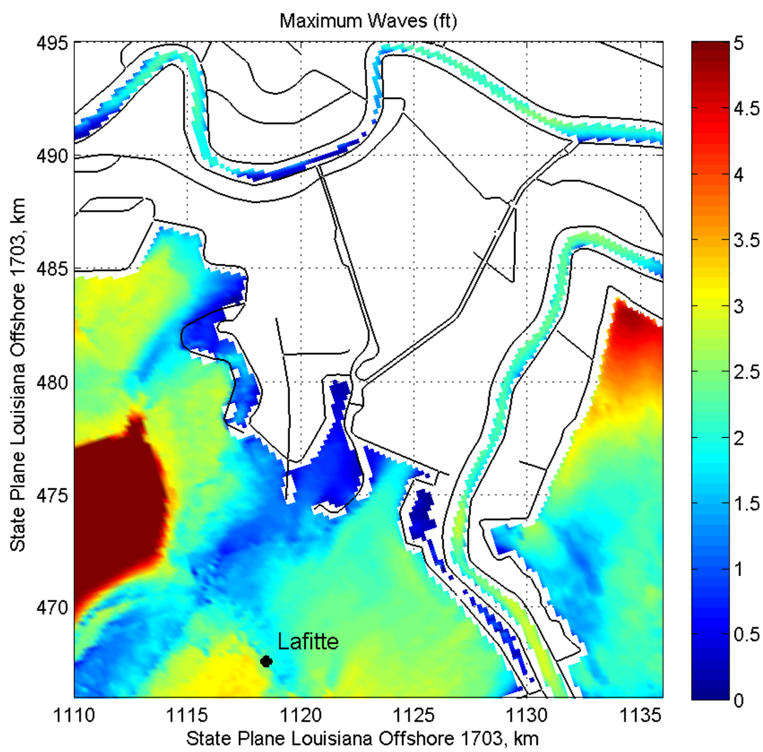
Maximum Waves (ft NAVD88 2004.65) for Storm 069



Maximum Waves (ft NAVD88 2004.65) for Storm 083

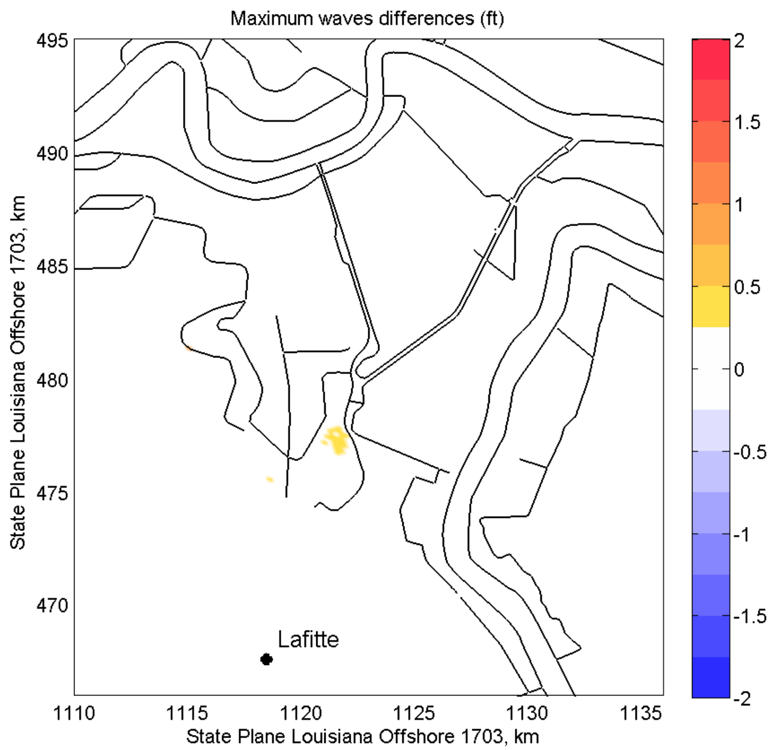


Maximum Waves (ft NAVD88 2004.65) for Storm 101

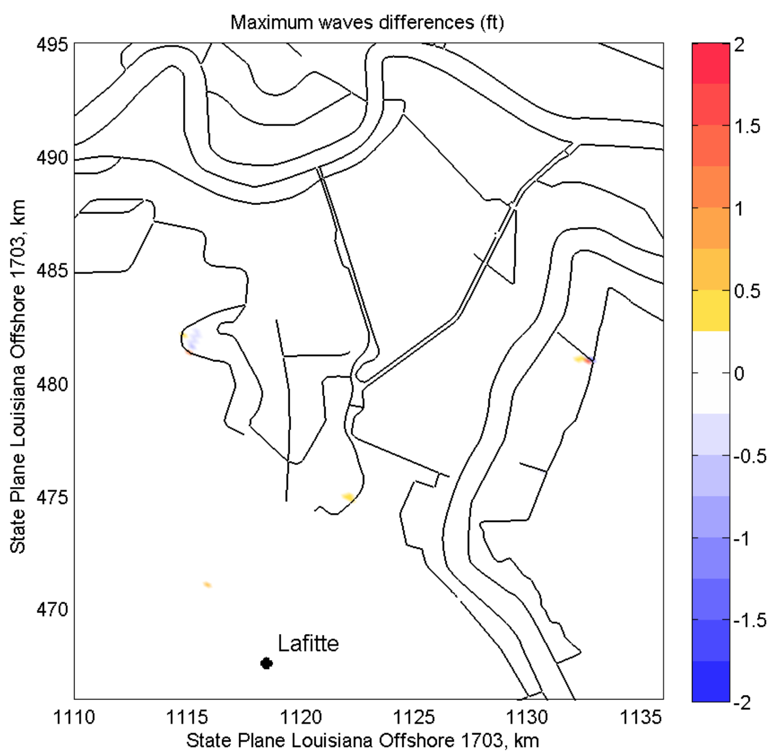


Maximum Waves (ft NAVD88 2004.65) for Storm 160

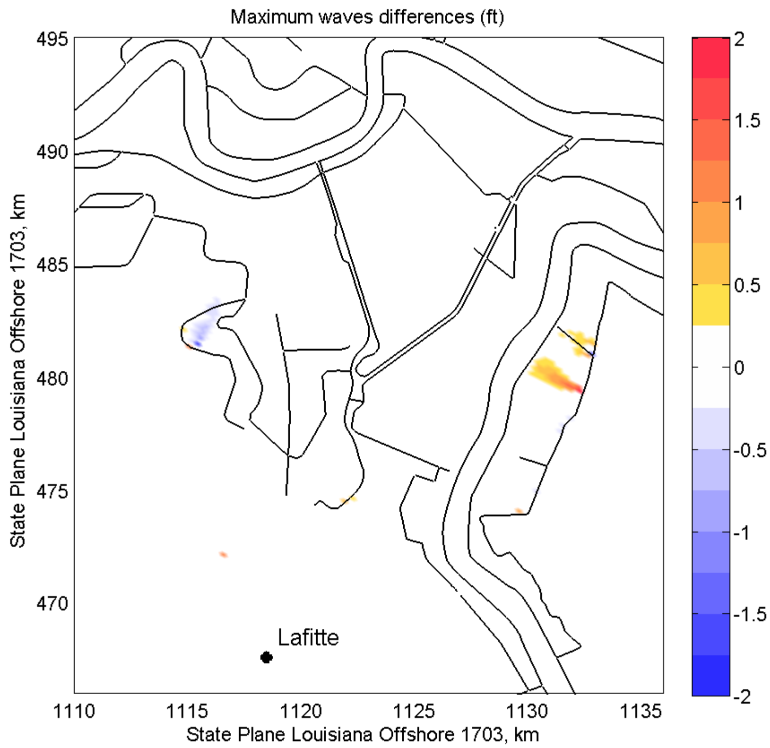
Appendix F: Maximum Waves Difference Figures for WCC minus 2010



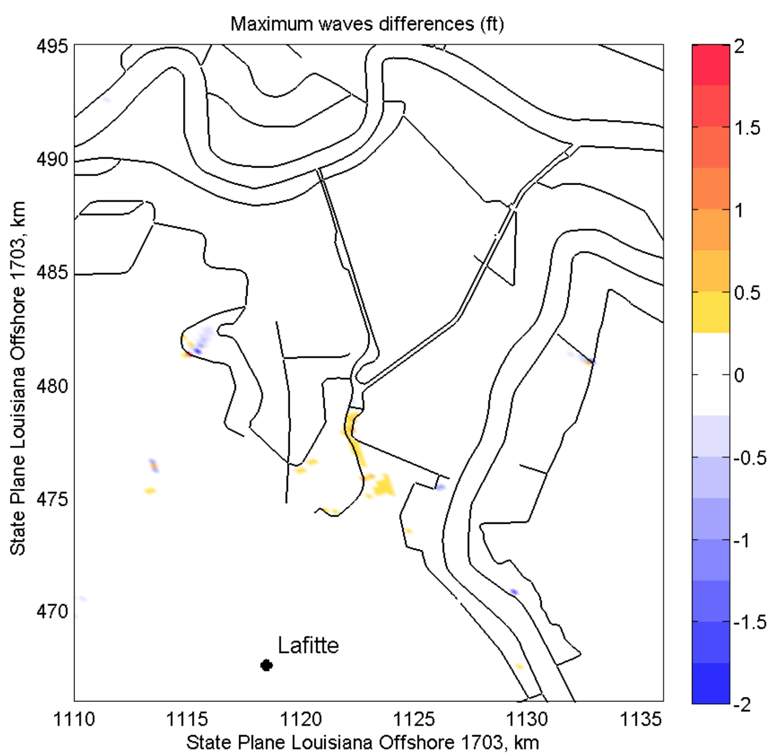
Maximum Waves Differences (ft) for Storm 003



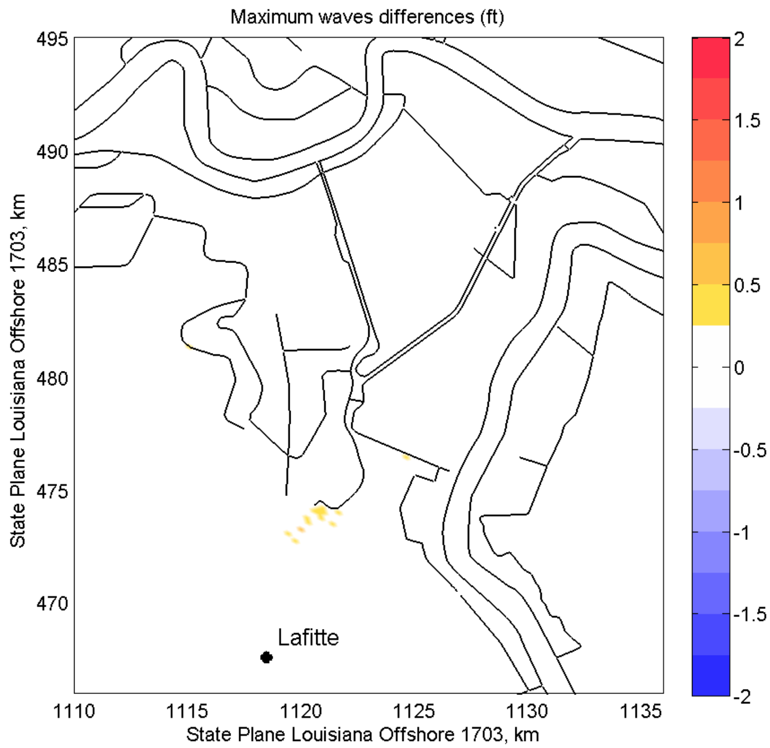
Maximum Waves Differences (ft) for Storm 006



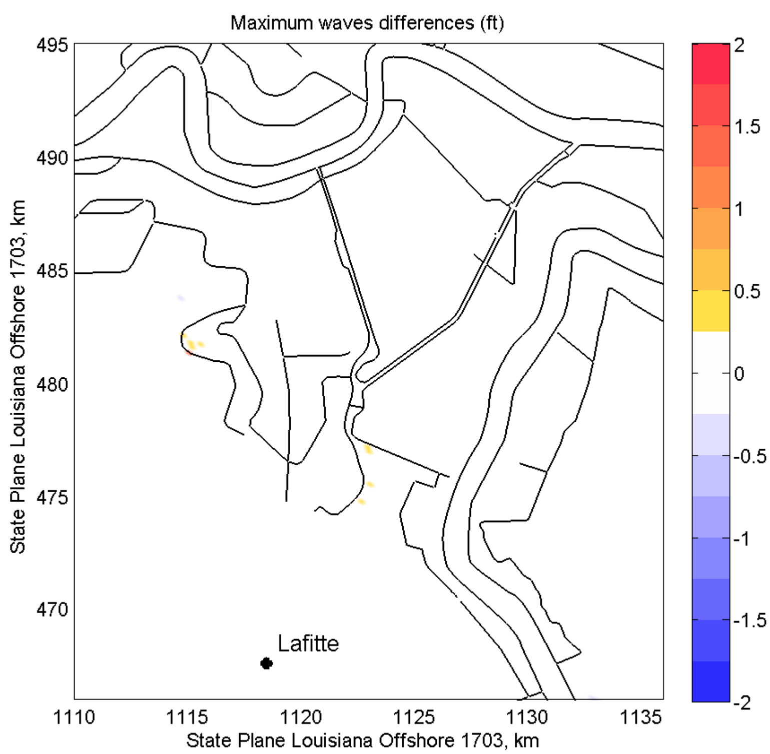
Maximum Waves Differences (ft) for Storm 008



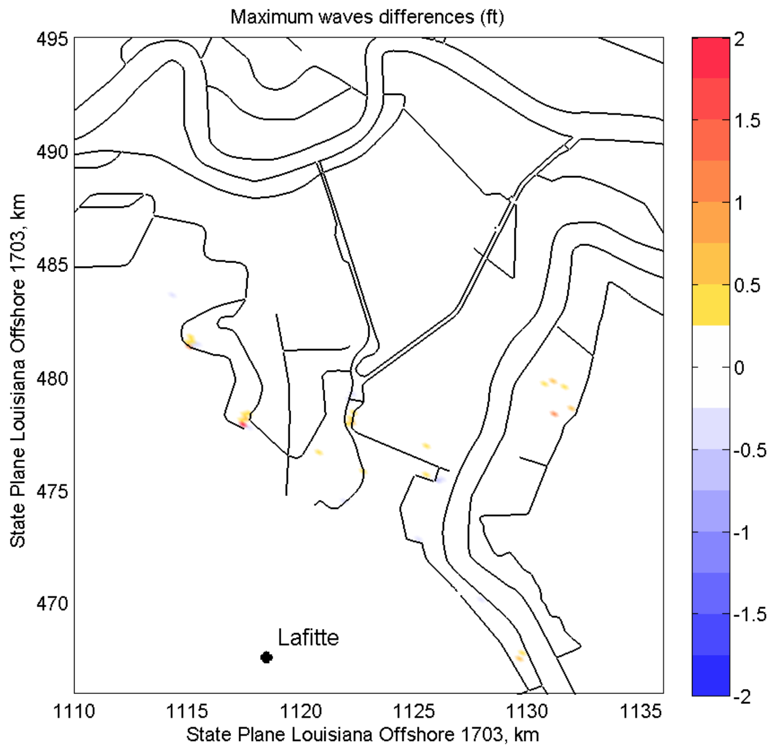
Maximum Waves Differences (ft) for Storm 017



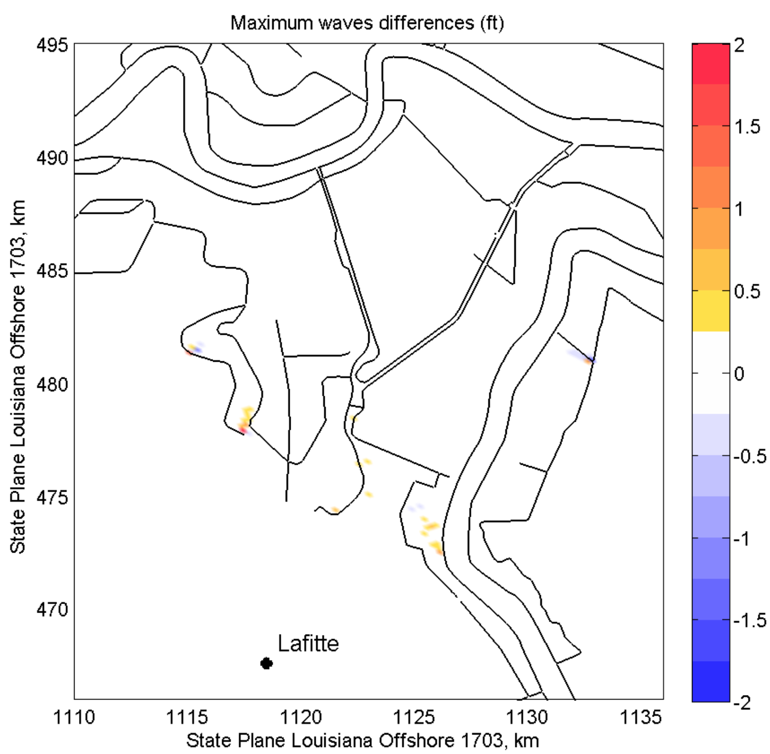
Maximum Waves Differences (ft) for Storm 050



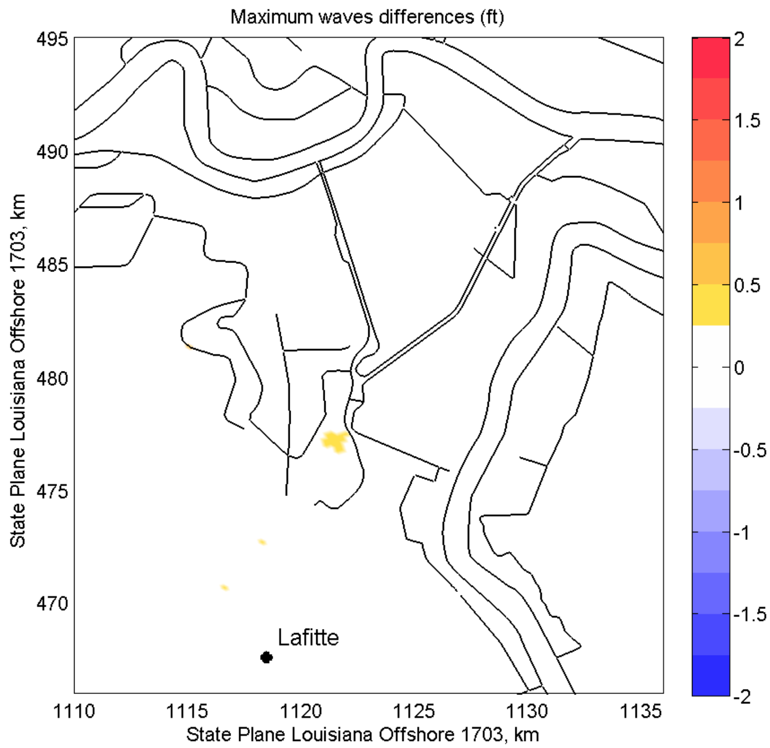
Maximum Waves Differences (ft) for Storm 066



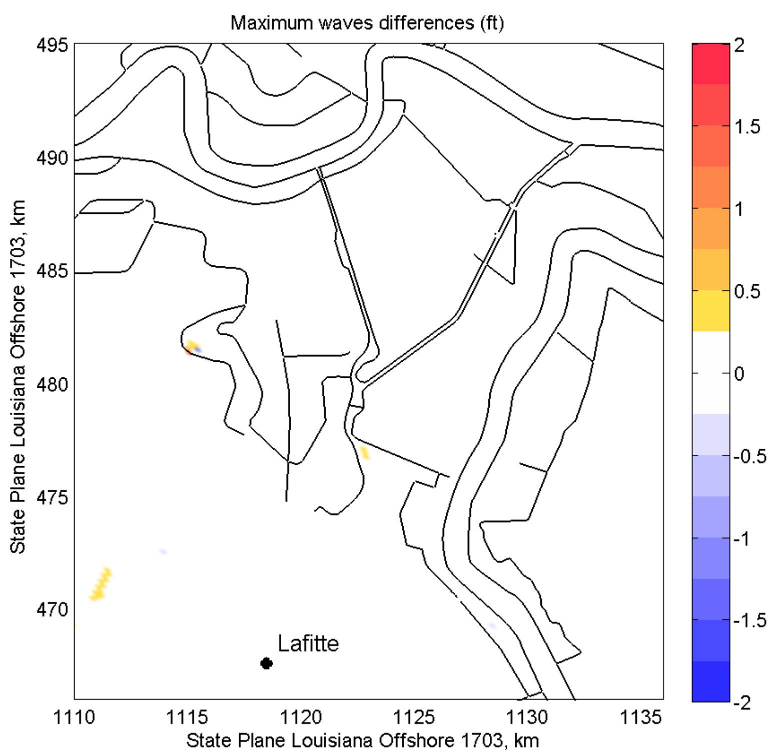
Maximum Waves Differences (ft) for Storm 069



Maximum Waves Differences (ft) for Storm 083



Maximum Waves Differences (ft) for Storm 101



Maximum Waves Differences (ft) for Storm 160

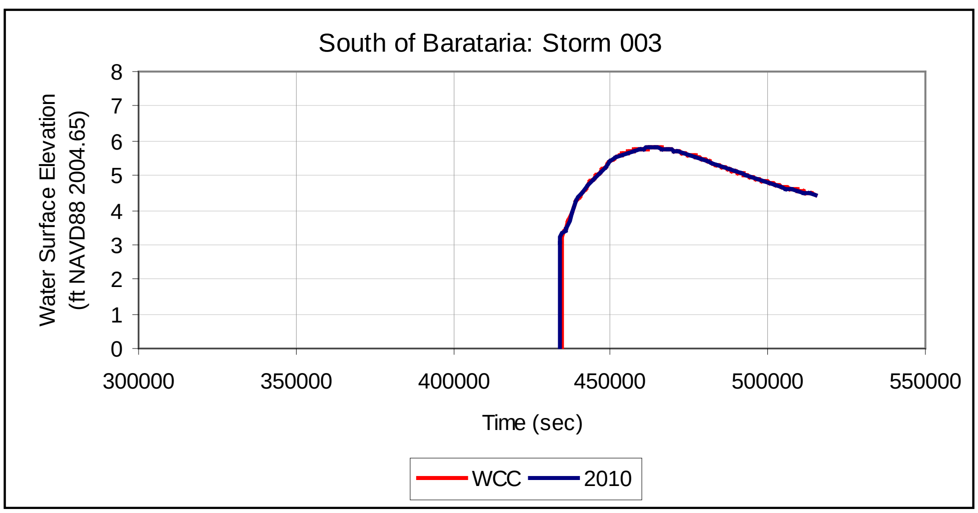
Appendix G: Time Series Plots

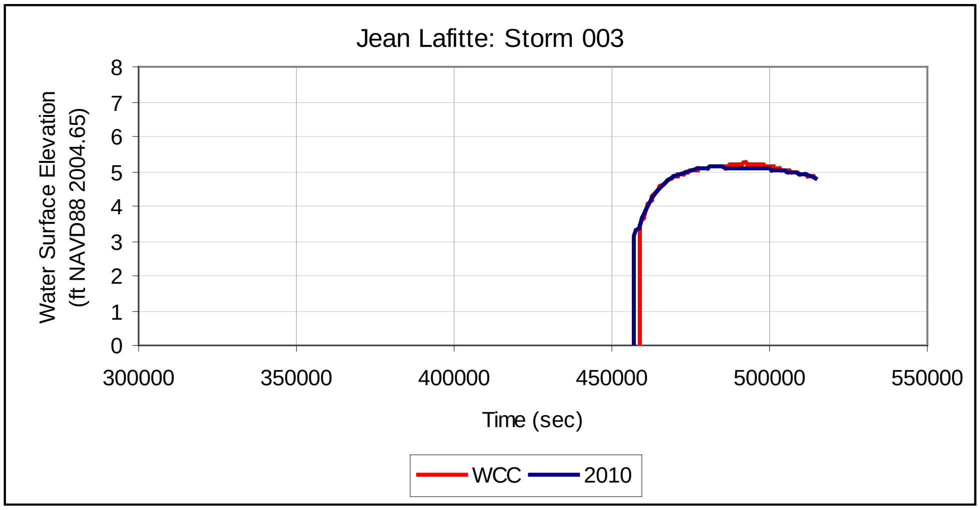
Notes regarding the time series plots:

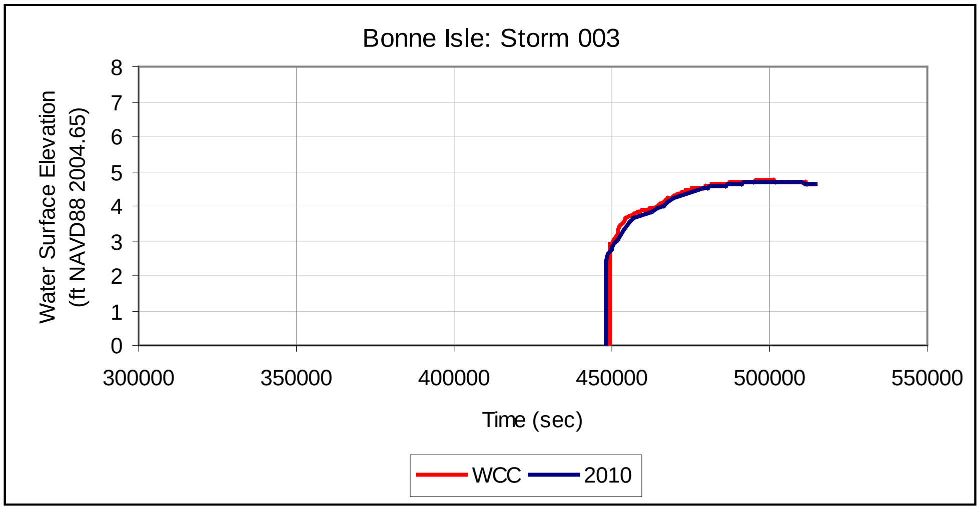
1) All of the save locations, except for Barataria, are location in land areas that are initially "dry." The save location Barataria is located within the Barataria Bay Waterway and is initially "wet." This explains why the water surface elevation time series figures show an initial non-zero value for Barataria while the other locations become instantaneously inundated at some point during the storm simulation.

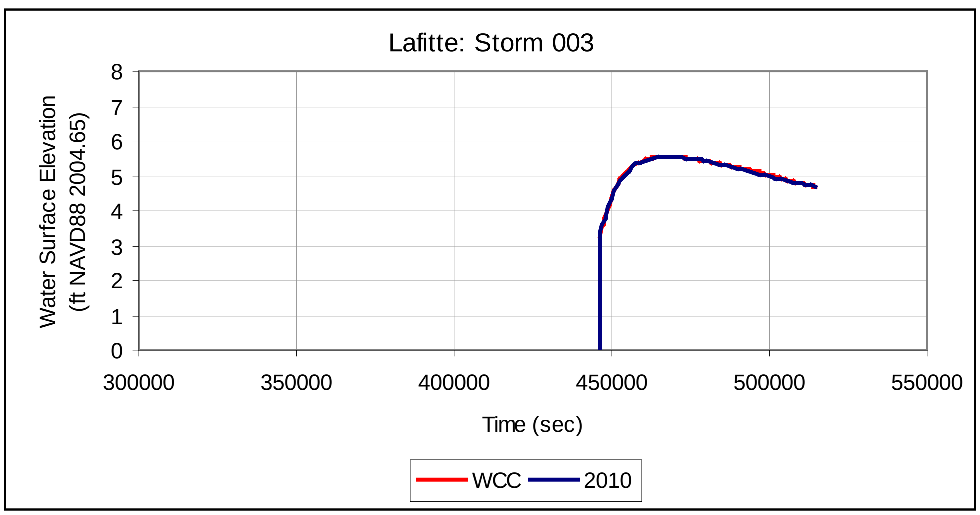
2) Jean Lafitte appears to become inundated at different times for some storms when comparing the with-project versus without-project conditions due to interpolation differences with regards to data extraction methods. The without-project (base/2010) condition was run for a prior study (LACPR/FEMA) and the "nearest node method" was used to extract water surface elevation data for the seven save locations. For the with-project (WCC) condition, the save locations were known a priori and setup within the ADCIRC model prior to making the simulations. Hence, the ADCIRC model code internally calculated the water surface elevations via interpolating among the nearest nodes. The latter data extraction method yields data at the exact latitude/longitude of the save locations whereas the former method yields data at the nearest node latitude/longitude.

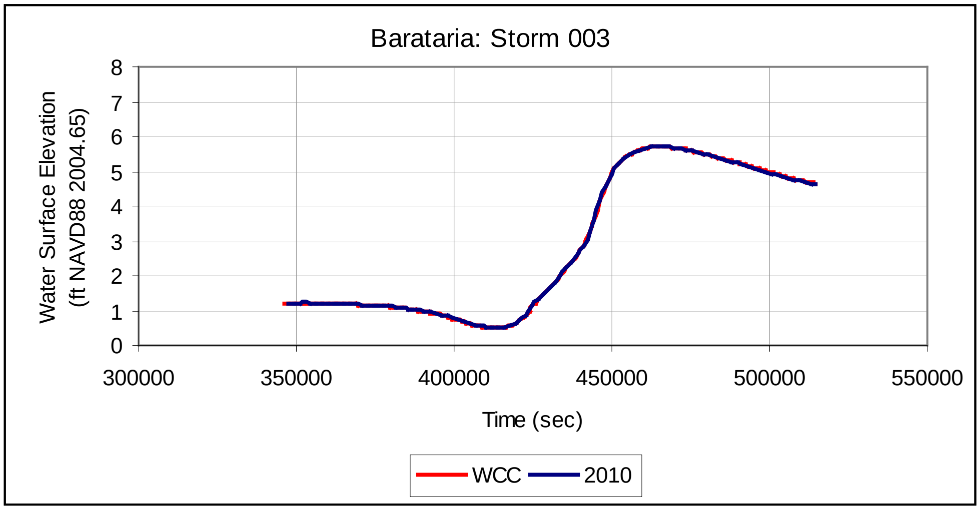
Storm 003

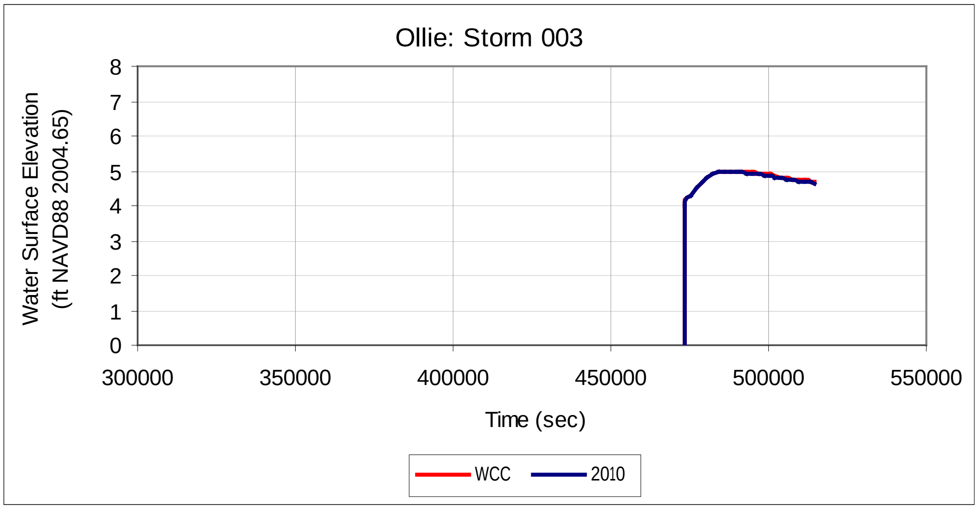



Jean Lafitte: Storm 003
Water Surface Elevation (ft NAVD88 2004.65)
Time (sec)
WCC
2010

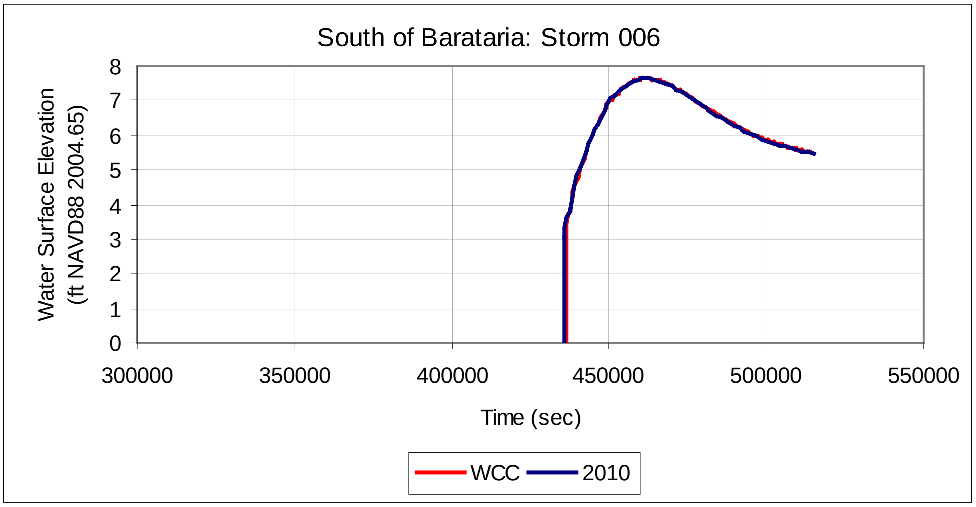

Bonne Isle: Storm 003
Water Surface Elevation (ft NAVD88 2004.65)
Time (sec)
WCC
2010


Lafitte: Storm 003
Water Surface Elevation (ft NAVD88 2004.65)
Time (sec)
WCC
2010

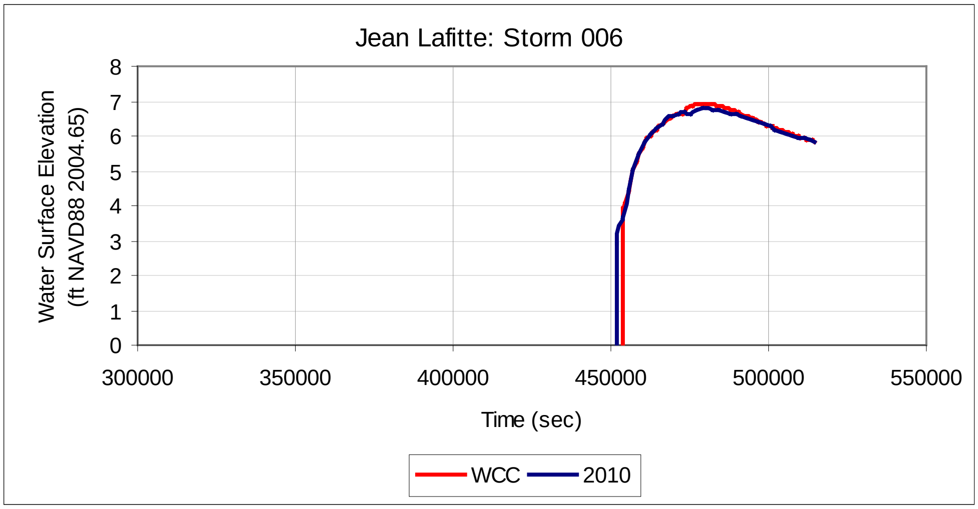

Barataria: Storm 003
Water Surface Elevation
(ft NAVD88 2004.65)
8
7
6
5
4
3
2
1
0
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010


Ollie: Storm 003
Water Surface Elevation
(ft NAVD88 2004.65)
8
7
6
5
4
3
2
1
0
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

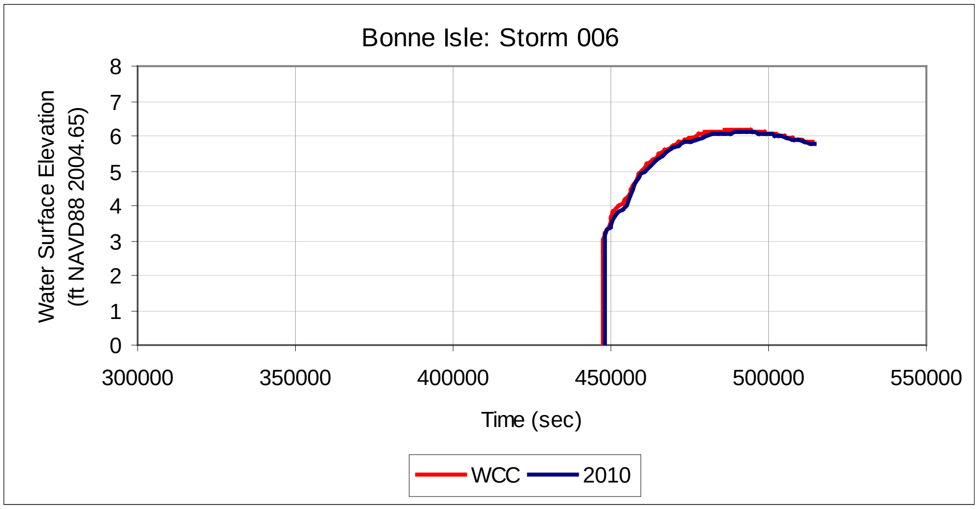
Storm 006



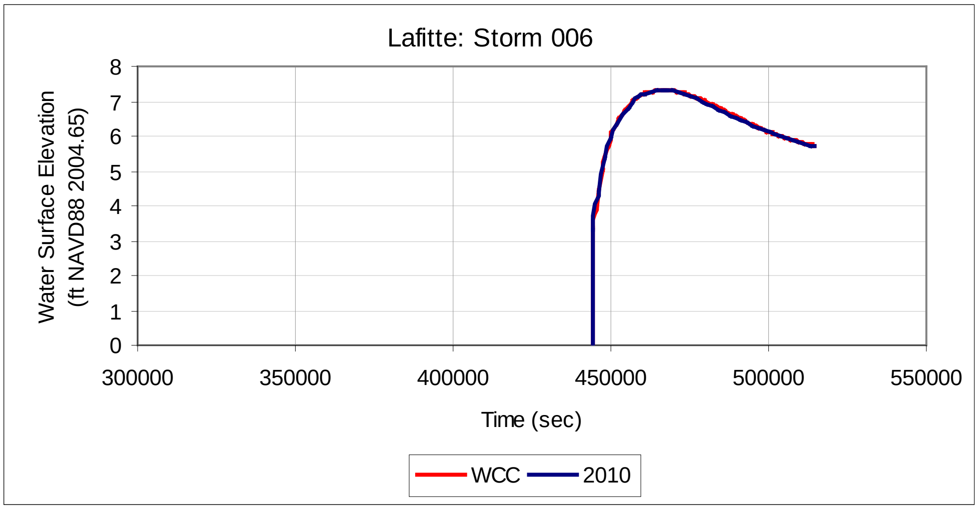
South of Barataria: Storm 006
Water Surface Elevation
(ft NAVD88 2004.65)
8
7
6
5
4
3
2
1
0
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

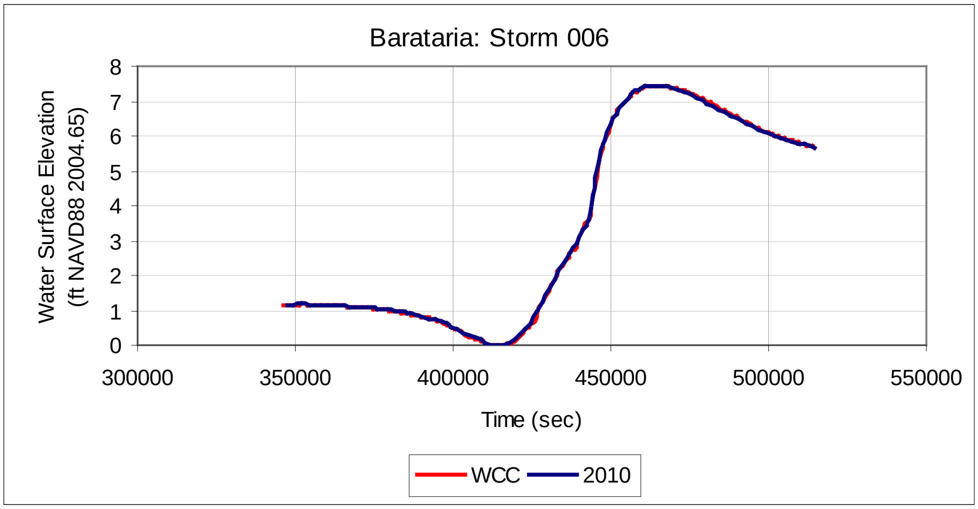


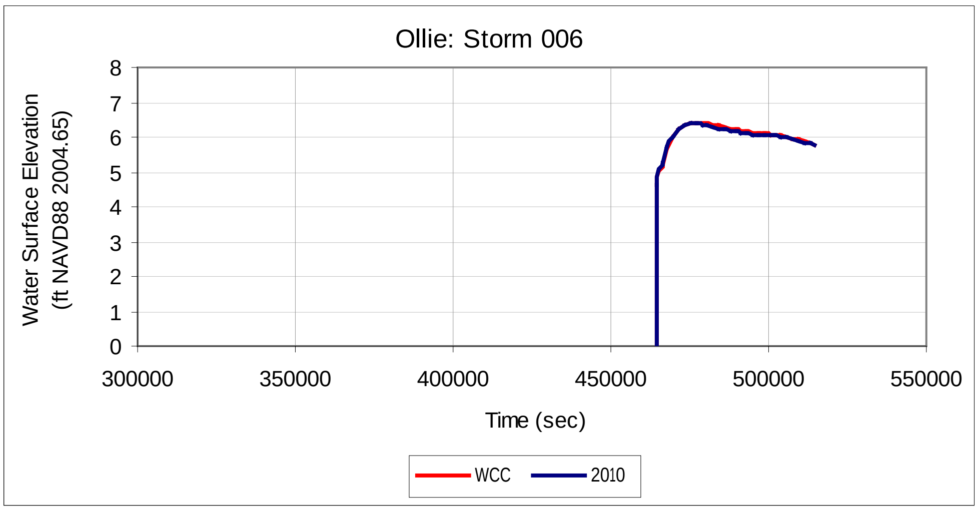
Jean Lafitte: Storm 006
Water Surface Elevation
(ft NAVD88 2004.65)
8
7
6
5
4
3
2
1
0
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010



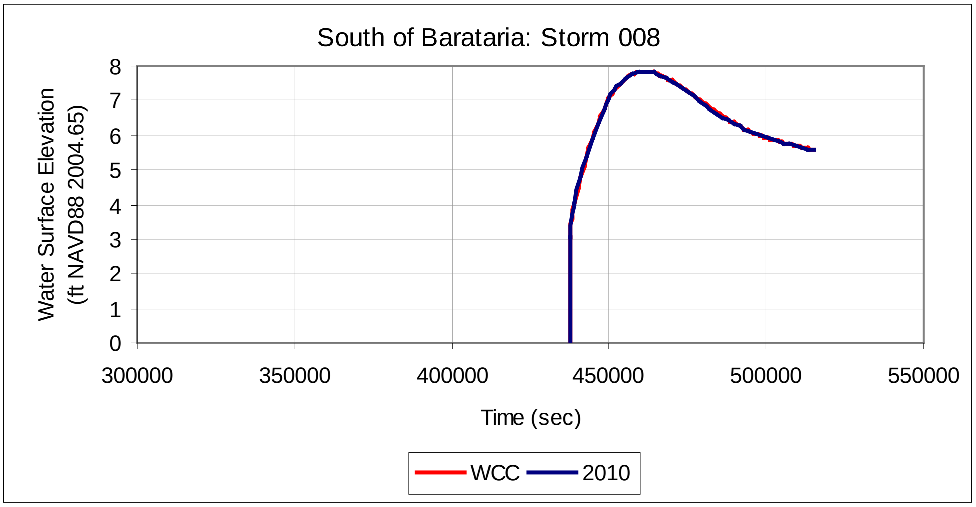
Bonne Isle: Storm 006
Water Surface Elevation
(ft NAVD88 2004.65)
8
7
6
5
4
3
2
1
0
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

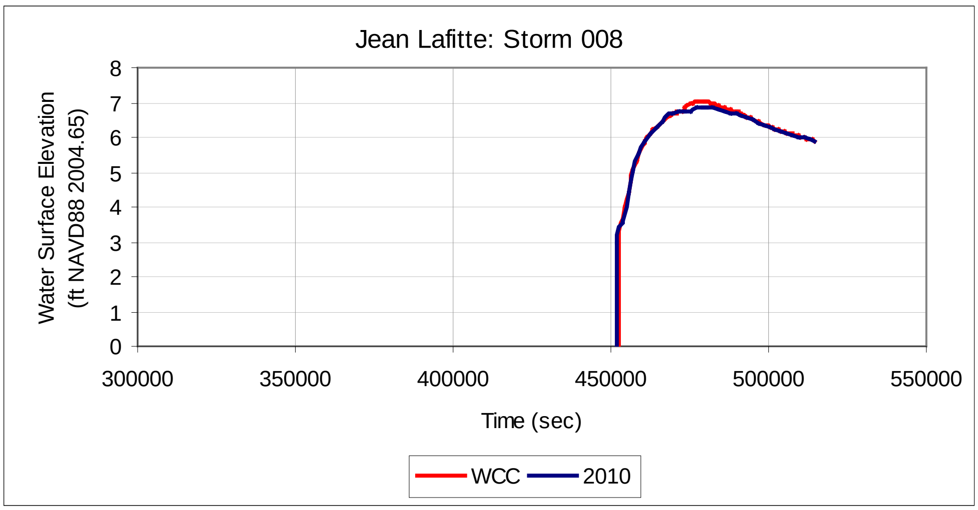

Lafitte: Storm 006
Water Surface Elevation
(ft NAVD88 2004.65)
0
1
2
3
4
5
6
7
8
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

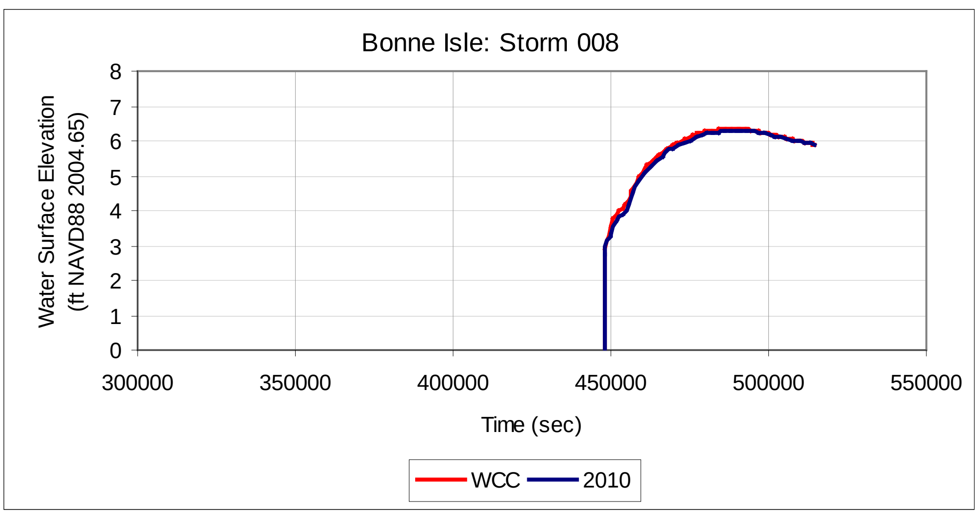

Barataria: Storm 006
Water Surface Elevation
(ft NAVD88 2004.65)
0
1
2
3
4
5
6
7
8
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

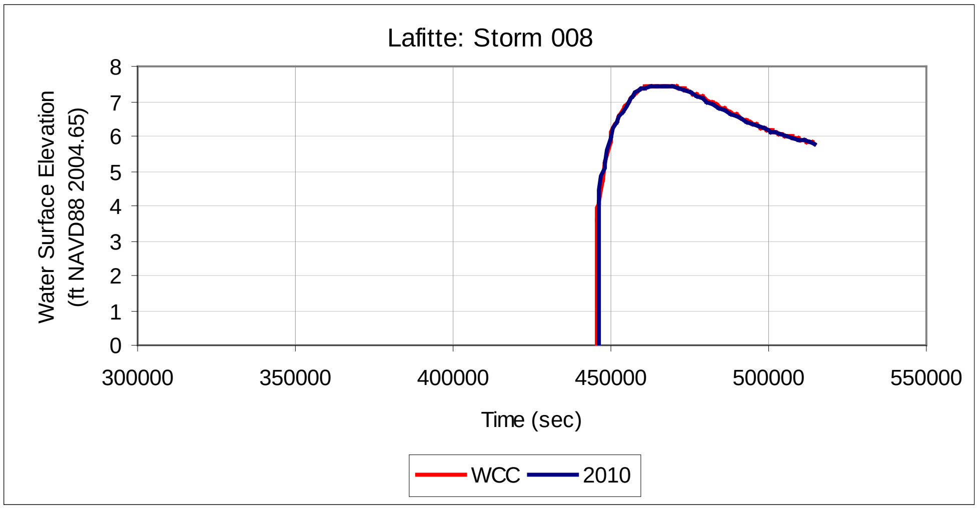

Ollie: Storm 006
Water Surface Elevation
(ft NAVD88 2004.65)
0
1
2
3
4
5
6
7
8
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

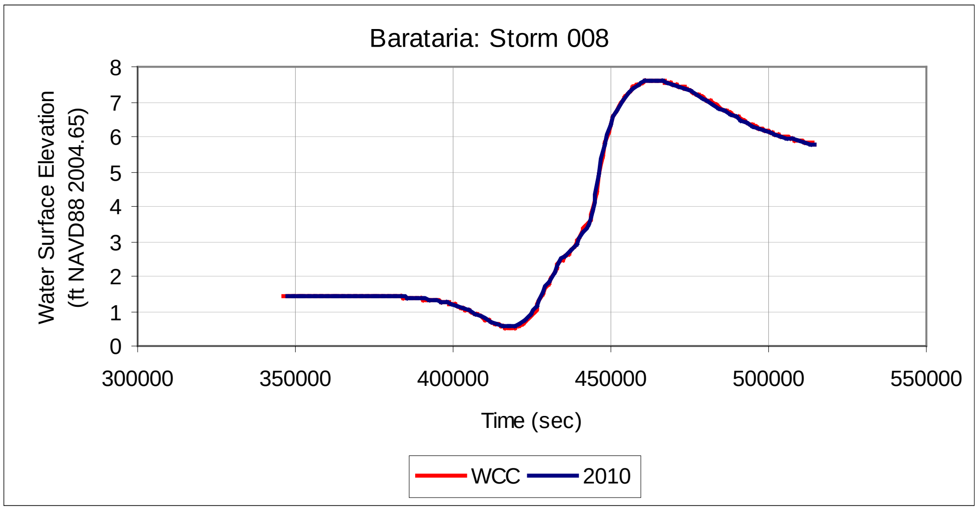
Storm 008

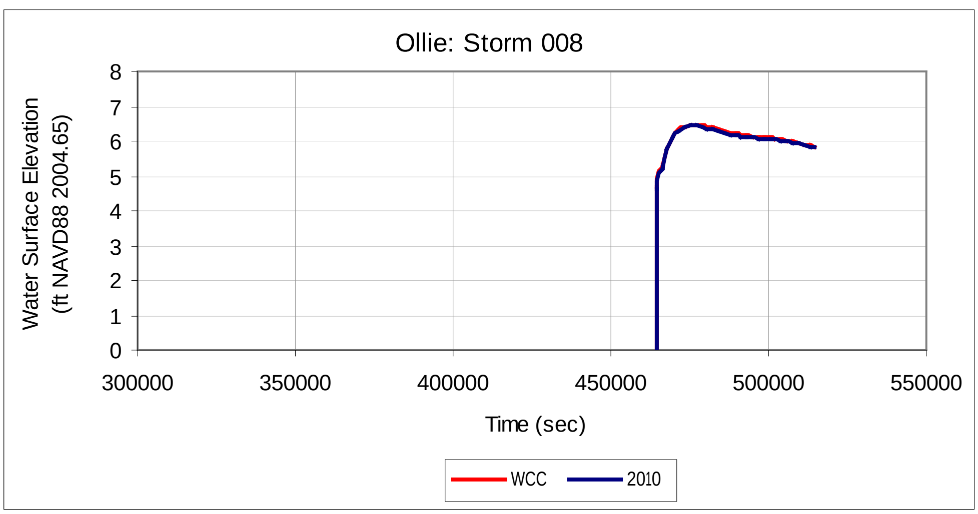

South of Barataria: Storm 008
Water Surface Elevation
(ft NAVD88 2004.65)
0
1
2
3
4
5
6
7
8
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010


Jean Lafitte: Storm 008
Water Surface Elevation
(ft NAVD88 2004.65)
0
1
2
3
4
5
6
7
8
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

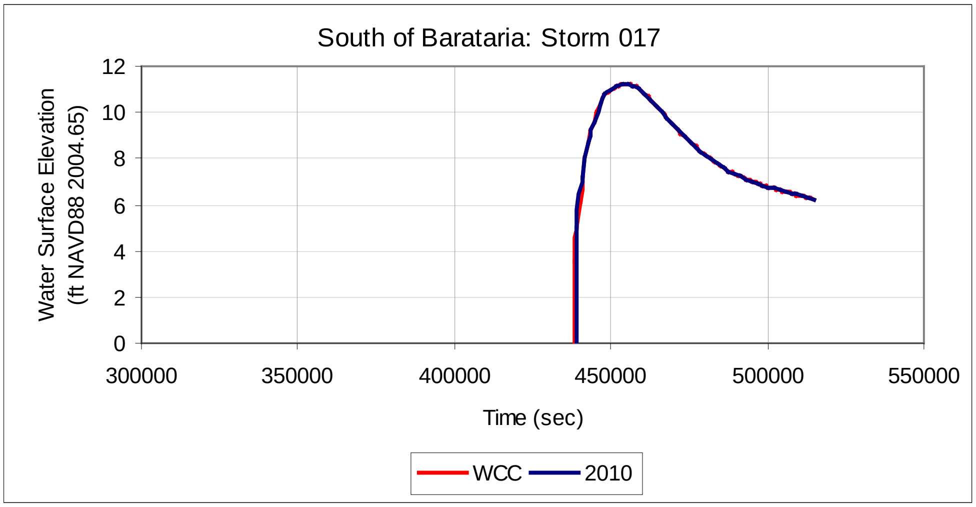

Bonne Isle: Storm 008
Water Surface Elevation
(ft NAVD88 2004.65)
0
1
2
3
4
5
6
7
8
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010


Lafitte: Storm 008
Water Surface Elevation
(ft NAVD88 2004.65)
8
7
6
5
4
3
2
1
0
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

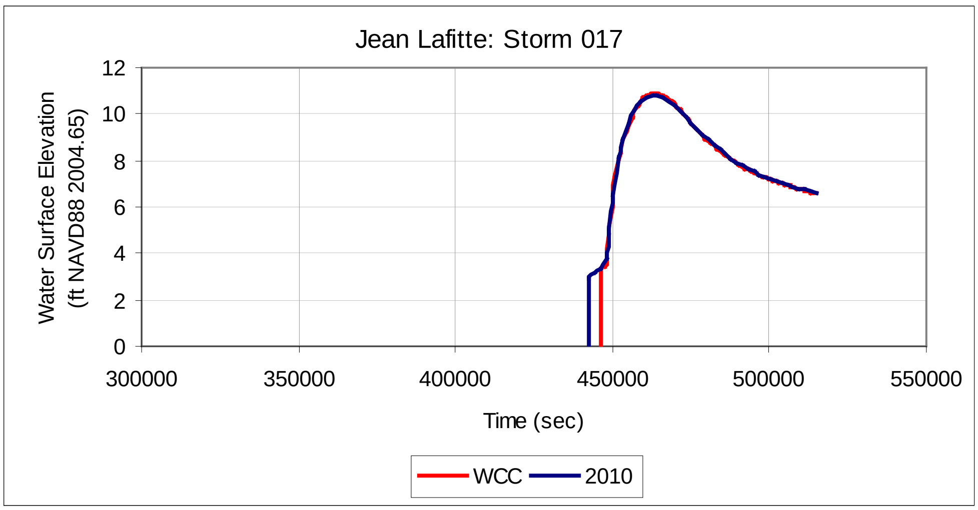

Barataria: Storm 008
Water Surface Elevation
(ft NAVD88 2004.65)
8
7
6
5
4
3
2
1
0
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010


Ollie: Storm 008
Water Surface Elevation
(ft NAVD88 2004.65)
8
7
6
5
4
3
2
1
0
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

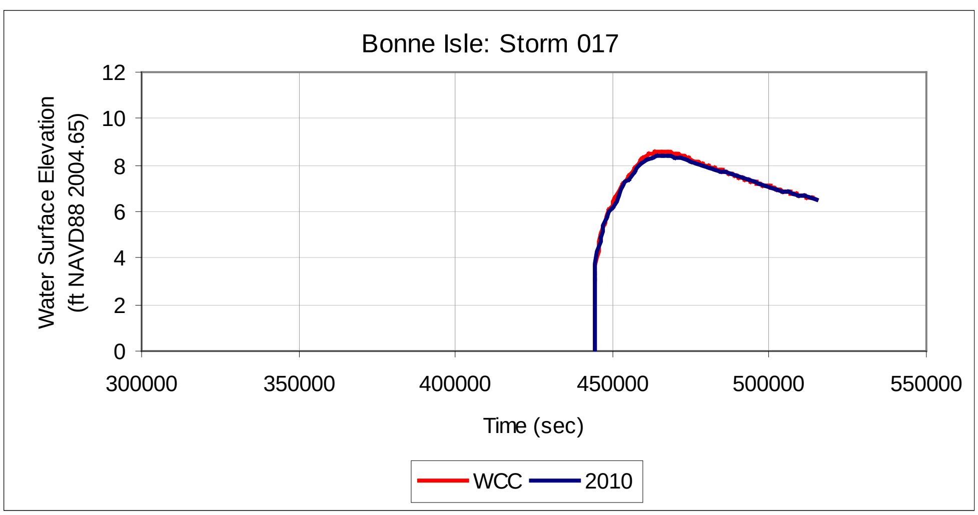
Storm 017



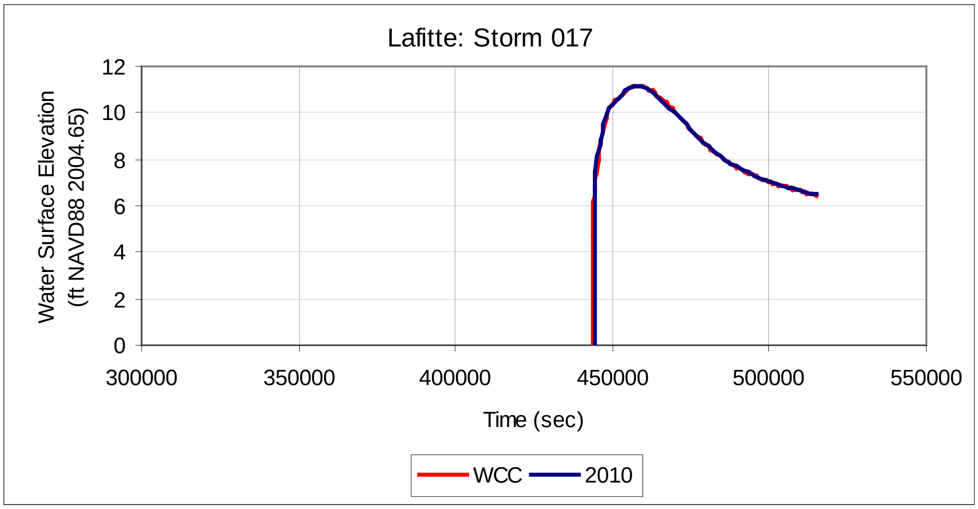
South of Barataria: Storm 017
Water Surface Elevation (ft NAVD88 2004.65)
0
2
4
6
8
10
12
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

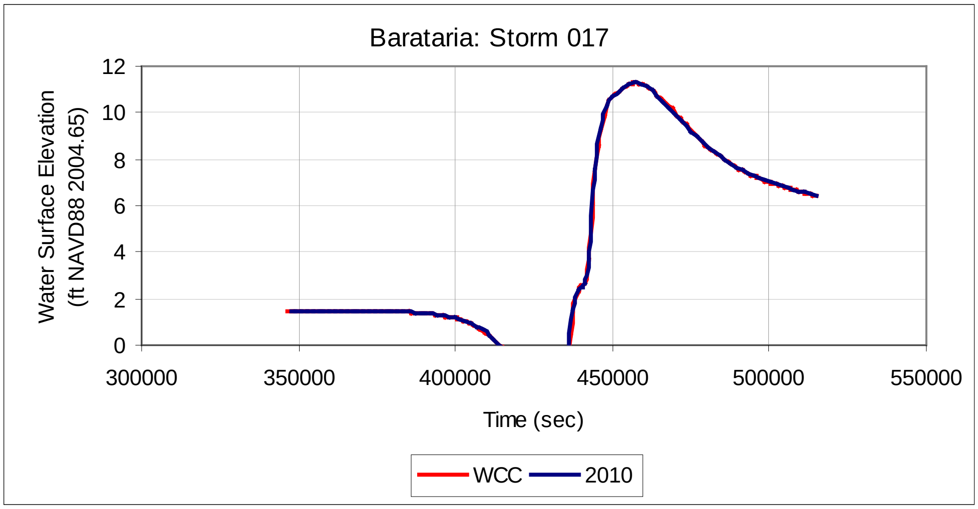


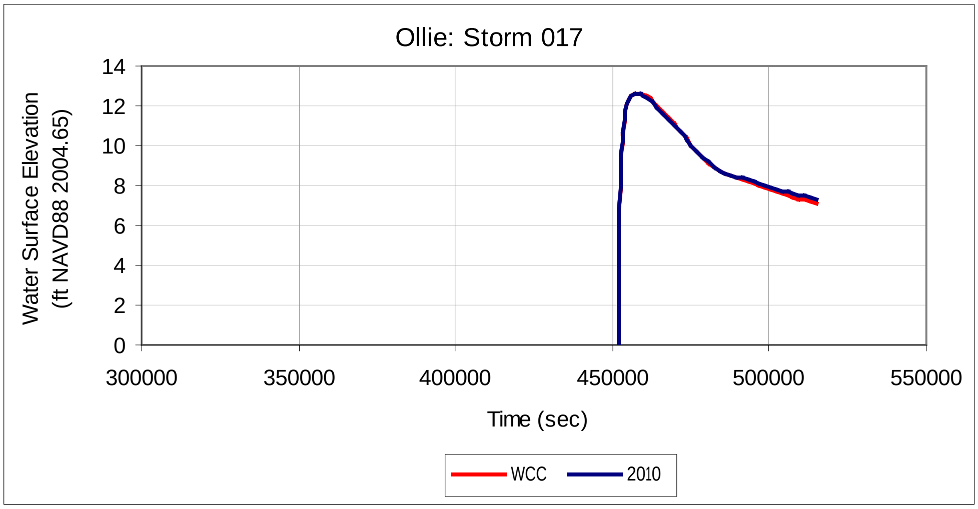
Jean Lafitte: Storm 017
Water Surface Elevation (ft NAVD88 2004.65)
0
2
4
6
8
10
12
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010



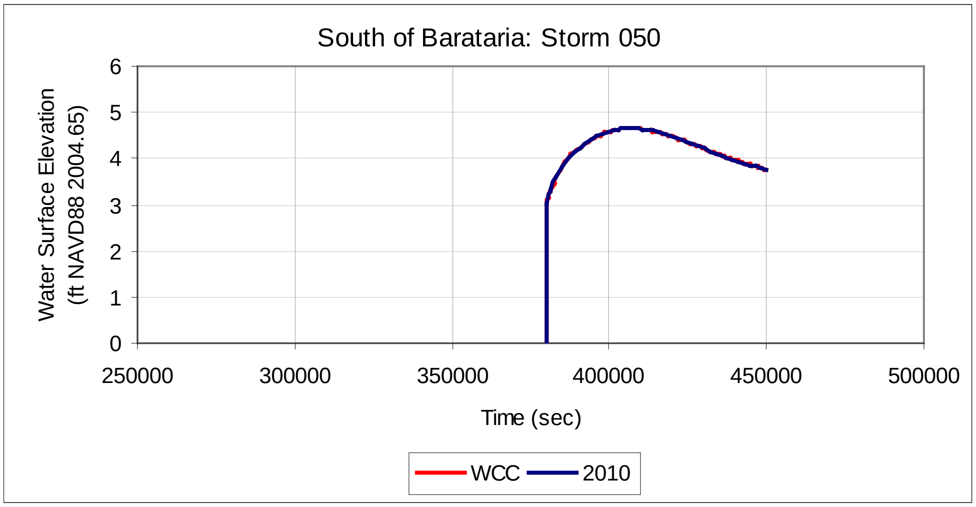
Bonne Isle: Storm 017
Water Surface Elevation (ft NAVD88 2004.65)
0
2
4
6
8
10
12
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

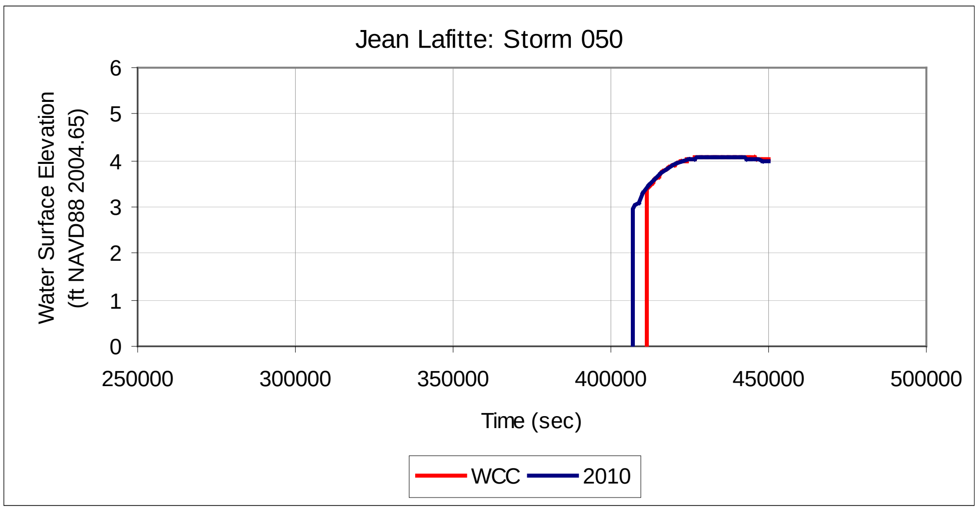

Lafitte: Storm 017
Water Surface Elevation (ft NAVD88 2004.65)
12
10
8
6
4
2
0
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010


Barataria: Storm 017
Water Surface Elevation (ft NAVD88 2004.65)
12
10
8
6
4
2
0
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010


Ollie: Storm 017
Water Surface Elevation (ft NAVD88 2004.65)
14
12
10
8
6
4
2
0
300000
350000
400000
450000
500000
550000
Time (sec)
WCC
2010

Storm 050


South of Barataria: Storm 050
Water Surface Elevation
(ft NAVD88 2004.65)
6
5
4
3
2
1
0
250000
300000
350000
400000
450000
500000
Time (sec)
WCC
2010


Jean Lafitte: Storm 050
Water Surface Elevation
(ft NAVD88 2004.65)
6
5
4
3
2
1
0
250000
300000
350000
400000
450000
500000
Time (sec)
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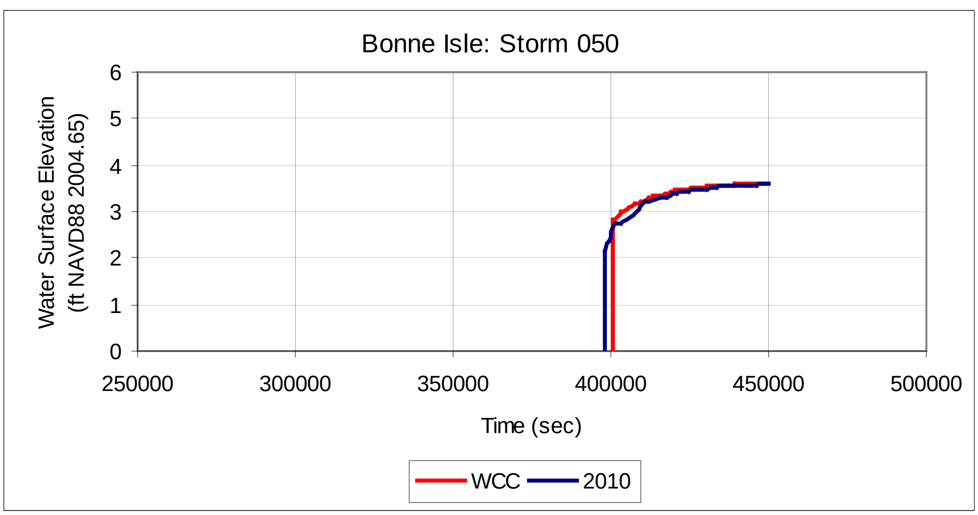
Bonne Isle: Storm 050
Water Surface Elevation
(ft NAVD88 2004.65)
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5
4
3
2
1
0
250000
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400000
450000
500000
Time (sec)
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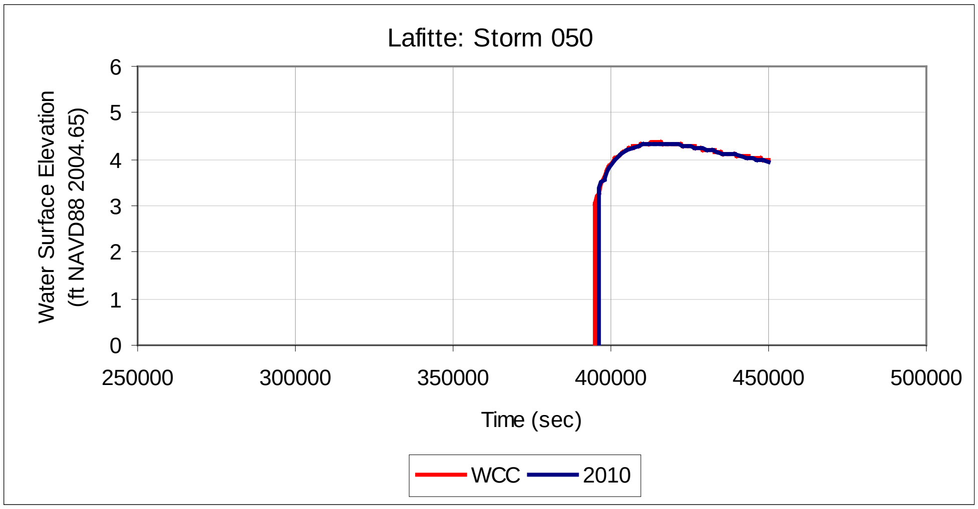

Lafitte: Storm 050
Water Surface Elevation (ft NAVD88 2004.65)
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3
4
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300000
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450000
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Time (sec)
WCC
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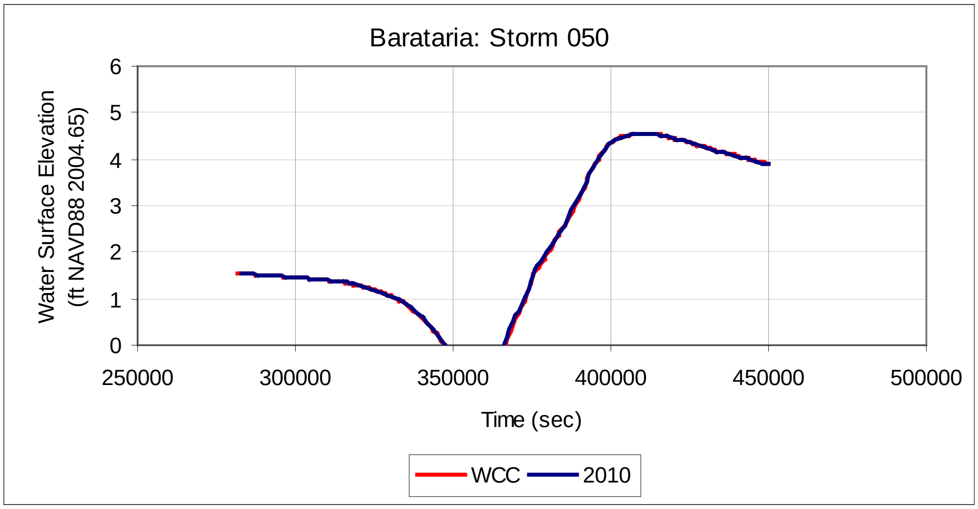

Barataria: Storm 050
Water Surface Elevation (ft NAVD88 2004.65)
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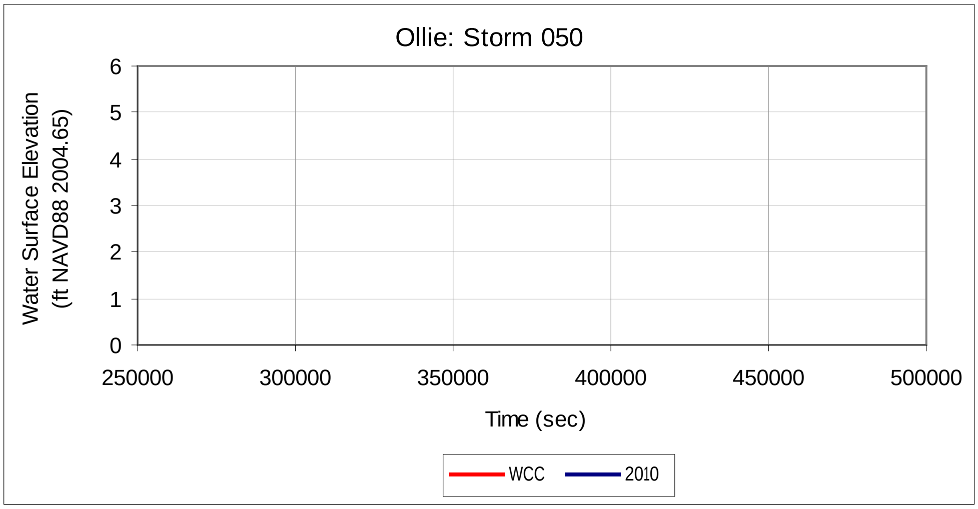
2
3
4
5
6
250000
300000
350000
400000
450000
500000
Time (sec)
WCC
2010



Ollie: Storm 050
Water Surface Elevation (ft NAVD88 2004.65)
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4
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6
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300000
350000
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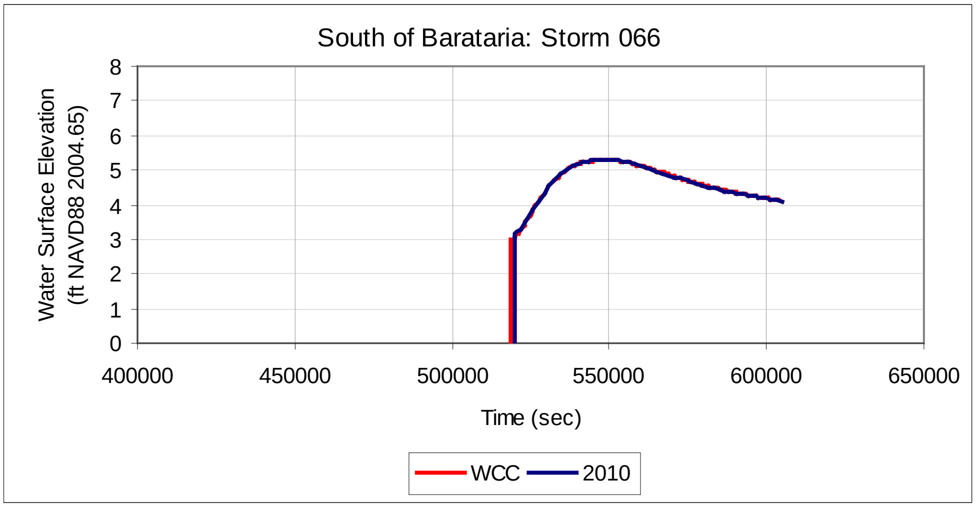
450000
500000
Time (sec)
WCC
2010

Storm 066



South of Barataria: Storm 066
Water Surface Elevation (ft NAVD88 2004.65)
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1
2
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4
5
6
7
8
400000
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500000
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Time (sec)
WCC
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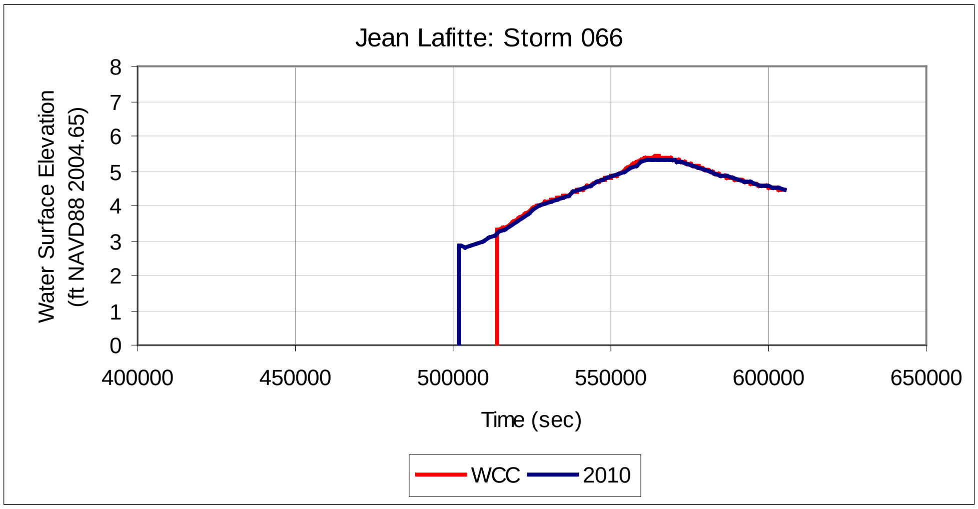

Jean Lafitte: Storm 066
Water Surface Elevation (ft NAVD88 2004.65)
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1
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6
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400000
450000
500000
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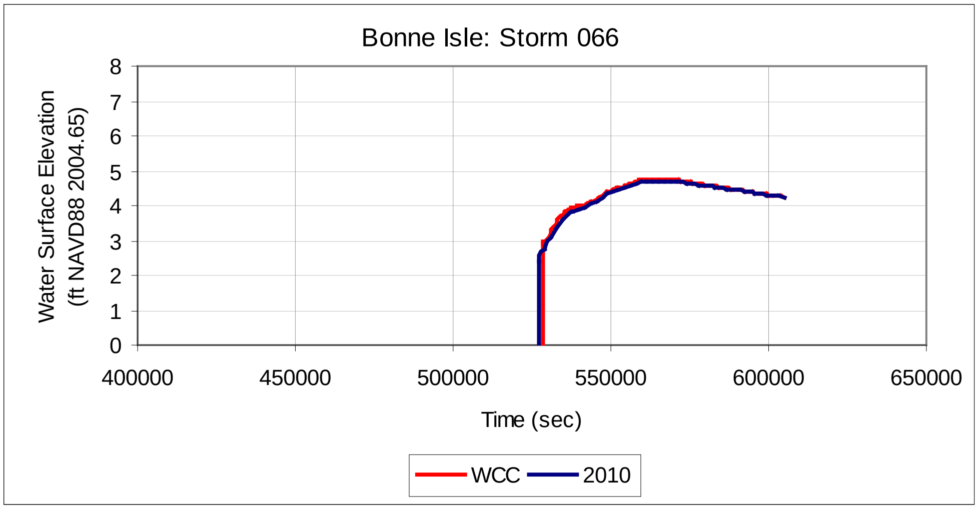
600000
650000
Time (sec)
WCC
2010



Bonne Isle: Storm 066
Water Surface Elevation (ft NAVD88 2004.65)
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400000
450000
500000
550000
600000
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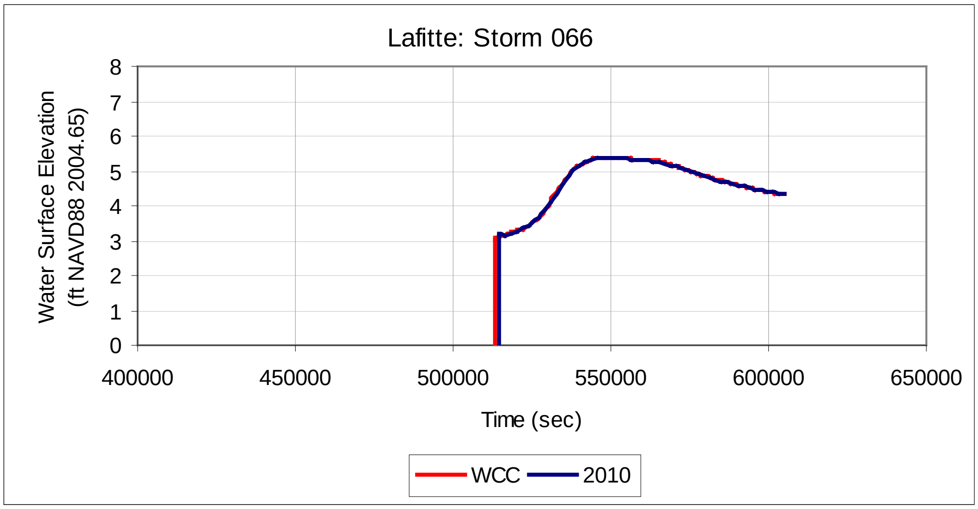
Time (sec)
WCC
2010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Time (sec)
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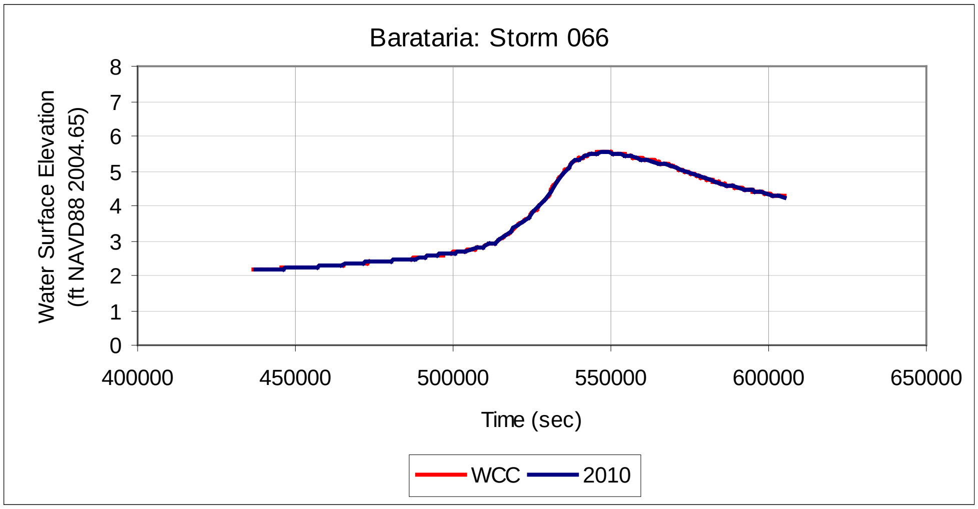
Barataria: Storm 066
Water Surface Elevation
(ft NAVD88 2004.65)
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5
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Time (sec)
WCC
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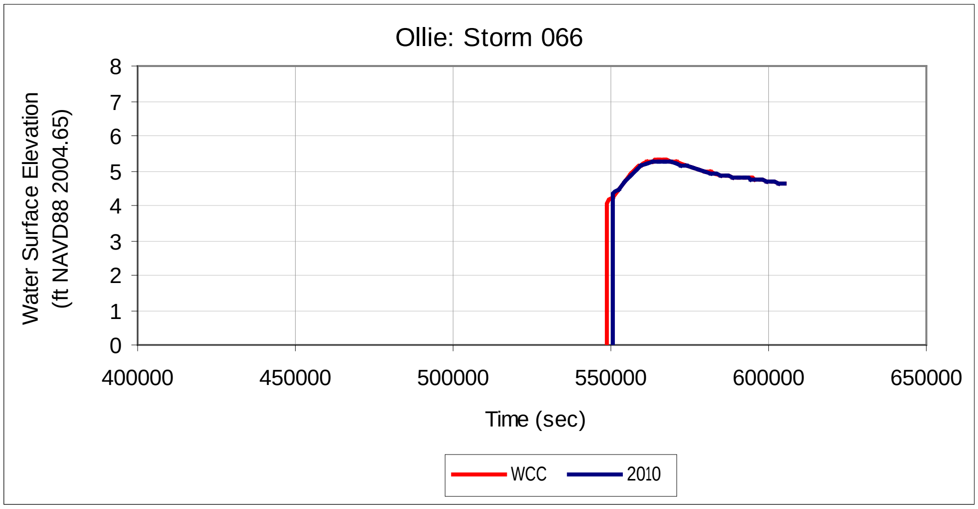
Ollie: Storm 066
Water Surface Elevation
(ft NAVD88 2004.65)
0
1
2
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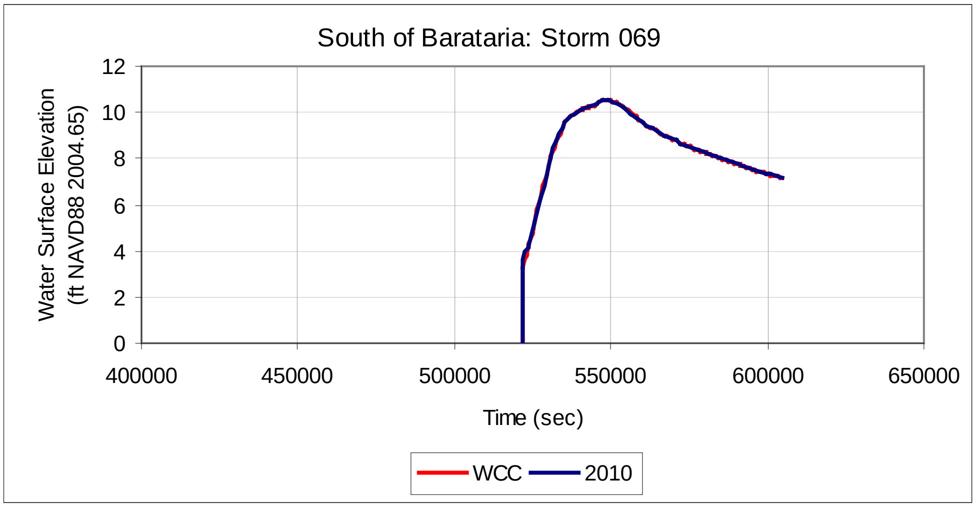
4
5
6
7
8
400000
450000
500000
550000
600000
650000
Time (sec)
WCC
2010

Storm 069


South of Barataria: Storm 069
Water Surface Elevation
(ft NAVD88 2004.65)
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10
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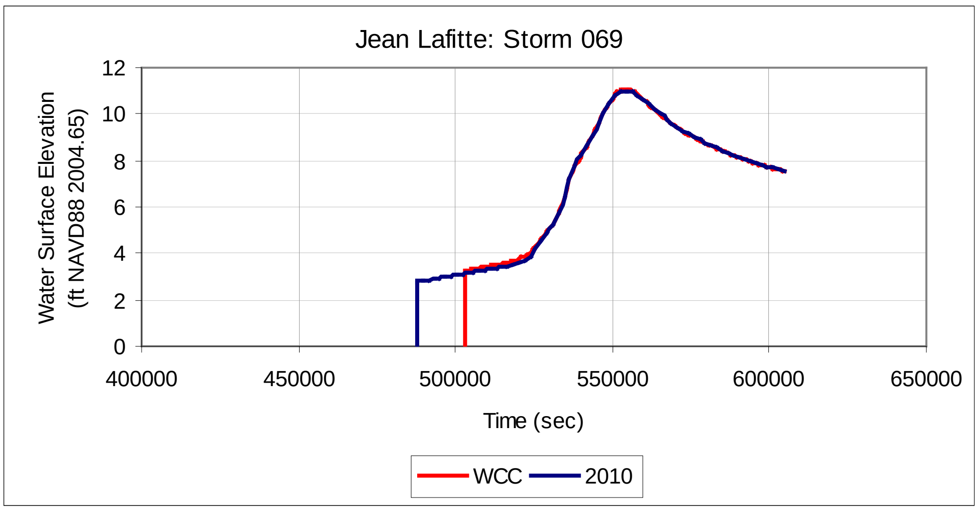
6
4
2
0
400000
450000
500000
550000
600000
650000
Time (sec)
WCC
2010


Jean Lafitte: Storm 069
Water Surface Elevation
(ft NAVD88 2004.65)
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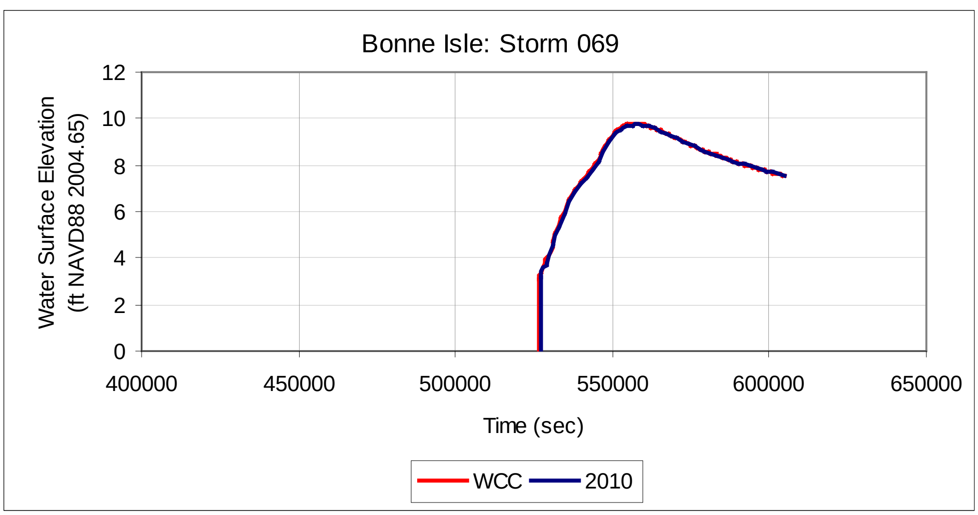
6
4
2
0
400000
450000
500000
550000
600000
650000
Time (sec)
WCC
2010


Bonne Isle: Storm 069
Water Surface Elevation
(ft NAVD88 2004.65)
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4
2
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600000
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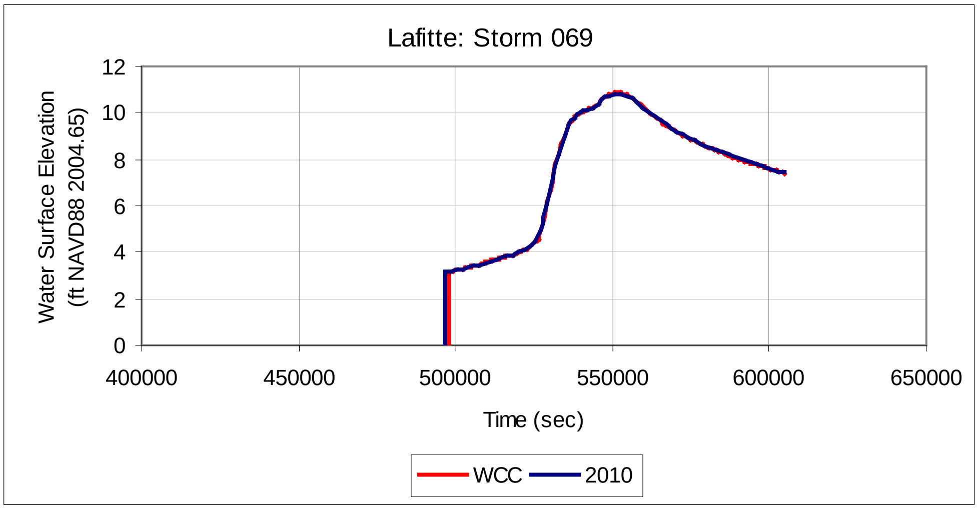
Time (sec)
WCC
2010



Lafitte: Storm 069
Water Surface Elevation (ft NAVD88 2004.65)
Time (sec)
WCC
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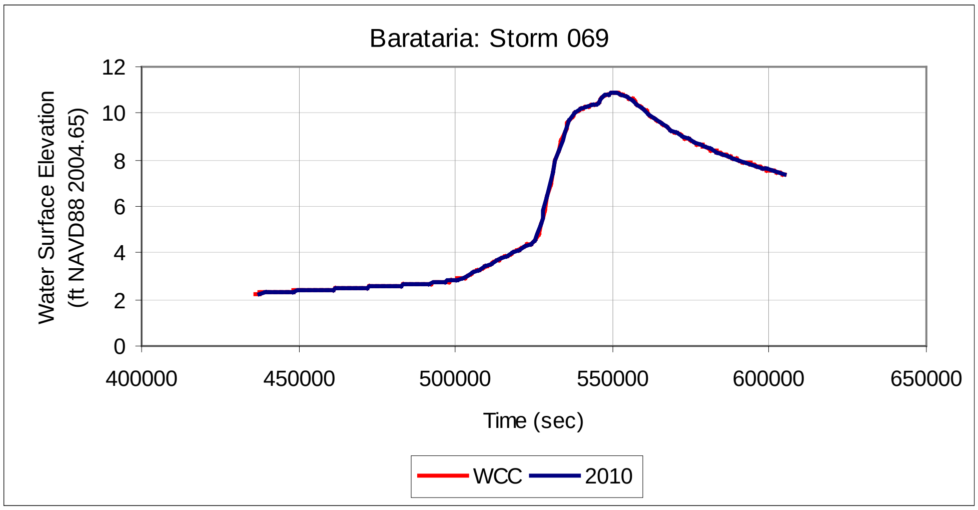
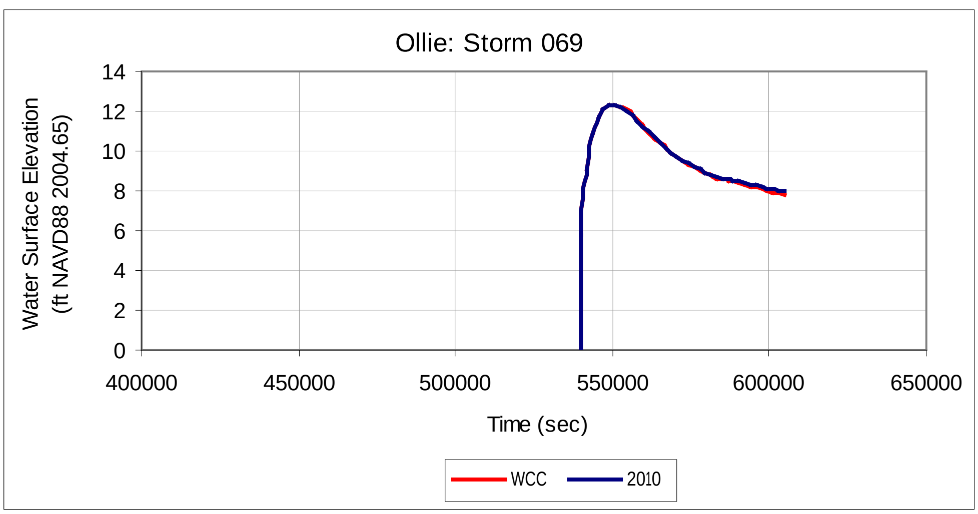
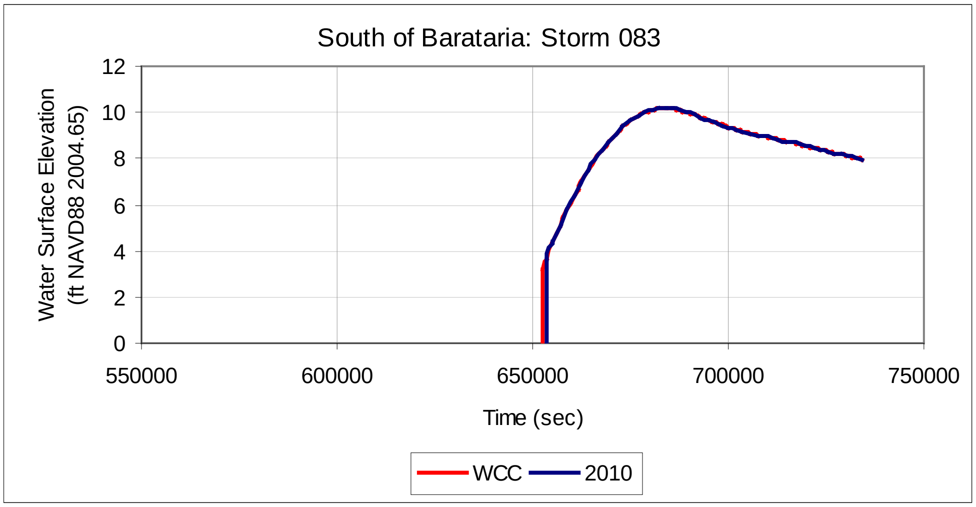


Barataria: Storm 069
Water Surface Elevation (ft NAVD88 2004.65)
Time (sec)
WCC
2010



Ollie: Storm 069
Water Surface Elevation (ft NAVD88 2004.65)
Time (sec)
WCC
2010

Storm 083


South of Barataria: Storm 083
Water Surface Elevation
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4
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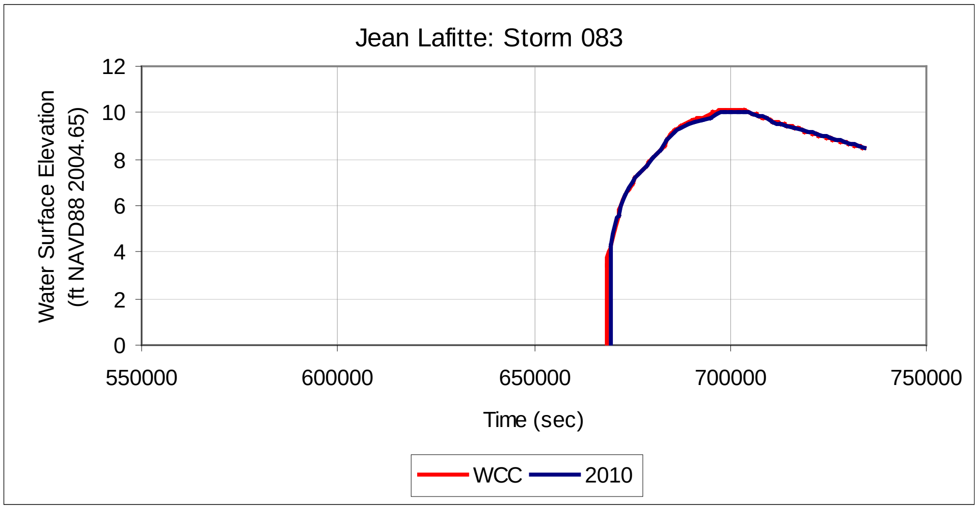
700000
750000
Time (sec)
WCC
2010


Jean Lafitte: Storm 083
Water Surface Elevation
(ft NAVD88 2004.65)
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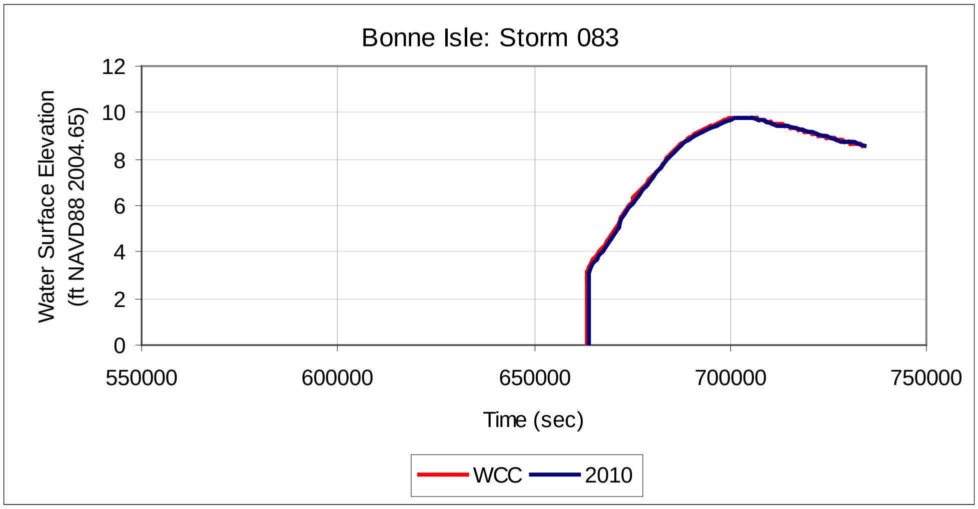
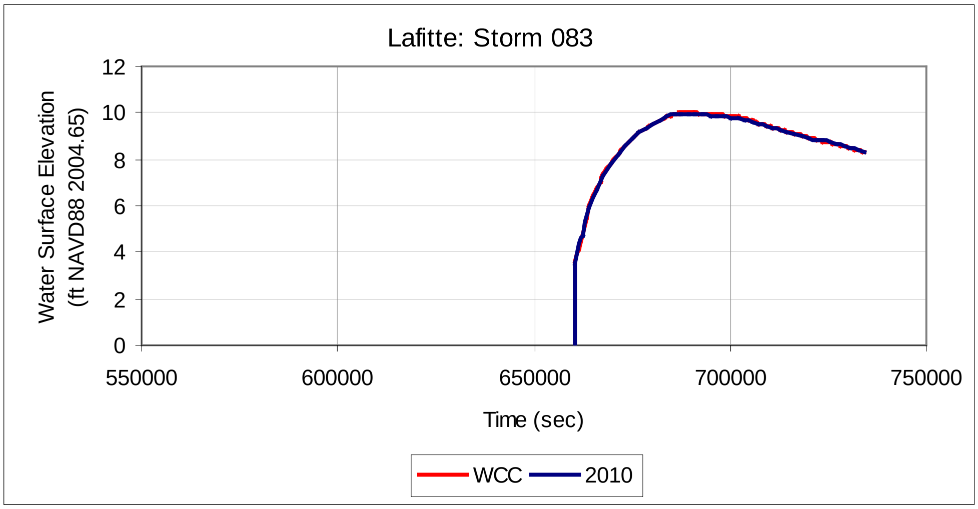
10
8
6
4
2
0
550000
600000
650000
700000
750000
Time (sec)
WCC
2010


Bonne Isle: Storm 083
Water Surface Elevation
(ft NAVD88 2004.65)
12
10
8
6
4
2
0
550000
600000
650000
700000
750000
Time (sec)
WCC
2010


Lafitte: Storm 083
Water Surface Elevation (ft NAVD88 2004.65)
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4
6
8
10
12
550000
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700000
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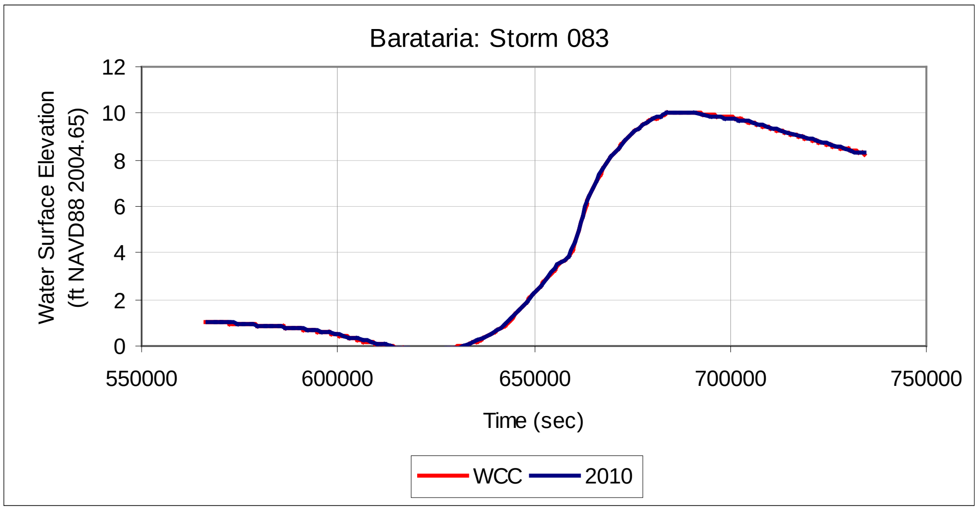
Time (sec)
WCC
2010


Barataria: Storm 083
Water Surface Elevation (ft NAVD88 2004.65)
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4
6
8
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12
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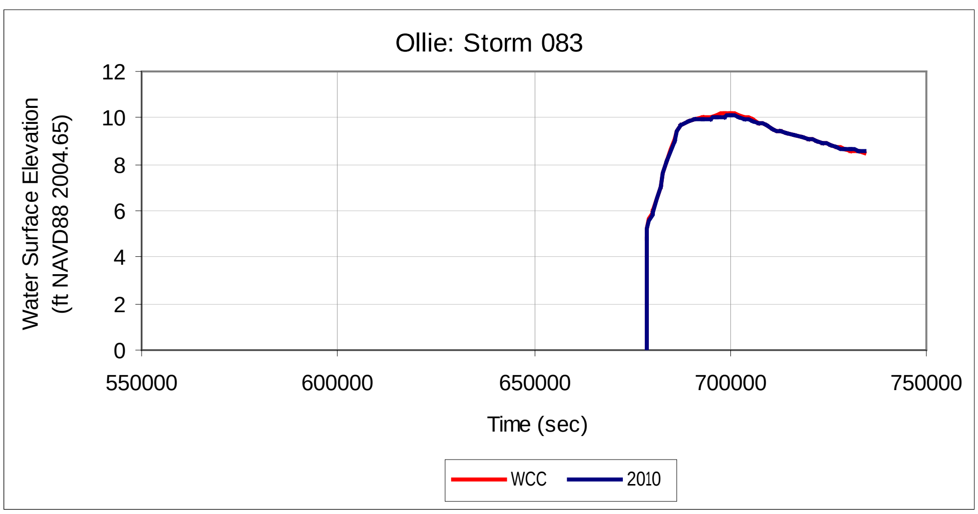
750000
Time (sec)
WCC
2010


Ollie: Storm 083
Water Surface Elevation (ft NAVD88 2004.65)
0
2
4
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12
550000
600000
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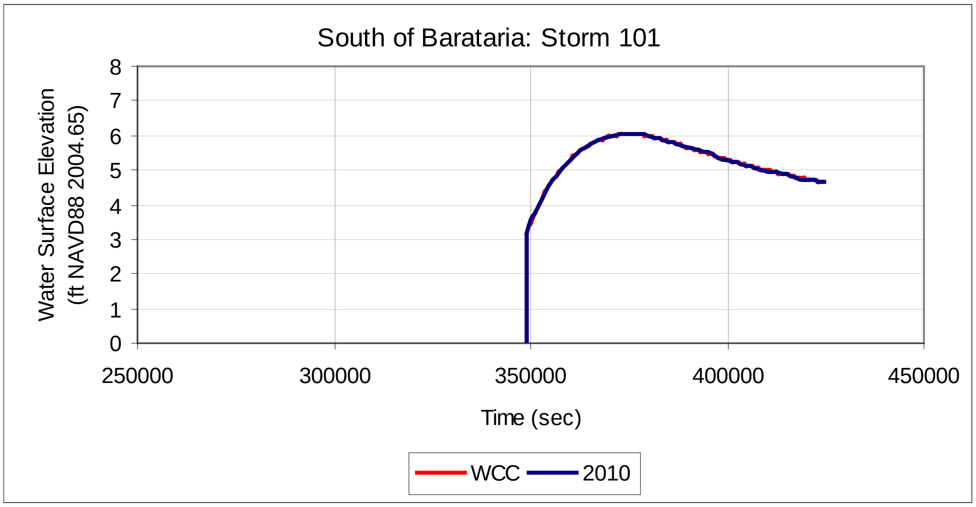
700000
750000
Time (sec)
WCC
2010

Storm 101


South of Barataria: Storm 101
Water Surface Elevation
(ft NAVD88 2004.65)
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6
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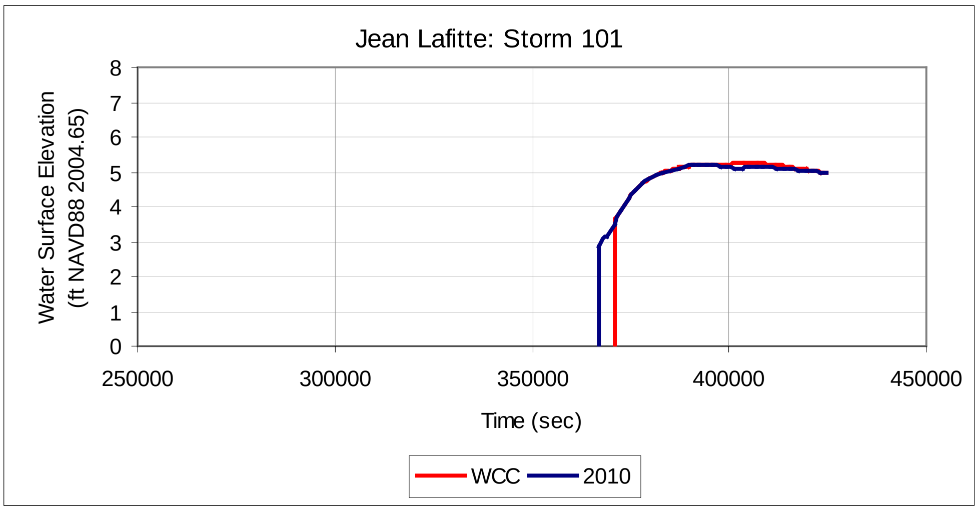
400000
450000
Time (sec)
WCC
2010


Jean Lafitte: Storm 101
Water Surface Elevation
(ft NAVD88 2004.65)
0
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2
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4
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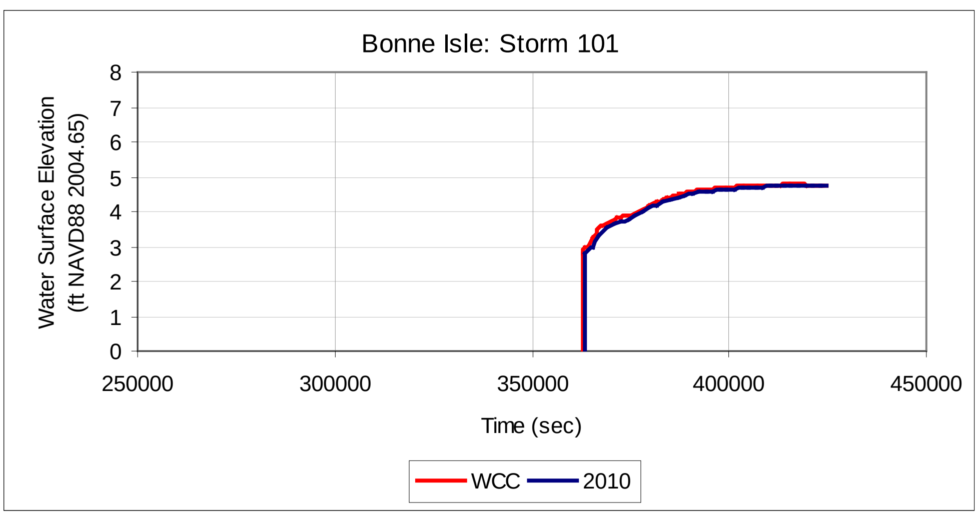
6
7
8
250000
300000
350000
400000
450000
Time (sec)
WCC
2010


Bonne Isle: Storm 101
Water Surface Elevation
(ft NAVD88 2004.65)
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1
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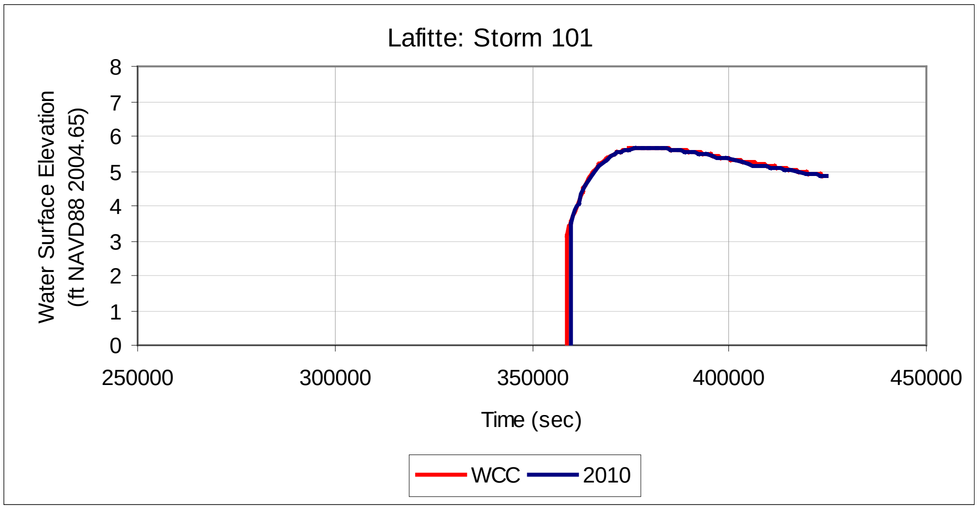
3
4
5
6
7
8
250000
300000
350000
400000
450000
Time (sec)
WCC
2010


Lafitte: Storm 101
Water Surface Elevation
(ft NAVD88 2004.65)
8
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6
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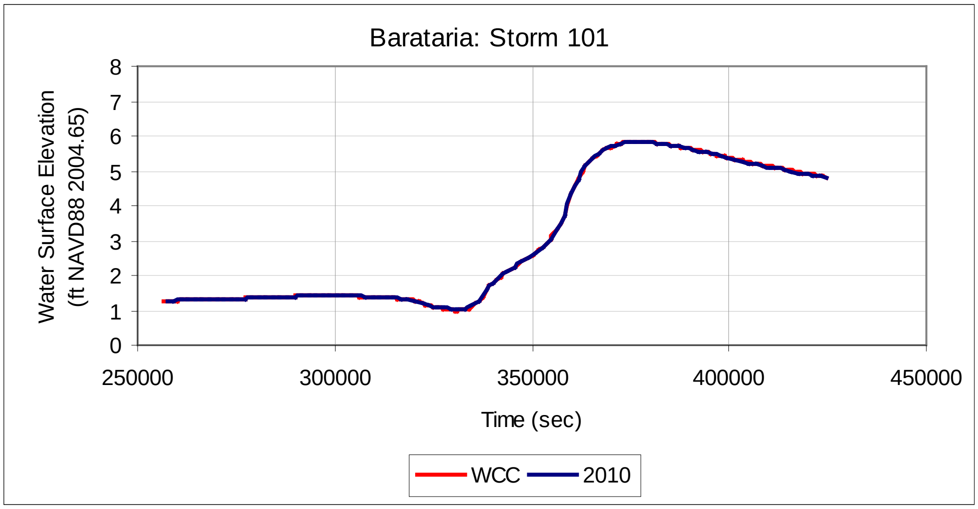
4
3
2
1
0
250000
300000
350000
400000
450000
Time (sec)
WCC
2010


Barataria: Storm 101
Water Surface Elevation
(ft NAVD88 2004.65)
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6
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4
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2
1
0
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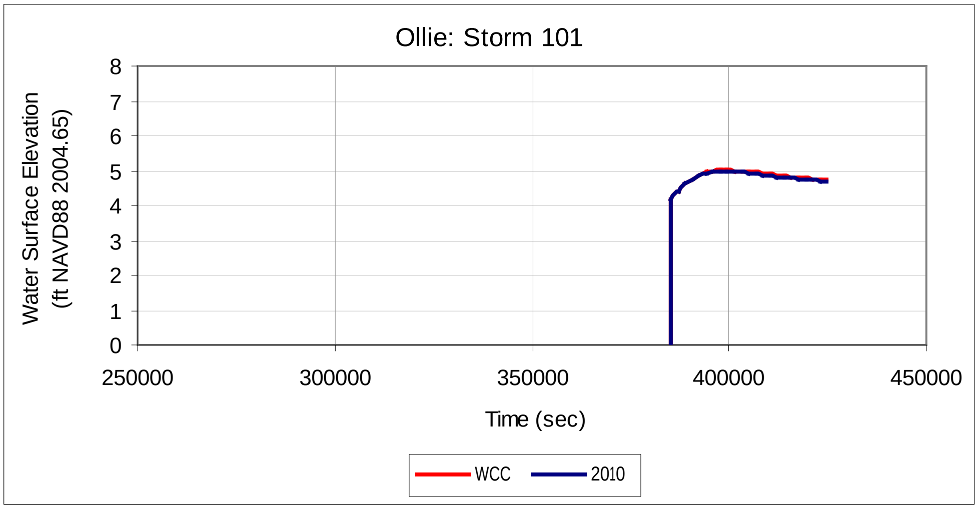
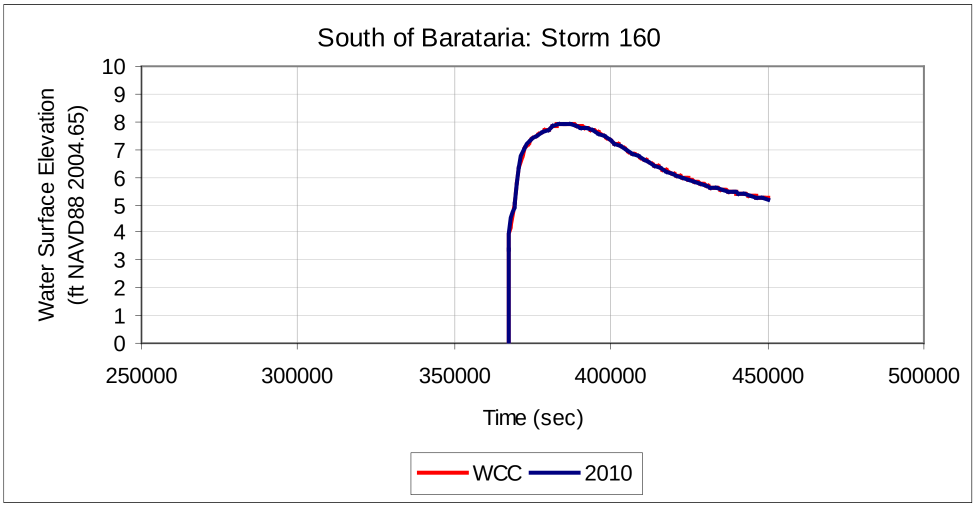
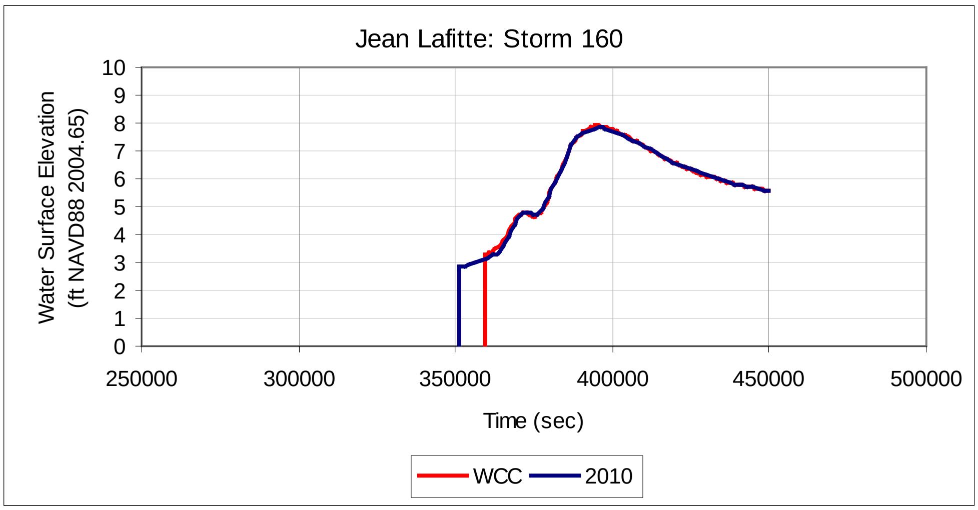
300000
350000
400000
450000
Time (sec)
WCC
2010


Ollie: Storm 101
Water Surface Elevation
(ft NAVD88 2004.65)
8
7
6
5
4
3
2
1
0
250000
300000
350000
400000
450000
Time (sec)
WCC
2010

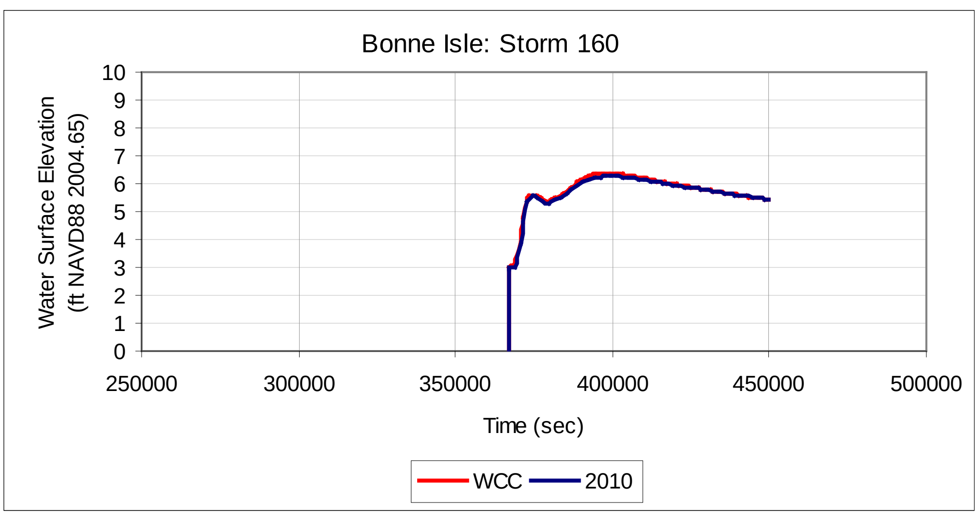
Storm 160



South of Barataria: Storm 160
Water Surface Elevation (ft NAVD88 2004.65)
0
1
2
3
4
5
6
7
8
9
10
250000
300000
350000
400000
450000
500000
Time (sec)
WCC
2010



Jean Lafitte: Storm 160
Water Surface Elevation (ft NAVD88 2004.65)
0
1
2
3
4
5
6
7
8
9
10
250000
300000
350000
400000
450000
500000
Time (sec)
WCC
2010



Bonne Isle: Storm 160
Water Surface Elevation (ft NAVD88 2004.65)
0
1
2
3
4
5
6
7
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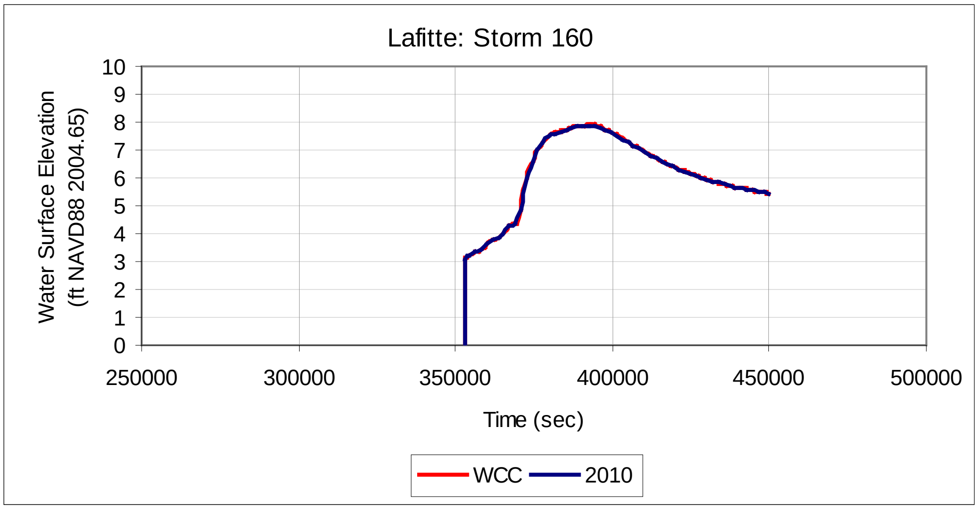
9
10
250000
300000
350000
400000
450000
500000
Time (sec)
WCC
2010


Lafitte: Storm 160
Water Surface Elevation (ft NAVD88 2004.65)
Time (sec)
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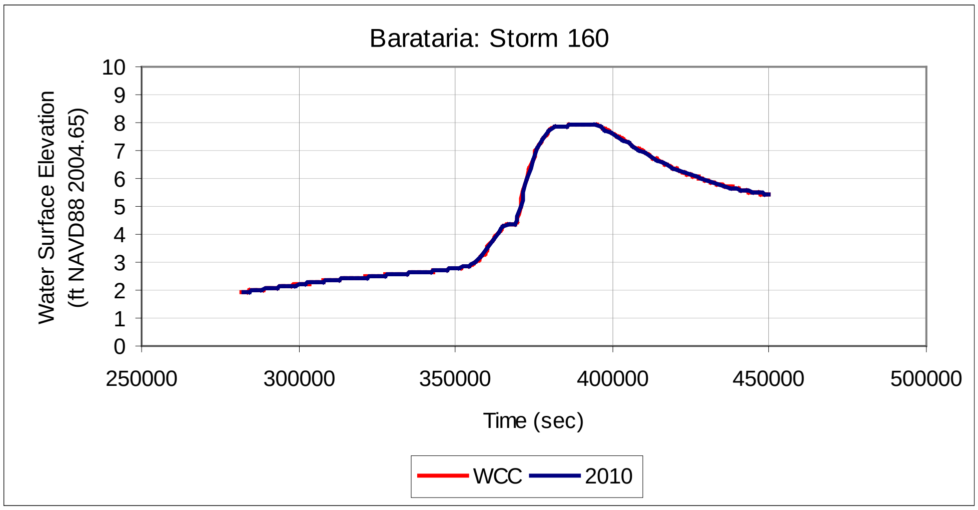
2010


Barataria: Storm 160
Water Surface Elevation (ft NAVD88 2004.65)
Time (sec)
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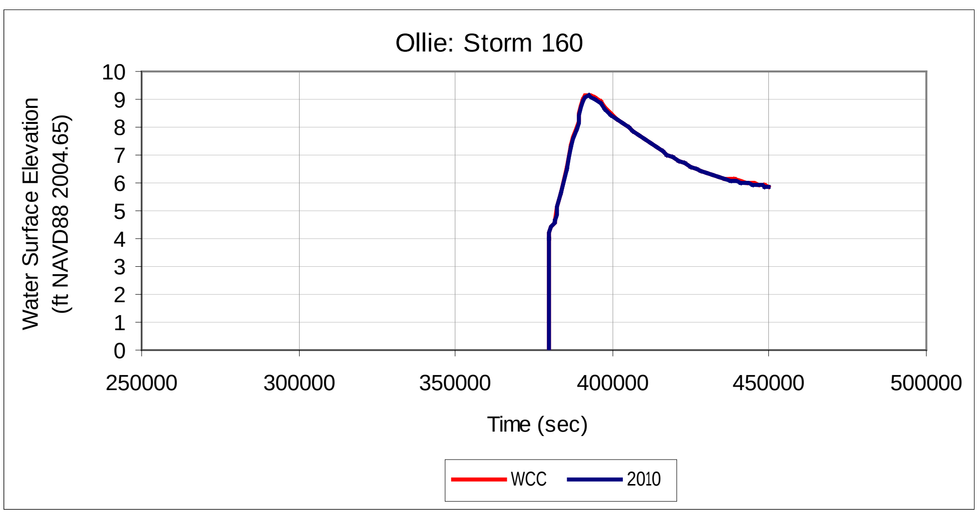
Ollie: Storm 160
Water Surface Elevation (ft NAVD88 2004.65)
Time (sec)
WCC
2010

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Appendix K

SWAN Modeling Lower Plaquemines Parish

(Version 20 Jul 2010)

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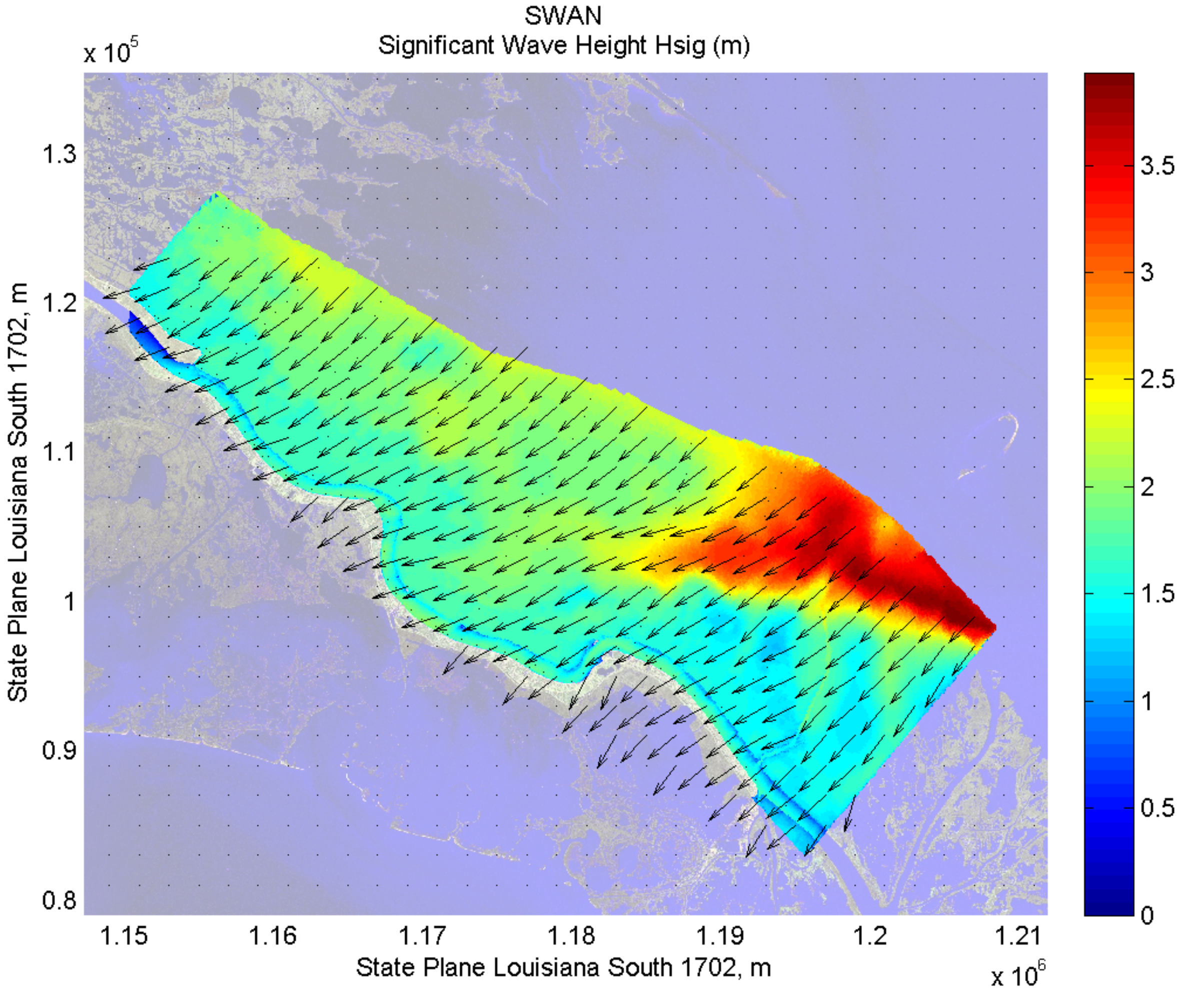
APPENDIX K – SWAN MODELLING LOWER PLAQUEMINES PARISH

Mississippi River Wave Modeling Below RM 44



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Executive Summary

A wave model was developed for the Mississippi River Levees (MRL) project below River Mile (RM) 44 to determine design wave conditions for the 0.2, 1, and 2% hurricane events. An unstructured SWAN model was chosen to provide high resolution in the area of interest. The model translates offshore wave boundary conditions to nearshore wave conditions. Steady state simulations were performed for a variety of scenarios. Each scenario was evaluated with different wave and wind boundary conditions. An analysis was performed to determine the sensitivity of the model to the wind and wave angle at the boundary, the wind speed value, and the water level amplification factor. The following list describes the 1% SWAN model scenarios for the MRL below RM 44:

Run 00 – Wind Speed = 77 mph at 45 deg
Run 01 – Wind Speed = 77 mph at 35 deg
Run 02 – Wind Speed = 77 mph at 40 deg
Run 03 – Wind Speed = 77 mph at 50 deg
Run 04 – Wind Speed = 77 mph at 55 deg
Run 05 – Wind Speed = 77 mph at 60 deg
Run 06 – Wind Speed = 77 mph at 65 deg
Run 07 – Wind Speed = 77 mph at 70 deg
Run 08 – Wind Speed = 77 mph at 45 deg with water level amplification factor = 0.85
Run 09 – Wind Speed = 77 mph at 45 deg with water level amplification factor = 1.15
Run 10 – Wind Speed = 88 mph at 45 deg
Run 11 – Wind Speed = 99 mph at 45 deg
Run 12 – Wind Speed = 77 mph at 45 deg with no offshore boundary conditions

The same scenarios were evaluated for the 0.2 and 2% events, except the wind speeds were adjusted. The 0.2% wind speed was selected to be 88 mph and the 2% wind speed was selected to be 70 mph.

The design conditions for the MRL below RM 44 were taken as the maximum wave values from scenarios 0 through 7. The following table summarizes the maximum 0.2, 1, and 2% wave conditions at each design reach.

Recommended 0.2%, 1% and 2% Wave Conditions at MRL Below RM 44						
River Mile	0.2% Significant Wave Height (ft)	0.2% Peak Period Tp (s)	1% Significant Wave Height (ft)	1% Peak Period Tp (s)	2% Significant Wave Height (ft)	2% Peak Period Tp (s)
11	7.00	5.9	5.50	5.2	4.75	4.9
12	7.00	5.9	5.50	5.2	4.75	4.9
13	7.25	6.0	5.50	5.2	4.75	4.9
14	7.50	6.1	6.00	5.5	4.75	4.9
15	7.50	6.1	6.00	5.5	4.75	4.9
16	7.50	6.1	6.00	5.5	5.00	5.0
17	7.75	6.2	6.00	5.5	5.00	5.0
18	7.75	6.2	6.00	5.5	5.00	5.0
19	7.75	6.2	6.00	5.5	5.00	5.0
20	7.75	6.2	6.00	5.5	5.00	5.0
21	7.75	6.2	6.00	5.5	5.00	5.0
22	7.75	6.2	6.00	5.5	5.00	5.0
23	7.75	6.2	6.00	5.5	5.00	5.0
24	7.75	6.2	6.00	5.5	5.00	5.0
25	7.75	6.2	6.00	5.5	5.00	5.0
26	8.00	6.3	6.50	5.7	5.50	5.2
27	8.00	6.3	6.50	5.7	5.50	5.2
28	8.25	6.4	6.50	5.7	5.50	5.2
29	8.25	6.4	6.50	5.7	5.50	5.2
30	8.25	6.4	6.50	5.7	5.50	5.2
31	8.25	6.4	6.50	5.7	5.50	5.2
32	8.25	6.4	6.50	5.7	5.50	5.2
33	7.50	6.1	6.00	5.5	5.50	5.2
34	7.50	6.1	6.00	5.5	5.00	5.0
35	7.50	6.1	6.00	5.5	5.00	5.0
36	7.50	6.1	6.00	5.5	5.00	5.0
37	7.50	6.1	6.00	5.5	5.00	5.0
38	7.50	6.1	6.00	5.5	5.00	5.0
39	7.50	6.1	6.00	5.5	5.00	5.0
40	7.50	6.1	6.00	5.5	5.00	5.0
41	7.50	6.1	5.50	5.2	4.50	4.7
42	7.25	6.0	5.50	5.2	4.50	4.7
43	6.75	5.8	5.00	5.0	4.00	4.5
44	6.75	5.8	5.00	5.0	4.00	4.5
45	6.25	5.6	4.50	4.7	3.75	4.3
46	5.50	5.2	4.25	4.6	3.75	4.3
47	5.25	5.1	4.00	4.5	3.50	4.2

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1.0 INTRODUCTION

1.1 OBJECTIVE

The objective of the SWAN wave model is to determine refined wave characteristics for the MRL below RM 44.

2.0 MODEL INPUTS

2.1 BATHYMETRY

Bathymetric data from the large scale ADCIRC model was used to develop a SWAN unstructured mesh of the area of interest. A section of the sl15_2007_IHNC ADCIRC mesh was extracted for the area of interest. **Table 1** summarizes the data sources used to construct the SWAN model mesh. **Figure 1** shows the bathymetry of the area of interest, including the Mississippi River channel below RM 44.

Table 10 – XYZ Data Sources

XYZ Data Sets	Source	Date
ADCIRC Bathymetry	sl15_2007_IHNC_r03q.grd	12/2008

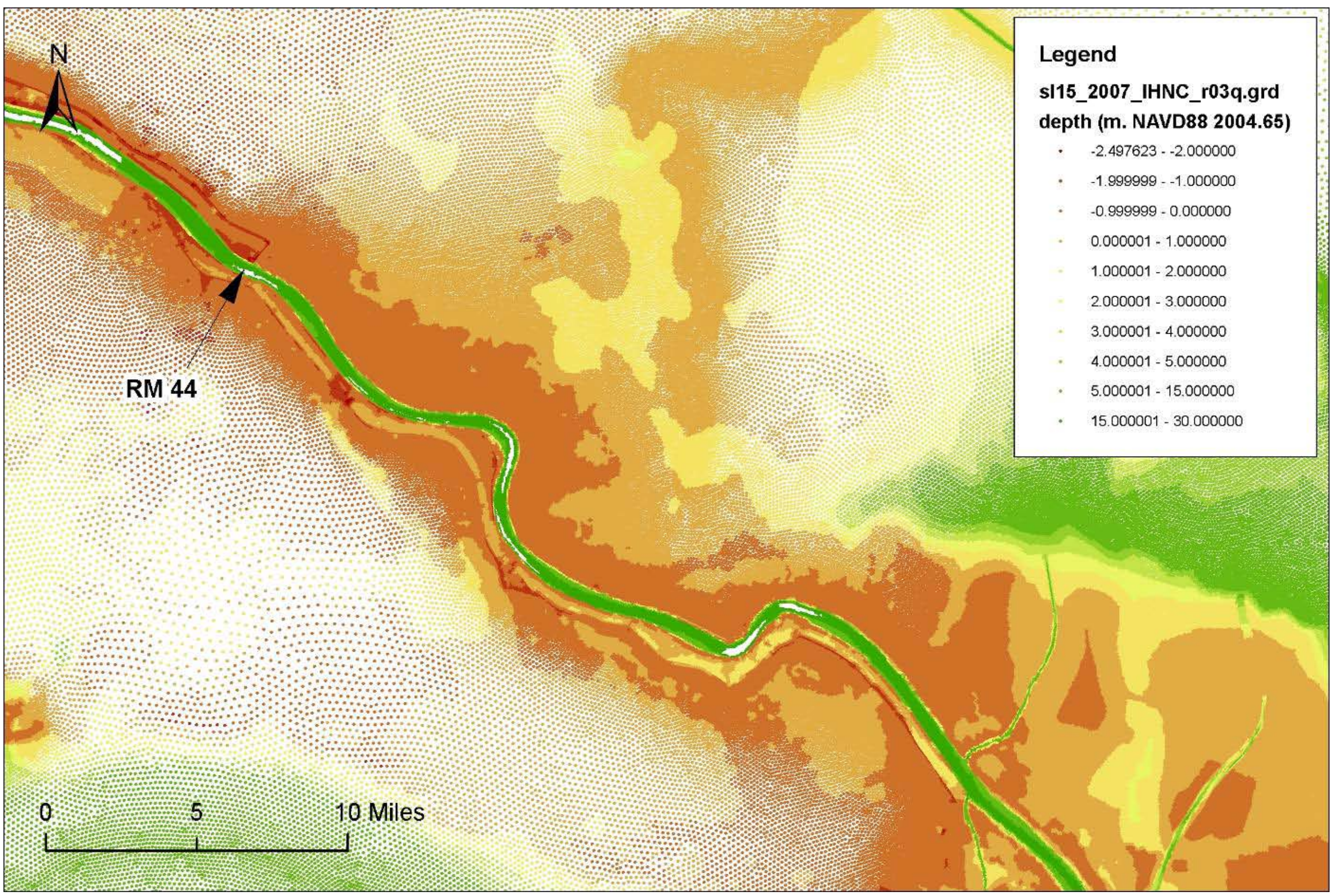


Figure 32 – ADCIRC Bathymetry

2.2 SWAN UNSTRUCTURED MESH DEVELOPMENT

Unstructured meshes allow high resolution in areas of interest, thereby decreasing the overall number of computational elements. Unstructured meshes also provide better representation of irregularly shaped boundaries than do conventional regular grids. SMS (Surface-water Modeling System) was used to construct the unstructured mesh for the area of interest by utilizing the ADCIRC (Advanced Circulation Model) unstructured mesh format. SMS was also used to assign land and ocean boundary conditions at appropriate boundaries. The mesh data was saved in a file named ‘fort.14’, which was one of the primary inputs into ADCIRC. The fort.14 file can also be interpreted by SWAN.

Figure 2 shows the bathymetry of the mesh that was extracted from the large scale IHNC ADCIRC grid. **Figure 3** shows the mesh resolution in the area of interest. The extracted mesh has 100,643 nodes and 199,596 elements. The element size varies from 50 to 160 meters.

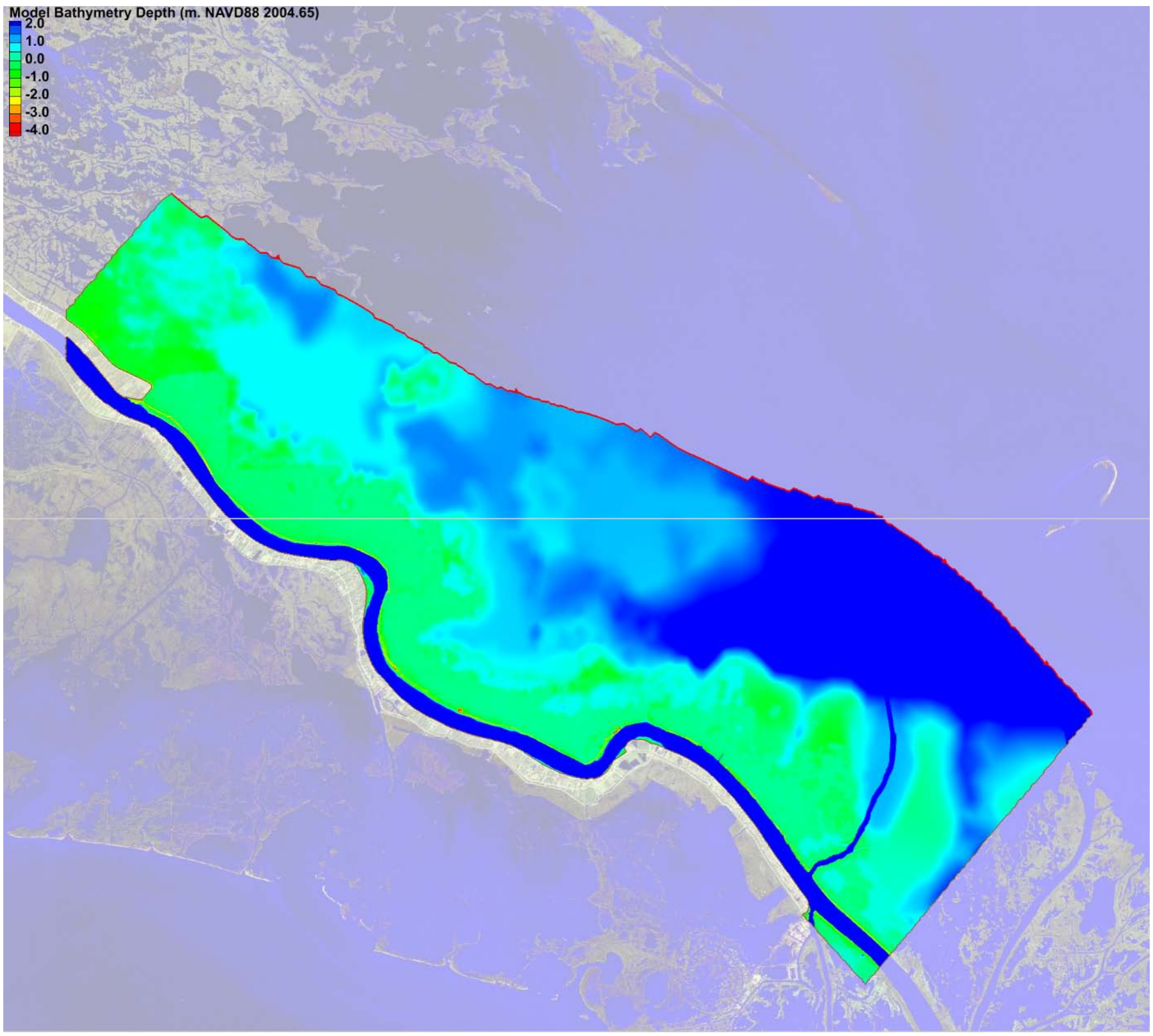


Figure 33 – SWAN Mesh Bathymetry and Boundaries

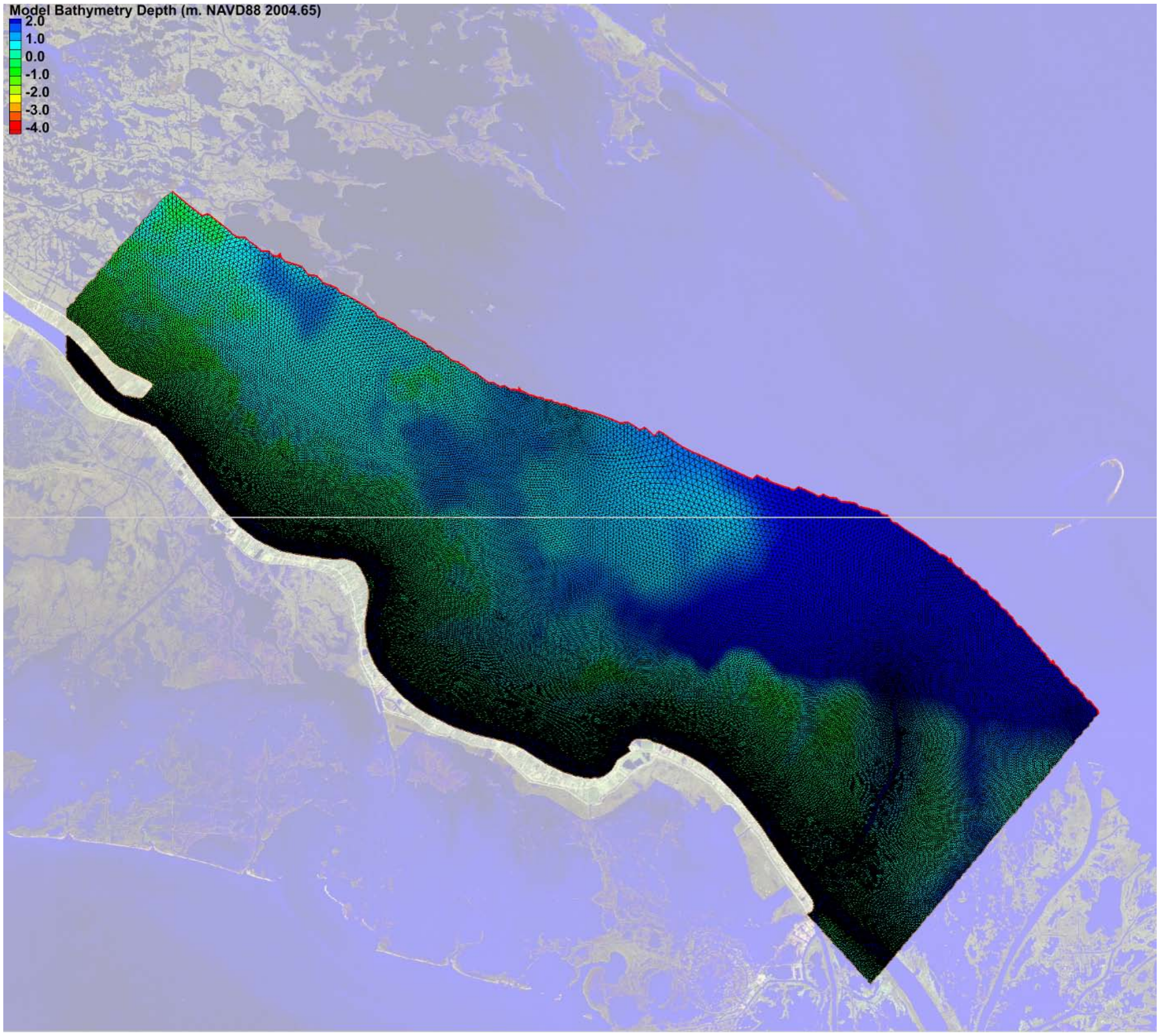


Figure 34 – SWAN Mesh Resolution and Boundaries

2.3 BOUNDARY CONDITIONS

Boundary conditions were applied at five segments of the northeastern edge of the SWAN mesh. The boundary node-strings are represented as red lines in **Figure 4**. **Table 2** summarizes the offshore boundary conditions for each scenario.

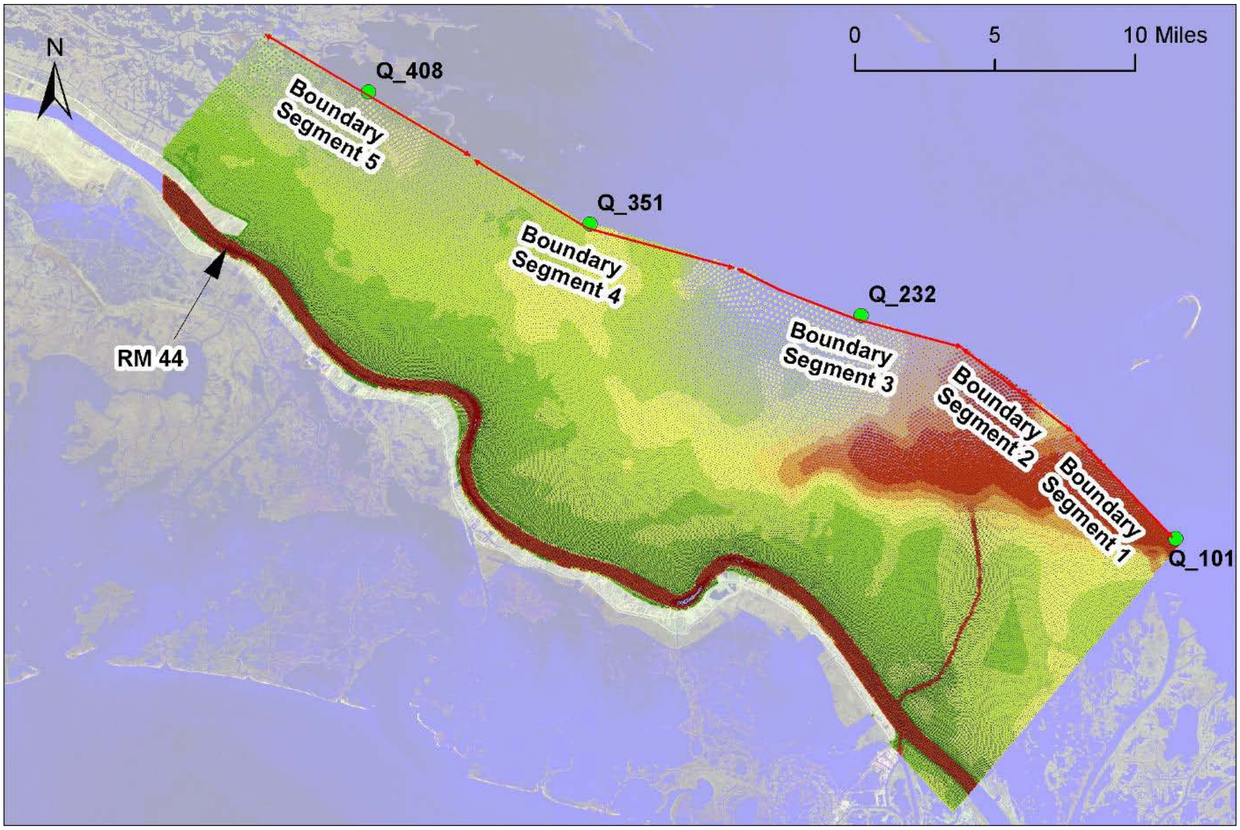


Figure 35 – Offshore Boundary Condition Segments

Table 11 – Summary of Offshore Wave Boundary Conditions

Offshore 0.2%, 1% and 2% Boundary Conditions							
Boundary Segment	ADCIRC Save Point ID	0.2% Significant Wave Height (ft)	0.2% Mean Wave Period (s)	1% Significant Wave Height (ft)	1% Mean Wave Period (s)	2% Significant Wave Height (ft)	2% Mean Wave Period (s)
1	Q-101	13.6	13.1	11.7	11.3	10.4	10.2
2	Interpolated between Q-101 and Q-232	11.4	12.6	9.55	10.75	8.35	9.6
3	Q-232	9.2	12.1	7.4	10.2	6.3	9
4	Q-351	9.3	11.8	7.4	9.9	6.2	8.8
5	Q-408	9.2	11.6	7	9.6	5.7	8.4

Boundary conditions were taken from the synthetic ADCIRC/STWAVE storm suite that was used for the design of the IHNC/GIWW barrier. Wave and Surge results from 152 storms are input into the JPM-OS statistical code to produce 0.2, 1, and 2% surge and wave conditions. These conditions were applied at the offshore boundary segments of the SWAN grid.

For each scenario, the water surface elevation was set equal to the 0.2, 1, and 2% surge elevation taken from the IHNC ADCIRC surge statistics. **Figure 5** and **Figure 6** show the 1 and 2% water surface elevations applied in the SWAN model.

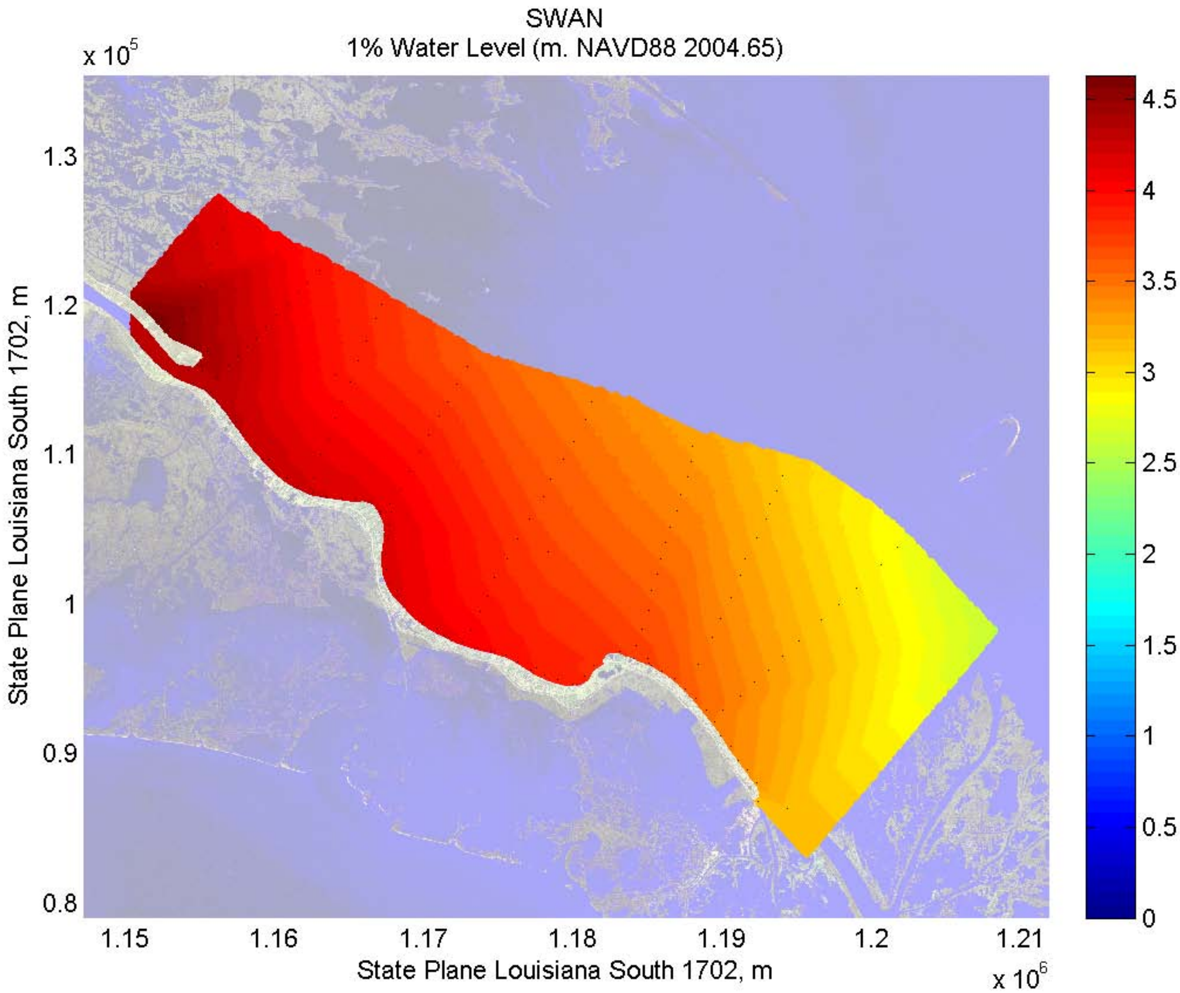


Figure 36 – 2% Surge Elevation Surface (m. NAVD88 2004.65)

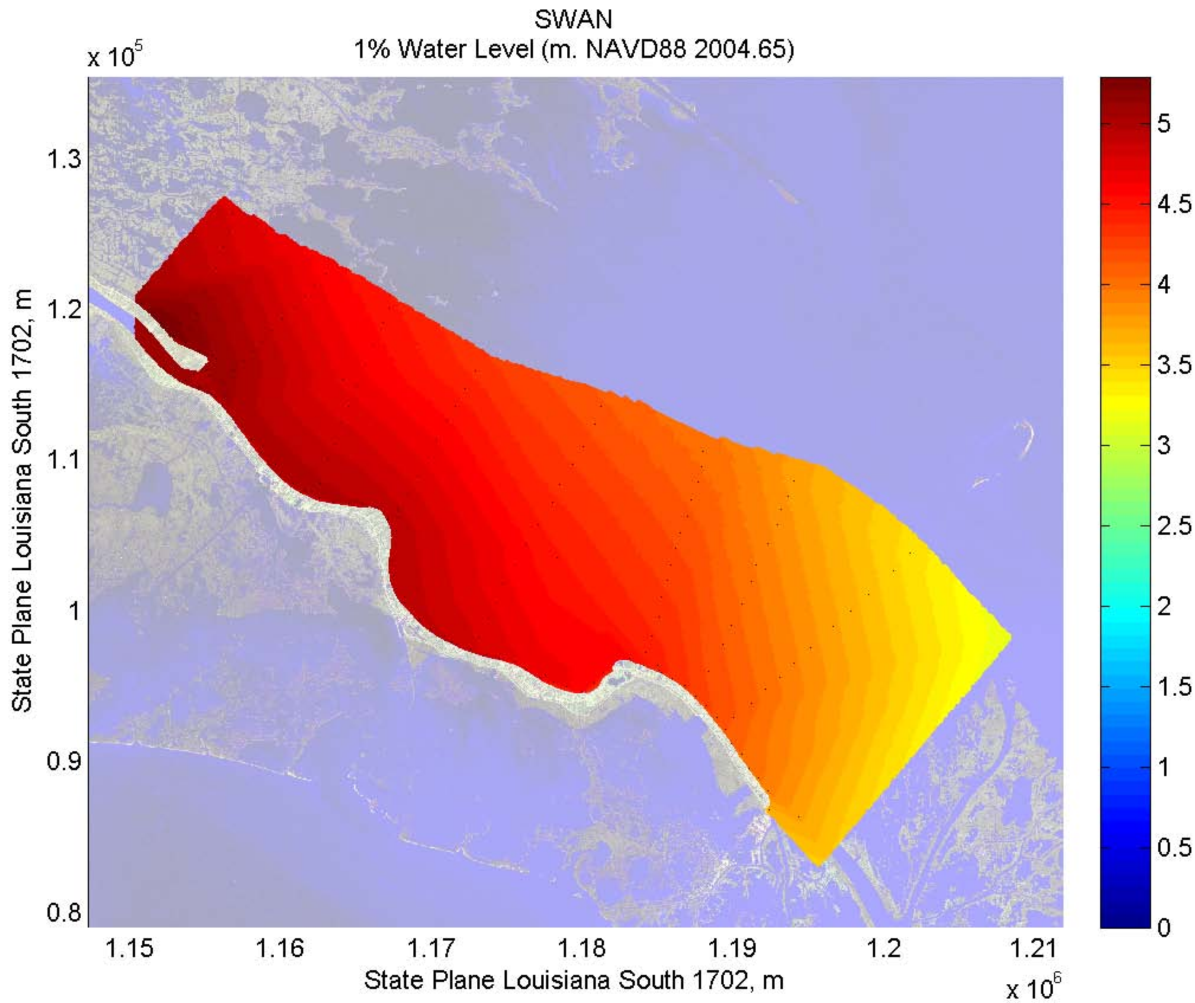


Figure 37 – 1% Surge Surface Elevation (m. NAVD88 2004.65)

2.4 FRICTION

Friction was not applied in the SWAN model.

2.5 NUMERICAL SETTINGS

The solution of the equations in SWAN requires an iterative approach. The model was set to stop iterating when wave heights and peak wave periods in 99% of the active nodes do not change with more than 2%, when compared to the previous iteration. Due to time constraints, all simulations were set up to conduct a maximum of 100 iterations. However, the model runs met the stopping criterion well before 100 iterations. A sample of a SWAN output print file is found in **Appendix J2**. This sample print file shows the number of iterations and the percent of converged elements.

2.6 POINTS OF INTEREST

SWAN allows the user to define a set of X, Y points at which to record and tabulate detailed model output. **Figure 7** and **Figure 8** show the locations of the selected output locations for the area of interest. A line of points was drawn from RM 0 to RM 44 directly in front of the East Bank levee. Also a series of points were placed perpendicular to the levee for wave decay plots.

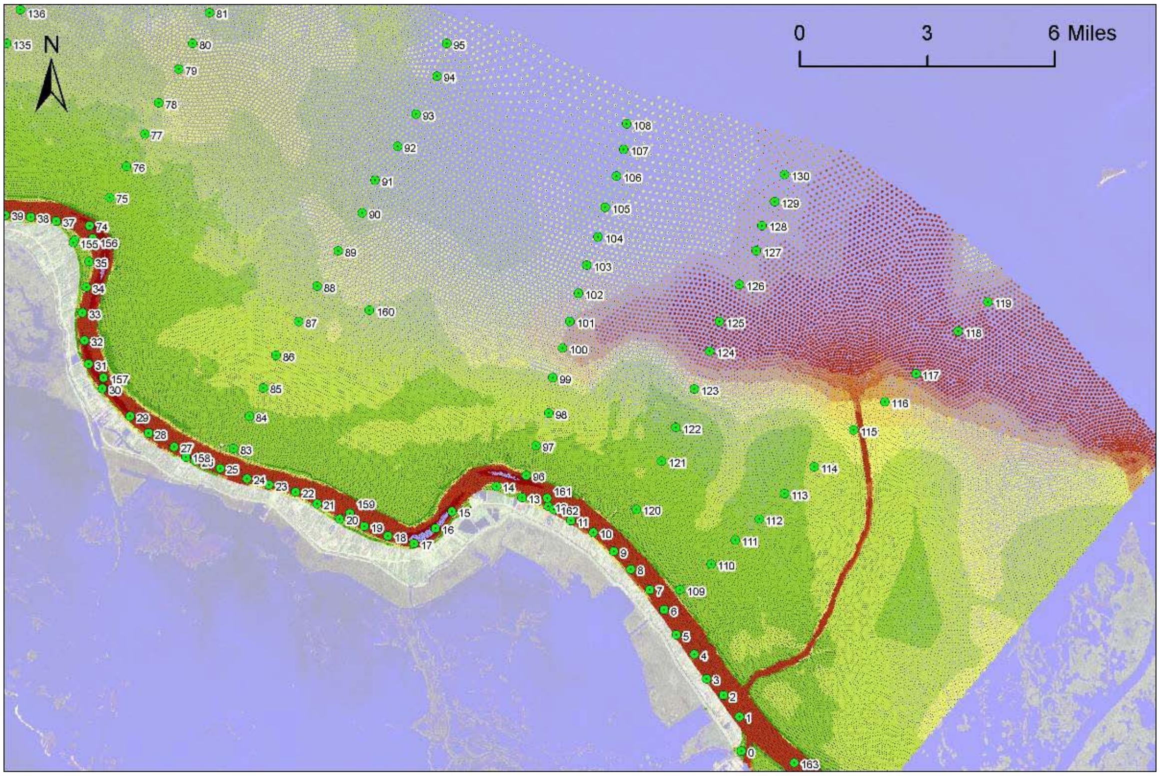


Figure 38 – SWAN Detailed Output Points at Area of Interest – Lower Section

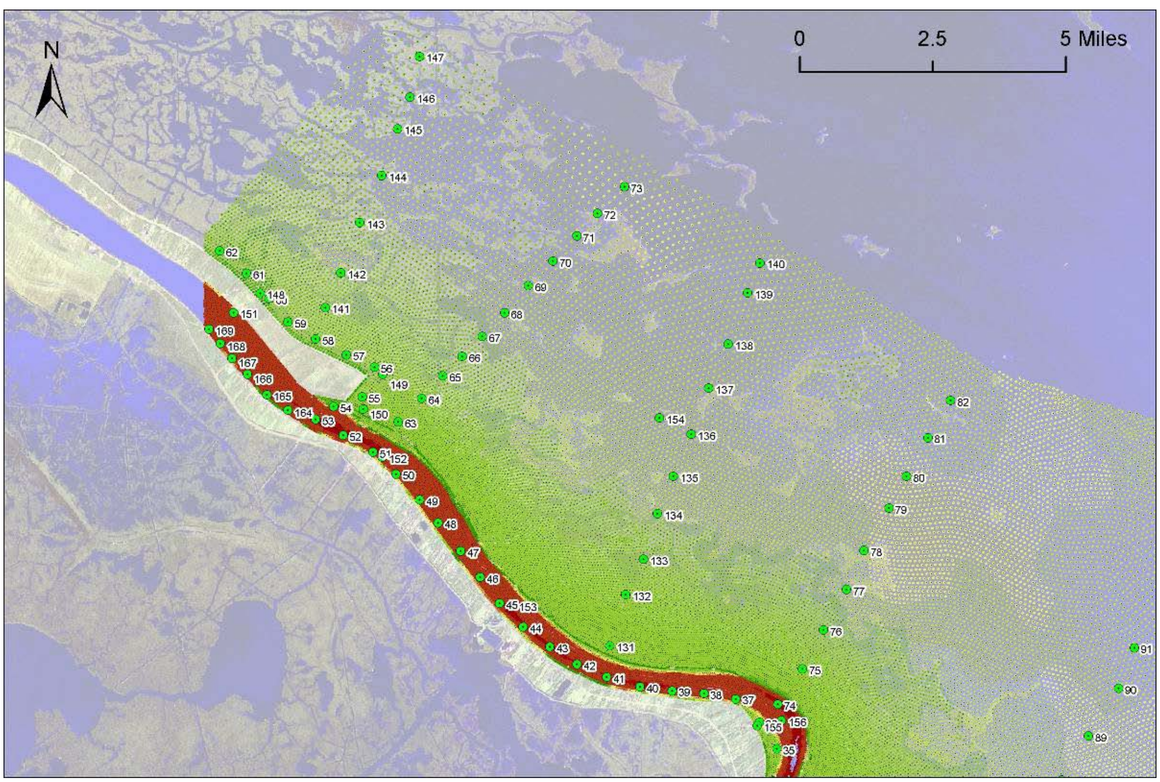


Figure 39 – SWAN Detailed Output Points at Area of Interest – Upper Section

2.7 SUMMARY OF SWAN MODEL RUNS

Specific sensitivity analyses include the following scenarios:

Run 00 – Wind Speed = 77 mph at 45 deg
Run 01 – Wind Speed = 77 mph at 35 deg
Run 02 – Wind Speed = 77 mph at 40 deg
Run 03 – Wind Speed = 77 mph at 50 deg
Run 04 – Wind Speed = 77 mph at 55 deg
Run 05 – Wind Speed = 77 mph at 60 deg
Run 06 – Wind Speed = 77 mph at 65 deg
Run 07 – Wind Speed = 77 mph at 70 deg
Run 08 – Wind Speed = 77 mph at 45 deg with water level amplification factor = 0.85
Run 09 – Wind Speed = 77 mph at 45 deg with water level amplification factor = 1.15
Run 10 – Wind Speed = 88 mph at 45 deg
Run 11 – Wind Speed = 99 mph at 45 deg
Run 12 – Wind Speed = 77 mph at 45 deg with no offshore boundary conditions

3.0 SWAN RESULTS

3.1 SWAN RESULTS FOR SCENARIOS 0 THROUGH 12

SWAN results for 12 different scenarios are shown in this section. Each scenario represents changes to wind speed, surge levels and/or boundary conditions angle. **Table 3**, **Table 5**, and **Table 7** present the significant wave height along the levee of interest for each scenario. **Table 4**, **Table 6**, and **Table 8** show the mean wave period at points along the levee of interest for each scenario.

For a description of H_Sig and TM_10 see **Appendix C** – SWAN Variable Definitions. **Table 3** through **Table 8** shows the maximum wave conditions taken from the series of points immediately in front of the Mississippi River levee. See **Figure 7** and **Figure 8** for the location of the output points.

Figure 9 through **Figure 14** present the recommended design values by river mile. The recommended design values are based on the maximum values of the first eight scenarios (Run 0 through Run 7). Recommended design peak wave periods were calculated by assuming a 4% wave steepness for the given significant wave height. Assuming 4% steepness resulted in significantly higher peak wave periods.

Table 12 – Significant Wave Height with 2% Boundary Conditions

	Significant Wave Height with 2% Boundary Conditions														
		Run 0	**Run 1**	**Run 2**	**Run 3**	**Run 4**	**Run 5**	**Run 6**	**Run 7**	**Run 8**	**Run 9**	**Run 10**	**Run 11**	**Run 12**		
	Wind	70	70	70	70	70	70	70	70	70	70	60	80	70		
	Surge Factor	1	1	1	1	1	1	1	1	0.85	1.15	1	1	1		
	Boundary Angle	45	35	40	50	55	60	65	70	45	45	45	45	n/a	MIN	MAX
Save Point ID	0	2.6	2.5	2.5	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.4	2.7	2.3	2.3	2.7
	2	4.3	4.3	4.3	4.4	4.4	4.5	4.5	4.5	4.3	4.3	3.9	4.8	3.7	3.7	4.8
	4	4.4	4.3	4.4	4.4	4.5	4.5	4.5	4.5	4.4	4.4	4.0	4.8	4.1	4.0	4.8
	6	4.7	4.7	4.7	4.7	4.7	4.8	4.8	4.8	4.7	4.7	4.4	5.1	4.7	4.4	5.1
	8	4.6	4.5	4.6	4.6	4.6	4.7	4.7	4.8	4.6	4.6	4.2	5.0	4.6	4.2	5.0
	10	4.9	4.8	4.8	4.9	5.0	5.1	5.1	5.2	4.9	4.9	4.5	5.3	5.0	4.5	5.3
	12	4.8	4.6	4.7	4.8	4.9	4.9	5.0	5.0	4.8	4.8	4.4	5.2	5.0	4.4	5.2
	14	4.3	4.2	4.3	4.3	4.2	4.2	4.2	4.2	4.3	4.3	4.0	4.5	4.8	4.0	4.5
	16	4.4	4.5	4.5	4.3	4.2	4.0	3.9	3.7	4.4	4.4	3.9	5.0	5.0	3.7	5.0
	18	4.6	4.5	4.6	4.6	4.6	4.6	4.6	4.5	4.6	4.6	4.1	5.2	4.7	4.1	5.2
	20	4.7	4.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.3	5.1	4.2	4.2	5.1
	22	4.7	4.5	4.6	4.8	4.9	5.0	5.0	5.0	4.7	4.7	4.2	5.1	4.3	4.2	5.1
	24	4.8	4.5	4.7	4.9	5.0	5.1	5.2	5.3	4.8	4.8	4.3	5.3	4.0	4.0	5.3
	26	4.8	4.6	4.7	4.9	4.9	4.9	4.9	4.9	4.8	4.8	4.3	5.2	4.4	4.3	5.2
	28	5.4	5.2	5.3	5.4	5.4	5.4	5.3	5.3	5.4	5.3	5.0	5.7	4.5	4.5	5.7
	30	4.8	4.6	4.7	4.8	4.9	4.9	4.9	4.9	4.8	4.7	4.5	5.0	4.0	4.0	5.0
	32	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.7	5.2	3.9	3.9	5.2
	34	4.2	4.1	4.2	4.2	4.2	4.3	4.3	4.3	4.2	4.2	4.0	4.4	3.5	3.5	4.4
	36	4.7	4.6	4.6	4.7	4.7	4.8	4.7	4.8	4.7	4.7	4.4	4.9	4.0	4.0	4.9
	38	4.9	4.9	4.9	4.9	4.9	4.9	4.8	4.9	4.9	4.9	4.6	5.2	4.5	4.5	5.2
	40	4.9	4.7	4.8	4.9	4.9	5.0	4.9	5.0	4.8	4.9	4.4	5.2	4.4	4.4	5.2
	42	4.6	4.3	4.4	4.7	4.7	4.8	4.8	5.0	4.5	4.5	4.1	4.9	4.3	4.1	5.0
	44	4.7	4.5	4.6	4.7	4.7	4.8	4.8	4.9	4.6	4.7	4.3	4.9	4.3	4.3	4.9
	46	4.5	4.4	4.5	4.6	4.5	4.7	4.7	4.9	4.4	4.5	4.2	4.6	4.2	4.2	4.9
	48	4.4	4.4	4.4	4.4	4.4	4.4	4.4	4.5	4.4	4.4	4.1	4.6	4.2	4.1	4.6
	50	4.2	4.1	4.1	4.2	4.2	4.2	4.2	4.3	4.2	4.2	3.9	4.4	3.9	3.9	4.4
	52	4.0	3.8	3.9	4.0	4.0	4.0	4.0	4.0	4.0	4.0	3.7	4.2	3.6	3.6	4.2
	54	2.6	2.5	2.6	2.6	2.7	2.7	2.8	2.8	2.6	2.6	2.5	2.7	2.2	2.2	2.8

Table 13 – Mean Wave Period - with 2% Boundary Conditions

Mean Wave Period - with 2% Boundary Conditions															
	Run 0	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7	Run 8	Run 9	Run 10	Run 11	Run 12		
Wind	70	70	70	70	70	70	70	70	70	70	60	80	70		
Surge Factor	1	1	1	1	1	1	1	1	0.85	1.15	1	1	1		
Boundary Angle	45	35	40	50	55	60	65	70	45	45	45	45	n/a	MIN	MAX
Save Point ID															
0	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.9	2.8	2.8	2.8	2.9	2.3	2.8	2.9
2	3.0	3.0	3.0	3.1	3.1	3.1	3.2	3.2	3.0	3.0	3.0	3.1	2.8	3.0	3.2
4	3.1	3.0	3.0	3.1	3.1	3.2	3.2	3.3	3.1	3.1	3.0	3.1	3.0	3.0	3.3
6	3.2	3.2	3.2	3.2	3.3	3.3	3.3	3.4	3.2	3.2	3.2	3.3	3.2	3.2	3.4
8	3.2	3.1	3.1	3.2	3.2	3.3	3.3	3.3	3.2	3.2	3.1	3.2	3.2	3.1	3.3
10	3.2	3.2	3.2	3.3	3.3	3.4	3.4	3.4	3.2	3.2	3.2	3.3	3.4	3.2	3.4
12	3.2	3.1	3.1	3.3	3.3	3.3	3.4	3.4	3.2	3.2	3.2	3.2	3.4	3.1	3.4
14	3.2	3.1	3.2	3.2	3.3	3.3	3.3	3.3	3.2	3.2	3.1	3.2	3.6	3.1	3.3
16	3.1	3.1	3.1	3.1	3.1	3.0	3.0	2.9	3.1	3.1	3.0	3.3	3.4	2.9	3.3
18	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.0	3.2	3.3	3.0	3.2
20	3.2	3.2	3.2	3.3	3.3	3.3	3.4	3.4	3.2	3.2	3.2	3.3	3.1	3.2	3.4
22	3.2	3.1	3.1	3.2	3.3	3.3	3.4	3.4	3.2	3.2	3.1	3.2	3.1	3.1	3.4
24	3.3	3.1	3.2	3.3	3.4	3.4	3.5	3.5	3.3	3.3	3.2	3.4	2.9	3.1	3.5
26	3.2	3.1	3.1	3.3	3.3	3.3	3.4	3.4	3.2	3.2	3.1	3.3	3.1	3.1	3.4
28	3.5	3.4	3.5	3.6	3.6	3.6	3.7	3.7	3.5	3.5	3.5	3.5	3.1	3.4	3.7
30	3.3	3.2	3.2	3.3	3.4	3.4	3.5	3.5	3.3	3.3	3.3	3.3	2.8	3.2	3.5
32	3.4	3.4	3.4	3.5	3.5	3.6	3.6	3.6	3.4	3.4	3.5	3.4	2.9	3.4	3.6
34	3.2	3.1	3.1	3.2	3.3	3.3	3.3	3.4	3.2	3.2	3.2	3.1	2.7	3.1	3.4
36	3.3	3.2	3.2	3.4	3.4	3.5	3.5	3.5	3.3	3.3	3.3	3.3	3.0	3.2	3.5
38	3.3	3.3	3.3	3.4	3.4	3.5	3.5	3.6	3.3	3.3	3.3	3.3	3.1	3.3	3.6
40	3.3	3.2	3.2	3.4	3.4	3.5	3.5	3.6	3.3	3.3	3.2	3.3	3.1	3.2	3.6
42	3.1	3.0	3.1	3.2	3.3	3.4	3.4	3.5	3.1	3.1	3.0	3.2	3.0	3.0	3.5
44	3.2	3.1	3.1	3.3	3.3	3.4	3.5	3.6	3.2	3.2	3.2	3.2	3.0	3.1	3.6
46	3.2	3.1	3.1	3.3	3.3	3.4	3.4	3.5	3.2	3.2	3.2	3.1	3.0	3.1	3.5
48	3.1	3.0	3.1	3.1	3.2	3.3	3.3	3.4	3.1	3.1	3.1	3.1	3.0	3.0	3.4
50	3.0	2.9	2.9	3.1	3.1	3.2	3.3	3.3	3.0	3.0	3.0	3.0	2.8	2.9	3.3
52	2.9	2.8	2.8	3.0	3.0	3.1	3.1	3.1	2.9	2.9	2.9	2.9	2.7	2.8	3.1
54	3.0	2.7	2.8	3.0	3.1	3.1	3.1	3.1	3.0	3.0	3.0	2.9	2.5	2.7	3.1

Table 14 – Significant Wave Height with 1% Boundary Conditions

Significant Wave Height with 1% Boundary Conditions

		Run 0	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7	Run 8	Run 9	Run 10	Run 11	Run 12		
	Wind Speed (mph)	77	77	77	77	77	77	77	77	77	77	88	99	77		
	Surge Factor	1	1	1	1	1	1	1	1	0.85	1.15	1	1	1		
	Boundary Angle (°naut)	45	35	40	50	55	60	65	70	45	45	45	45	n/a	MIN	MAX
Save Point ID	0	3.3	3.2	3.3	3.3	3.4	3.4	3.4	3.4	2.7	3.9	3.5	3.7	2.9	2.7	3.9
	2	5.3	5.2	5.3	5.3	5.4	5.4	5.5	5.5	4.7	5.9	5.8	6.3	4.5	4.7	6.3
	4	5.4	5.4	5.4	5.5	5.5	5.5	5.5	5.6	4.7	6.1	5.9	6.4	5.1	4.7	6.4
	6	5.8	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.0	6.6	6.2	6.7	5.8	5.0	6.7
	8	5.7	5.6	5.7	5.7	5.8	5.8	5.9	5.9	4.9	6.4	6.1	6.6	5.7	4.9	6.6
	10	5.9	5.8	5.8	5.9	6.0	6.1	6.2	6.2	5.2	6.6	6.4	7.0	6.0	5.2	7.0
	12	5.8	5.7	5.8	5.9	6.0	6.0	6.0	6.0	5.1	6.7	6.3	6.8	6.2	5.1	6.8
	14	5.1	5.1	5.1	5.1	5.1	5.1	5.0	5.0	4.5	5.9	5.4	5.9	5.8	4.5	5.9
	16	5.0	5.1	5.0	4.9	4.7	4.5	4.3	4.1	4.8	5.3	5.6	6.5	5.8	4.1	6.5
	18	5.3	5.3	5.3	5.4	5.3	5.2	5.2	5.2	5.0	5.9	6.0	6.7	5.7	5.0	6.7
	20	5.7	5.6	5.7	5.8	5.8	5.8	5.7	5.7	5.0	6.5	6.1	6.7	5.2	5.0	6.7
	22	5.8	5.6	5.7	5.9	5.9	6.0	6.0	6.1	5.1	6.6	6.3	7.0	5.1	5.1	7.0
	24	5.9	5.7	5.8	6.1	6.1	6.2	6.3	6.4	5.2	6.8	6.4	7.2	4.9	5.2	7.2
	26	5.9	5.7	5.8	6.0	6.1	6.2	6.2	6.3	5.1	6.6	6.4	7.2	5.2	5.1	7.2
	28	6.6	6.4	6.5	6.7	6.6	6.6	6.6	6.6	5.6	7.5	7.0	7.3	5.5	5.6	7.5
	30	6.0	5.8	5.9	6.1	6.2	6.2	6.2	6.2	5.0	6.8	6.4	6.8	4.7	5.0	6.8
	32	6.0	6.0	6.0	6.1	6.0	6.0	6.0	6.0	5.1	7.1	6.2	6.3	4.6	5.1	7.1
	34	5.3	5.2	5.3	5.4	5.4	5.4	5.4	5.5	4.4	6.3	5.6	5.8	4.3	4.4	6.3
	36	5.7	5.6	5.6	5.8	5.8	5.8	5.8	5.9	4.9	6.6	6.0	6.5	4.8	4.9	6.6
	38	5.9	6.0	5.9	5.9	5.8	5.8	5.8	5.8	5.1	6.8	6.2	6.8	5.4	5.1	6.8
	40	5.8	5.7	5.7	5.9	5.9	5.9	5.8	5.7	5.2	6.5	6.2	6.9	5.3	5.2	6.9
	42	5.5	5.2	5.3	5.6	5.6	5.7	5.7	5.8	4.9	6.2	5.9	6.6	5.2	4.9	6.6
	44	5.8	5.6	5.7	5.8	5.8	5.8	5.8	5.9	4.9	6.7	6.1	6.4	5.2	4.9	6.7
	46	5.7	5.6	5.7	5.8	5.9	5.9	6.0	6.1	4.7	6.7	6.0	6.4	5.2	4.7	6.7
	48	5.5	5.5	5.5	5.5	5.5	5.4	5.4	5.5	4.5	6.4	5.7	6.0	5.1	4.5	6.4
	50	5.2	5.0	5.1	5.3	5.2	5.2	5.1	5.2	4.3	6.1	5.4	6.0	4.8	4.3	6.1
	52	5.0	4.7	4.9	5.1	5.0	5.0	4.9	4.9	4.1	5.8	5.3	5.8	4.2	4.1	5.8
	54	3.4	3.3	3.4	3.5	3.5	3.6	3.7	3.7	2.6	4.2	3.6	3.8	2.7	2.6	4.2

Table 15 – Mean Wave Period - with 1% Boundary Conditions

Mean Wave Period - with 1% Boundary Conditions

Save Point ID		Run 0	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7	Run 8	Run 9	Run 10	Run 11	Run 12		
	Wind Speed (mph)	77	77	77	77	77	77	77	77	77	77	88	99	77		
	Surge Factor	1	1	1	1	1	1	1	1	0.85	1.15	1	1	1		
	Boundary Angle (°naut)	45.0	35.0	40.0	50.0	55.0	60.0	65.0	70.0	45.0	45.0	45.0	45.0	n/a	MIN	MAX
	0	3.3	3.2	3.2	3.3	3.3	3.3	3.3	3.3	2.9	3.5	3.3	3.4	2.6	2.6	3.5
	2	3.4	3.3	3.3	3.4	3.5	3.5	3.6	3.6	3.1	3.6	3.4	3.6	3.1	3.1	3.6
	4	3.4	3.4	3.4	3.5	3.5	3.6	3.6	3.7	3.1	3.7	3.5	3.6	3.3	3.1	3.7
	6	3.6	3.5	3.6	3.6	3.7	3.7	3.8	3.8	3.3	3.9	3.6	3.7	3.5	3.3	3.9
	8	3.6	3.5	3.5	3.6	3.7	3.7	3.7	3.8	3.2	3.8	3.6	3.7	3.6	3.2	3.8
	10	3.6	3.5	3.5	3.6	3.7	3.7	3.8	3.8	3.3	3.9	3.6	3.8	3.7	3.3	3.9
	12	3.6	3.4	3.5	3.6	3.7	3.7	3.7	3.8	3.3	3.9	3.6	3.8	3.8	3.3	3.9
	14	3.5	3.4	3.5	3.6	3.6	3.6	3.6	3.6	3.3	3.8	3.6	3.7	3.9	3.3	3.9
	16	3.3	3.3	3.3	3.3	3.2	3.2	3.1	3.0	3.2	3.4	3.4	3.7	3.7	3.0	3.7
	18	3.3	3.3	3.3	3.4	3.4	3.4	3.4	3.4	3.2	3.5	3.5	3.7	3.7	3.2	3.7
	20	3.6	3.5	3.6	3.7	3.7	3.7	3.8	3.7	3.3	3.9	3.7	3.8	3.5	3.3	3.9
	22	3.6	3.5	3.5	3.7	3.7	3.8	3.9	3.9	3.2	3.9	3.7	3.8	3.4	3.2	3.9
	24	3.7	3.5	3.6	3.8	3.8	3.9	4.0	4.0	3.3	4.0	3.8	3.9	3.2	3.2	4.0
	26	3.6	3.4	3.5	3.7	3.8	3.8	3.9	3.9	3.3	3.9	3.7	3.9	3.3	3.3	3.9
	28	3.9	3.8	3.9	4.0	4.1	4.1	4.1	4.2	3.5	4.3	4.0	4.0	3.4	3.4	4.3
	30	3.7	3.6	3.7	3.8	3.9	3.9	4.0	4.0	3.3	4.1	3.7	3.7	3.1	3.1	4.1
	32	3.9	3.7	3.8	3.9	4.0	4.0	4.1	4.1	3.4	4.2	3.8	3.9	3.1	3.1	4.2
	34	3.7	3.5	3.6	3.7	3.8	3.9	3.9	4.0	3.2	4.0	3.6	3.7	3.0	3.0	4.0
	36	3.7	3.5	3.6	3.8	3.9	3.9	4.0	4.0	3.3	4.0	3.7	3.9	3.3	3.3	4.0
	38	3.7	3.6	3.6	3.7	3.8	3.8	3.9	3.9	3.4	4.0	3.7	3.9	3.4	3.4	4.0
	40	3.6	3.5	3.5	3.7	3.7	3.8	3.8	3.8	3.4	3.8	3.7	3.9	3.4	3.4	3.9
	42	3.5	3.3	3.4	3.5	3.6	3.7	3.8	3.8	3.2	3.7	3.5	3.7	3.3	3.2	3.8
	44	3.6	3.5	3.5	3.7	3.8	3.9	3.9	3.9	3.2	3.9	3.6	3.7	3.3	3.2	3.9
	46	3.7	3.5	3.6	3.7	3.8	3.9	4.0	4.0	3.2	4.0	3.7	3.7	3.3	3.2	4.0
	48	3.5	3.4	3.5	3.6	3.7	3.8	3.8	3.9	3.1	3.9	3.5	3.6	3.3	3.1	3.9
	50	3.4	3.2	3.3	3.5	3.6	3.7	3.7	3.8	3.0	3.7	3.4	3.5	3.1	3.0	3.8
	52	3.3	3.1	3.2	3.4	3.5	3.5	3.5	3.6	2.9	3.7	3.3	3.5	2.9	2.9	3.7
	54	3.5	3.0	3.3	3.6	3.7	3.7	3.7	3.7	2.9	3.9	3.4	3.4	2.7	2.7	3.9

Table 16 – Significant Wave Height with 0.2% Boundary Conditions

Significant Wave Height with 0.2% Boundary Conditions

Save Point ID		Run 0	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7	Run 8	Run 9	Run 10	Run 11	Run 12		
	Wind Speed (mph)	88	88	88	88	88	88	88	88	88	88	101	110	88		
	Surge Factor	1	1	1	1	1	1	1	1	0.85	1.15	1	1	1		
	Boundary Angle (°naut)	45	35	40	50	55	60	65	70	45	45	45	45	n/a	MIN	MAX
	0	4.5	4.4	4.5	4.5	4.6	4.6	4.7	4.7	3.7	5.1	4.8	5.0	4.0	3.7	5.1
	2	6.7	6.7	6.8	6.8	6.9	6.9	6.9	6.9	6.0	7.5	7.3	7.8	5.5	6.0	7.8
	4	7.0	6.9	7.0	7.0	7.1	7.1	7.1	7.1	6.1	7.8	7.5	7.9	6.4	6.1	7.9
	6	7.4	7.3	7.4	7.4	7.5	7.5	7.5	7.6	6.5	8.3	7.9	8.2	7.3	6.5	8.3
	8	7.3	7.2	7.4	7.4	7.4	7.5	7.6	7.6	6.4	8.2	7.9	8.2	7.2	6.4	8.2
	10	7.4	7.2	7.5	7.5	7.6	7.7	7.8	7.9	6.6	8.3	8.0	8.4	7.5	6.6	8.4
	12	7.5	7.3	7.6	7.6	7.6	7.7	7.7	7.8	6.6	8.3	8.0	8.4	7.8	6.6	8.4
	14	6.5	6.4	6.4	6.4	6.4	6.4	6.4	6.4	5.6	7.3	6.8	7.0	7.2	5.6	7.3
	16	5.8	6.1	5.7	5.7	5.5	5.3	5.0	4.8	5.6	6.1	6.5	7.1	7.7	4.8	7.1
	18	6.6	6.6	6.6	6.6	6.5	6.5	6.4	6.3	6.1	7.2	7.3	7.9	6.8	6.1	7.9
	20	7.2	7.1	7.3	7.3	7.3	7.3	7.2	7.2	6.4	8.1	7.8	8.2	6.6	6.4	8.2
	22	7.4	7.2	7.4	7.4	7.5	7.6	7.6	7.7	6.5	8.3	7.9	8.4	6.3	6.5	8.4
	24	7.6	7.4	7.7	7.7	7.8	7.9	7.9	8.0	6.7	8.5	8.2	8.7	6.0	6.7	8.7
	26	7.4	7.2	7.6	7.6	7.7	7.8	7.9	7.9	6.6	8.4	8.1	8.6	6.4	6.6	8.6
	28	8.2	8.1	8.3	8.3	8.3	8.2	8.1	8.1	7.2	9.2	8.7	9.1	6.5	7.2	9.2
	30	7.5	7.3	7.6	7.6	7.7	7.8	7.7	7.8	6.6	8.5	8.0	8.4	5.7	6.6	8.5
	32	7.6	7.5	7.7	7.7	7.7	7.7	7.6	7.7	6.6	8.7	8.0	8.2	5.5	6.6	8.7
	34	6.9	6.7	6.9	6.9	6.9	7.0	7.0	7.1	5.8	7.9	7.1	7.3	5.2	5.8	7.9
	36	7.3	7.0	7.4	7.4	7.5	7.5	7.5	7.6	6.3	8.2	7.7	7.9	5.8	6.3	8.2
	38	7.4	7.3	7.4	7.4	7.4	7.3	7.3	7.3	6.5	8.2	7.8	8.1	6.7	6.5	8.2
	40	7.1	7.1	7.2	7.2	7.2	7.2	7.1	7.1	6.4	7.8	7.7	8.1	6.0	6.4	8.1
	42	6.8	6.5	6.9	6.9	6.9	7.0	7.0	7.0	6.1	7.6	7.3	7.7	6.3	6.1	7.7
	44	7.3	7.1	7.2	7.2	7.2	7.2	7.3	7.3	6.3	8.2	7.7	7.9	6.4	6.3	8.2
	46	7.2	7.0	7.3	7.3	7.4	7.5	7.5	7.6	6.1	8.3	7.6	7.9	6.4	6.1	8.3
	48	6.9	6.9	6.9	6.9	6.9	6.9	7.0	7.1	5.8	7.9	7.2	7.5	6.3	5.8	7.9
	50	6.6	6.4	6.7	6.7	6.7	6.8	6.9	7.0	5.7	7.5	7.0	7.2	6.2	5.7	7.5
	52	6.3	5.9	6.4	6.4	6.5	6.6	6.7	6.8	5.4	7.2	6.7	7.1	5.3	5.4	7.2
	54	4.6	4.3	4.7	4.7	4.7	4.8	4.9	5.0	3.6	5.5	4.8	4.9	3.5	3.6	5.5

Table 17 – Mean Wave Period with 0.2% Boundary Conditions

Mean Wave Period - with 0.2% Boundary Conditions

	Run 0	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7	Run 8	Run 9	Run 10	Run 11	Run 12		
Wind Speed (mph)	88	88	88	88	88	88	88	88	88	88	101	110	88		
Surge Factor	1	1	1	1	1	1	1	1	0.85	1.15	1	1	1		
Boundary Angle (°naut)	45.0	35.0	40.0	50.0	55.0	60.0	65.0	70.0	45.0	45.0	45.0	45.0	n/a	MIN	MAX
Save Point ID															
0	3.7	3.6	3.7	3.7	3.8	3.8	3.8	3.8	3.4	3.9	3.8	3.8	3.0	3.0	3.9
2	3.8	3.7	3.9	3.9	3.9	4.0	4.0	4.0	3.5	4.0	3.9	4.0	3.5	3.5	4.0
4	3.9	3.8	4.0	4.0	4.1	4.1	4.2	4.2	3.6	4.2	4.0	4.1	3.8	3.6	4.2
6	4.1	4.0	4.1	4.1	4.2	4.2	4.3	4.3	3.7	4.4	4.1	4.2	4.0	3.7	4.4
8	4.1	3.9	4.1	4.1	4.2	4.2	4.3	4.3	3.7	4.3	4.1	4.2	4.1	3.7	4.3
10	4.1	3.9	4.2	4.2	4.2	4.3	4.3	4.4	3.8	4.3	4.1	4.2	4.2	3.8	4.4
12	4.2	4.0	4.3	4.3	4.3	4.4	4.5	4.5	3.9	4.4	4.3	4.3	4.3	3.9	4.5
14	4.0	3.8	4.1	4.1	4.1	4.2	4.2	4.2	3.7	4.2	4.1	4.2	4.5	3.7	4.5
16	3.5	3.6	3.5	3.5	3.4	3.4	3.3	3.2	3.4	3.6	3.7	3.8	4.3	3.2	4.3
18	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.5	3.9	3.8	3.9	4.1	3.5	4.1
20	4.1	4.0	4.2	4.2	4.2	4.2	4.3	4.3	3.8	4.4	4.2	4.2	4.0	3.8	4.4
22	4.1	3.9	4.2	4.2	4.3	4.3	4.4	4.4	3.8	4.4	4.2	4.3	3.8	3.8	4.4
24	4.2	4.0	4.3	4.3	4.4	4.4	4.5	4.5	3.9	4.5	4.3	4.4	3.6	3.6	4.5
26	4.1	3.9	4.2	4.2	4.2	4.3	4.4	4.4	3.8	4.4	4.2	4.2	3.7	3.7	4.4
28	4.5	4.3	4.6	4.6	4.6	4.7	4.7	4.7	4.1	4.8	4.5	4.5	3.8	3.8	4.8
30	4.2	4.0	4.3	4.3	4.4	4.5	4.5	4.5	3.9	4.6	4.2	4.3	3.4	3.4	4.6
32	4.4	4.2	4.5	4.5	4.6	4.7	4.7	4.7	4.0	4.8	4.4	4.4	3.4	3.4	4.8
34	4.2	3.9	4.3	4.3	4.4	4.5	4.6	4.6	3.8	4.5	4.2	4.2	3.3	3.3	4.6
36	4.2	4.0	4.4	4.4	4.5	4.6	4.6	4.7	3.9	4.5	4.3	4.3	3.6	3.6	4.7
38	4.1	4.0	4.2	4.2	4.3	4.4	4.4	4.4	3.8	4.3	4.2	4.2	3.9	3.8	4.4
40	4.0	3.9	4.1	4.1	4.2	4.2	4.3	4.3	3.8	4.2	4.1	4.2	3.5	3.5	4.3
42	3.9	3.6	4.0	4.0	4.1	4.2	4.2	4.2	3.6	4.1	3.9	4.0	3.6	3.6	4.2
44	4.1	3.9	4.2	4.2	4.3	4.4	4.4	4.5	3.7	4.3	4.1	4.2	3.6	3.6	4.5
46	4.1	3.9	4.3	4.3	4.4	4.4	4.5	4.5	3.7	4.5	4.2	4.2	3.6	3.6	4.5
48	4.0	3.8	4.1	4.1	4.2	4.3	4.4	4.5	3.6	4.3	4.0	4.1	3.6	3.6	4.5
50	3.8	3.6	4.0	4.0	4.1	4.2	4.3	4.4	3.5	4.1	3.9	3.9	3.5	3.5	4.4
52	3.8	3.5	3.9	3.9	4.0	4.1	4.2	4.3	3.4	4.1	3.9	3.9	3.3	3.3	4.3
54	4.1	3.5	4.2	4.2	4.3	4.3	4.3	4.3	3.5	4.4	4.1	4.0	3.1	3.1	4.4

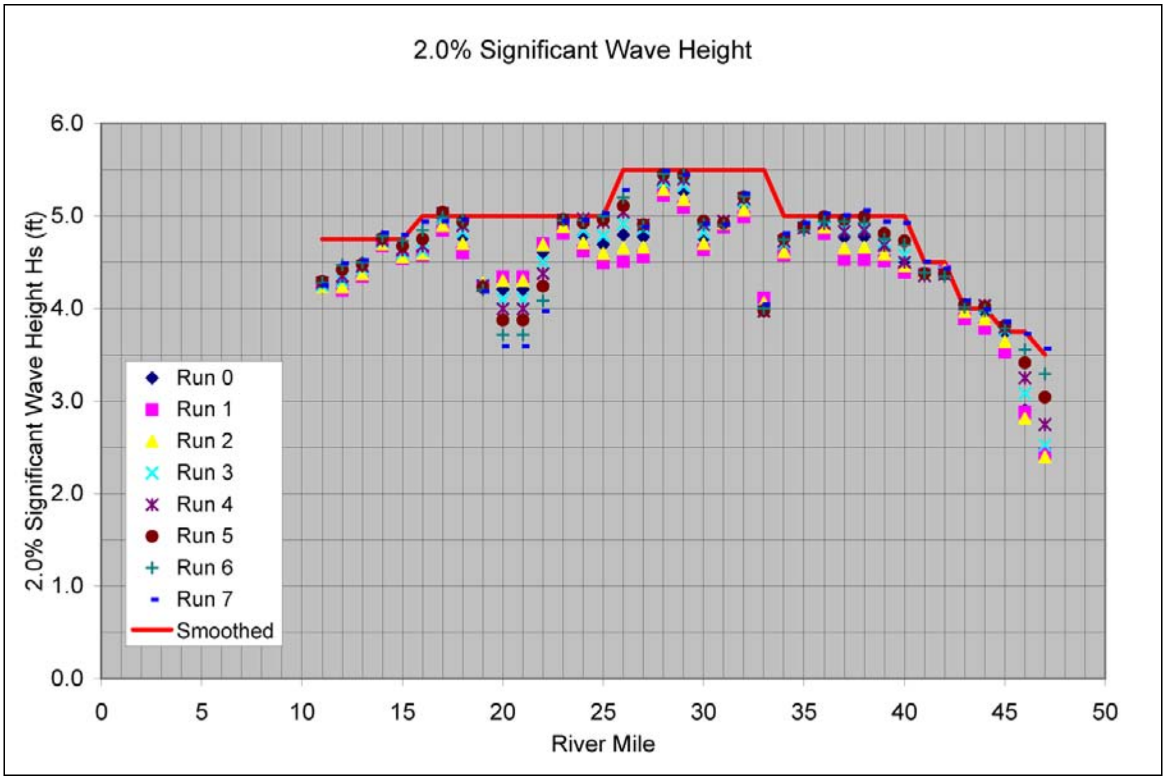


Figure 40 – 2.0% SWAN Significant Wave Height Results and Recommended Design Values

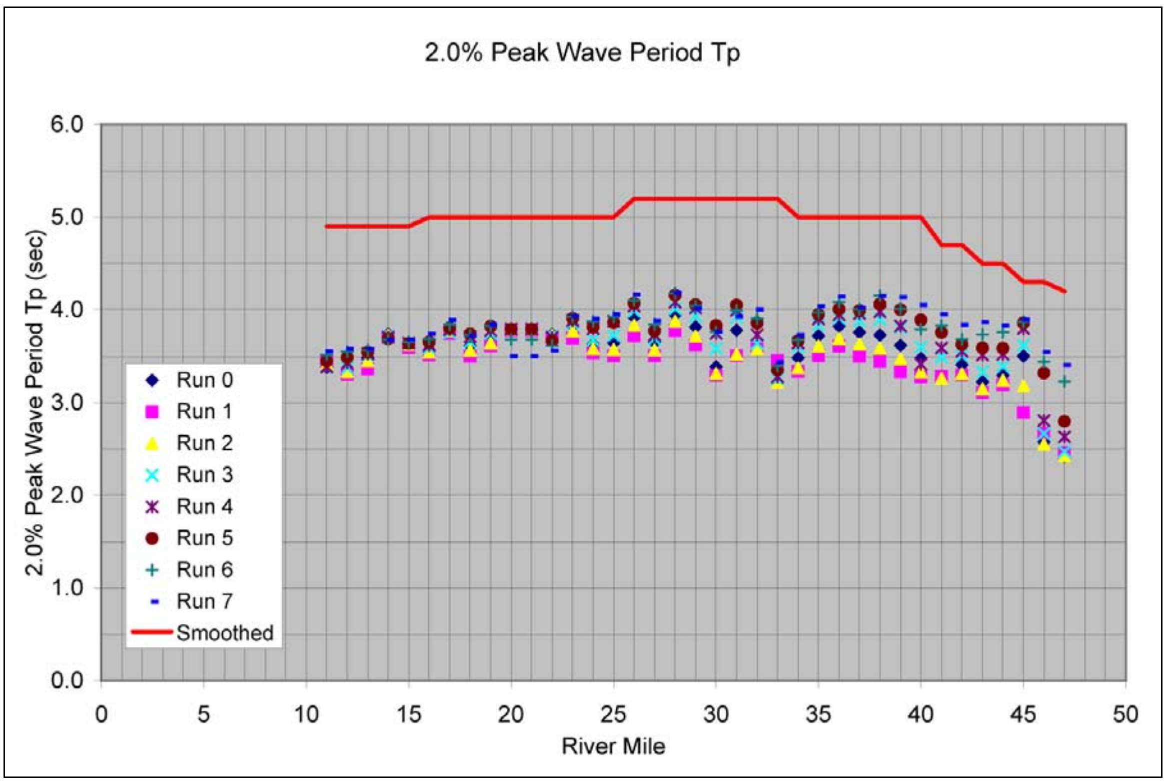


Figure 41 – 2.0% SWAN Peak Wave Period Results and Recommended Design Values

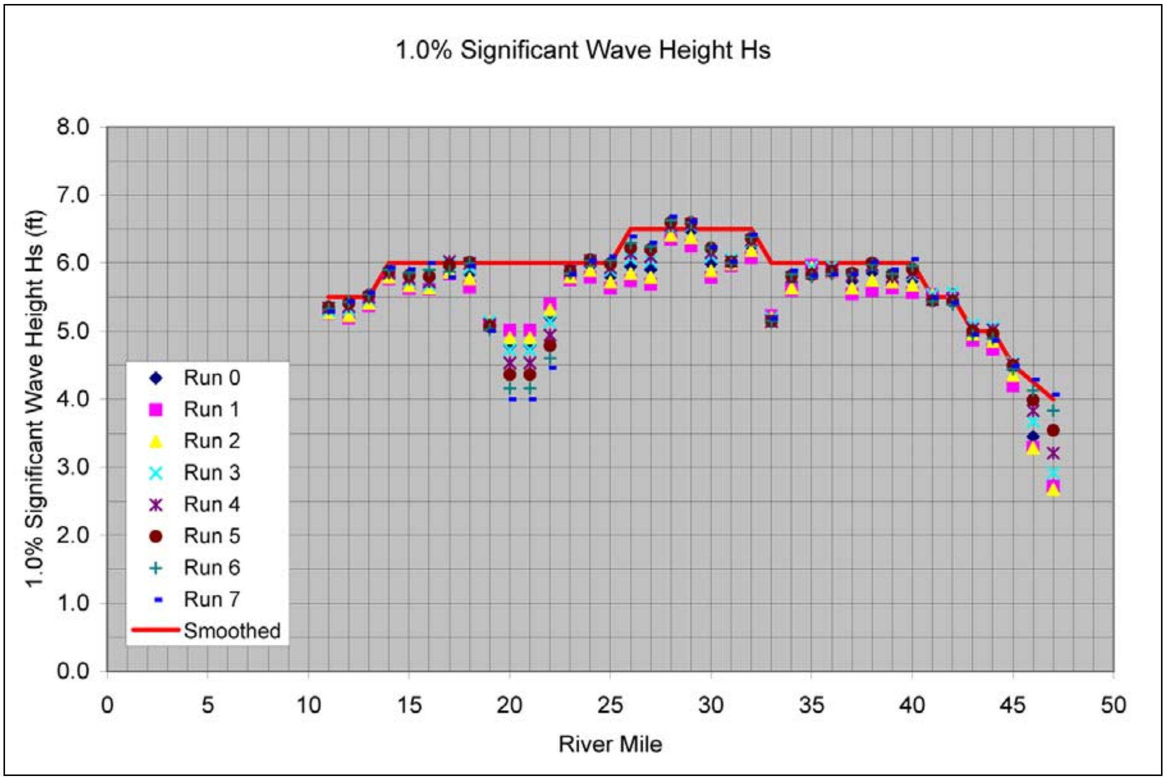


Figure 42 – 1.0% SWAN Significant Wave Height Results and Recommended Design Values

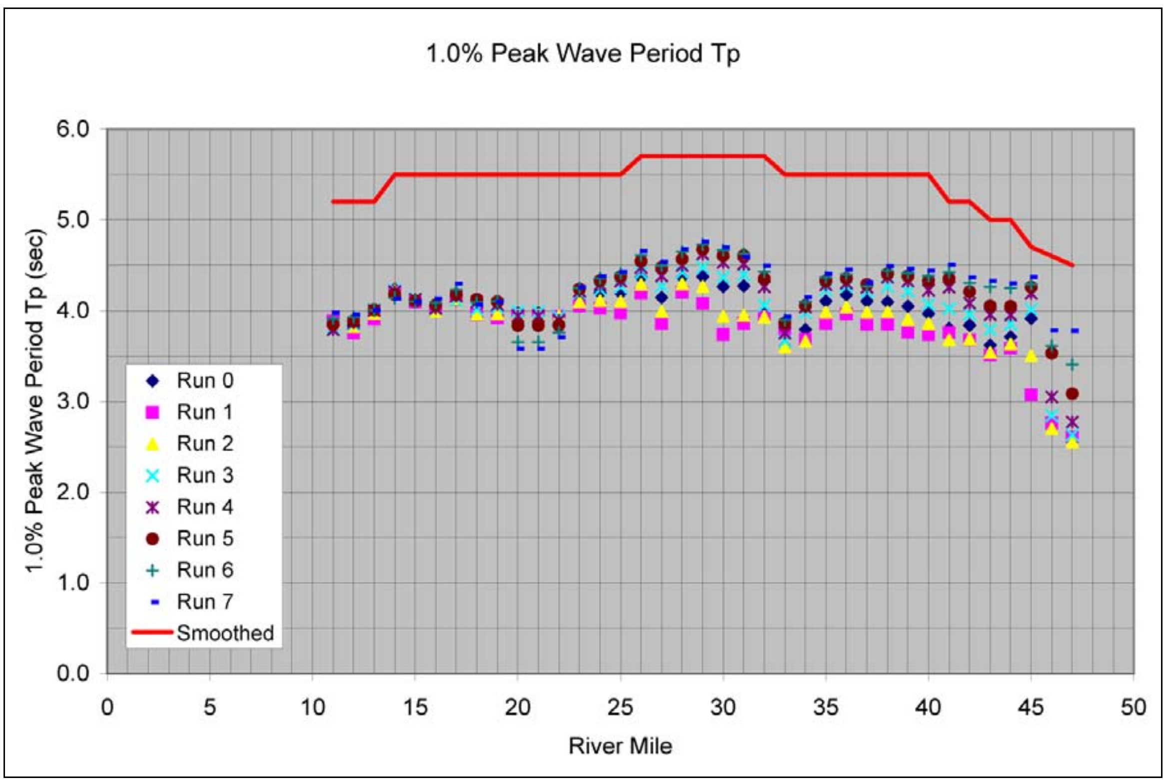


Figure 43 – 1.0% SWAN Peak Wave Period Results and Recommended Design Values

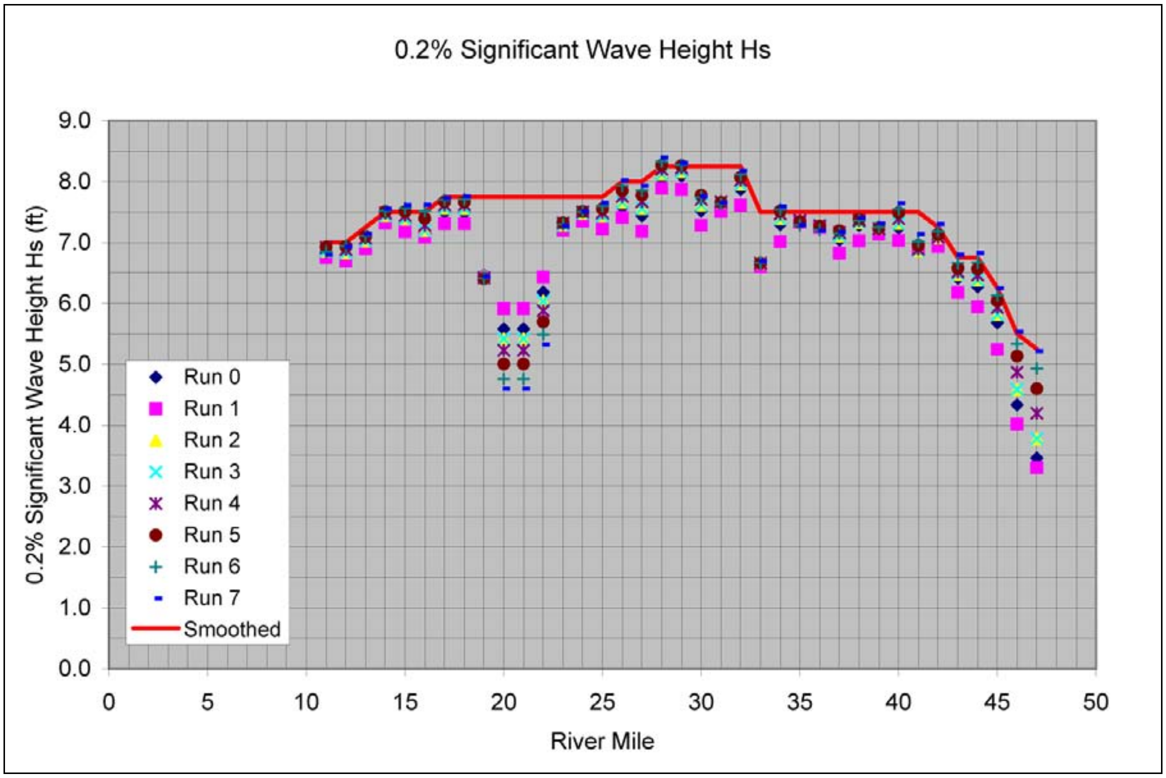


Figure 44 – 0.2% SWAN Significant Wave Height Results and Recommended Design Values

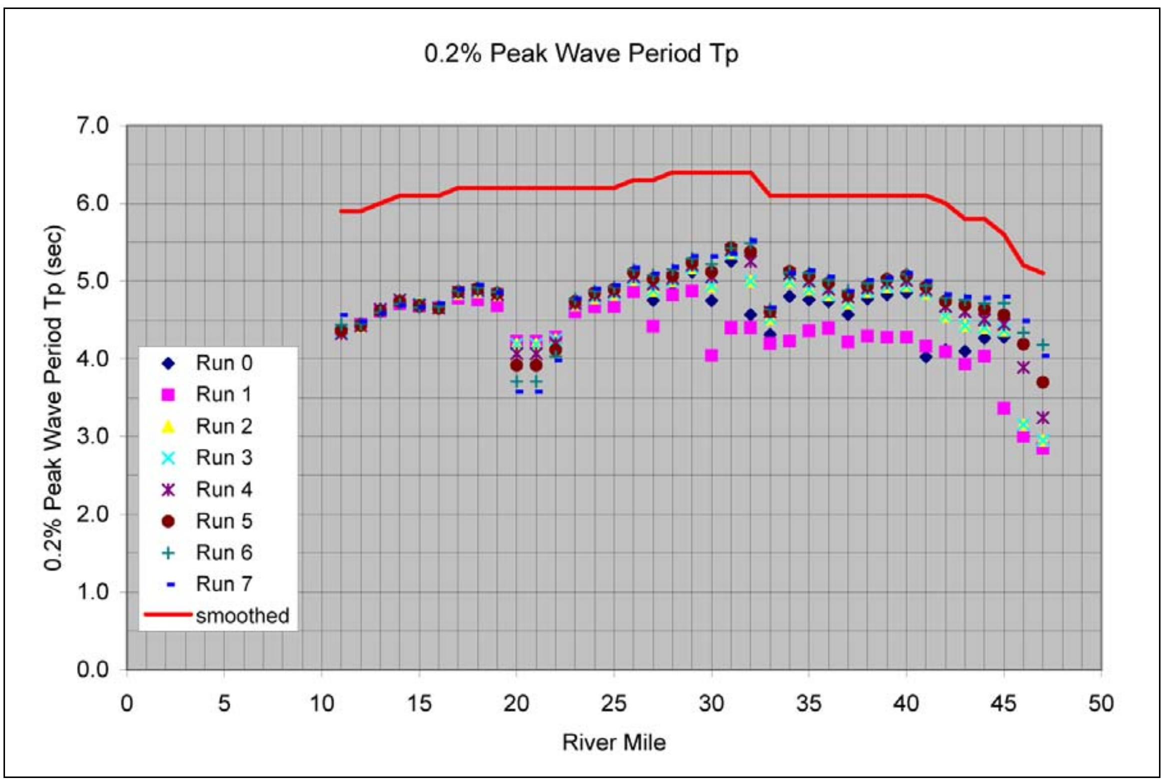


Figure 45 – 0.2% SWAN Peak Wave Period Results and Recommended Design Values

3.2 2D SWAN RESULTS

SWAN 2D results with 1% boundary conditions for each scenario are provided in **Appendix J1**.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

The results in **Table 3** though **Table 8** present a range of possible wave conditions at the levee of interest. The maximums values from scenarios 0 through 7 were selected for the 1% wave conditions. **Table 9** summarizes the recommended design wave values at each river mile.

Table 18 – Recommended Design 0.2%, 1.0%, and 2.0% Wave Conditions at River Miles

Recommended 0.2%, 1% and 2% Wave Conditions at MRL Below RM 44						
River Mile	**0.2% Significant Wave Height (ft)**	**0.2% Peak Period Tp (s)**	**1% Significant Wave Height (ft)**	**1% Peak Period Tp (s)**	**2% Significant Wave Height (ft)**	**2% Peak Period Tp (s)**
11	7.00	5.9	5.50	5.2	4.75	4.9
12	7.00	5.9	5.50	5.2	4.75	4.9
13	7.25	6.0	5.50	5.2	4.75	4.9
14	7.50	6.1	6.00	5.5	4.75	4.9
15	7.50	6.1	6.00	5.5	4.75	4.9
16	7.50	6.1	6.00	5.5	5.00	5.0
17	7.75	6.2	6.00	5.5	5.00	5.0
18	7.75	6.2	6.00	5.5	5.00	5.0
19	7.75	6.2	6.00	5.5	5.00	5.0
20	7.75	6.2	6.00	5.5	5.00	5.0
21	7.75	6.2	6.00	5.5	5.00	5.0
22	7.75	6.2	6.00	5.5	5.00	5.0
23	7.75	6.2	6.00	5.5	5.00	5.0
24	7.75	6.2	6.00	5.5	5.00	5.0
25	7.75	6.2	6.00	5.5	5.00	5.0
26	8.00	6.3	6.50	5.7	5.50	5.2
27	8.00	6.3	6.50	5.7	5.50	5.2
28	8.25	6.4	6.50	5.7	5.50	5.2
29	8.25	6.4	6.50	5.7	5.50	5.2
30	8.25	6.4	6.50	5.7	5.50	5.2
31	8.25	6.4	6.50	5.7	5.50	5.2
32	8.25	6.4	6.50	5.7	5.50	5.2
33	7.50	6.1	6.00	5.5	5.50	5.2
34	7.50	6.1	6.00	5.5	5.00	5.0
35	7.50	6.1	6.00	5.5	5.00	5.0
36	7.50	6.1	6.00	5.5	5.00	5.0
37	7.50	6.1	6.00	5.5	5.00	5.0
38	7.50	6.1	6.00	5.5	5.00	5.0
39	7.50	6.1	6.00	5.5	5.00	5.0
40	7.50	6.1	6.00	5.5	5.00	5.0
41	7.50	6.1	5.50	5.2	4.50	4.7
42	7.25	6.0	5.50	5.2	4.50	4.7
43	6.75	5.8	5.00	5.0	4.00	4.5
44	6.75	5.8	5.00	5.0	4.00	4.5
45	6.25	5.6	4.50	4.7	3.75	4.3
46	5.50	5.2	4.25	4.6	3.75	4.3
47	5.25	5.1	4.00	4.5	3.50	4.2

Appendix K1– 1% Boundary Conditions - SWAN 2D Results

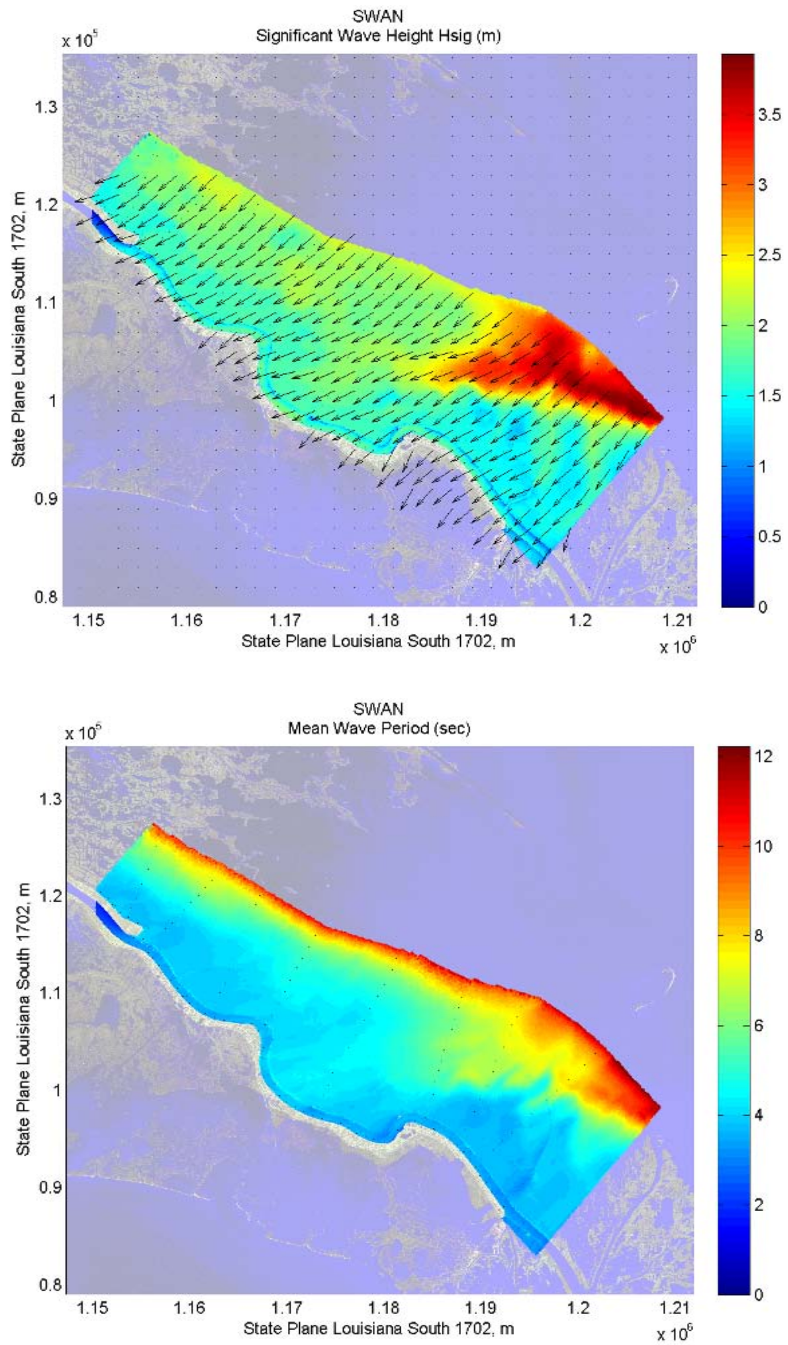
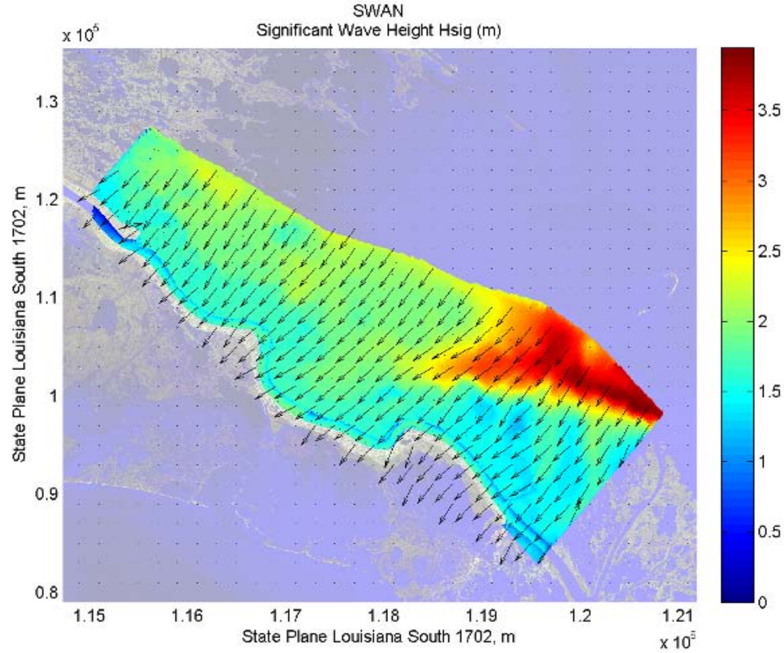


Figure 15 – Scenario 0 – Wind Speed = 77 mph at 45 deg - Significant Wave Height (m) and Mean Wave Period (s)



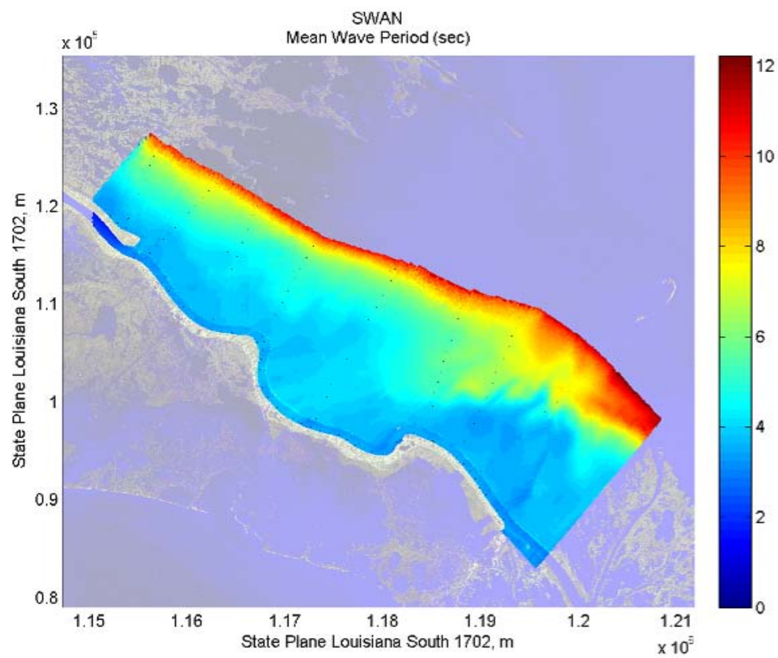
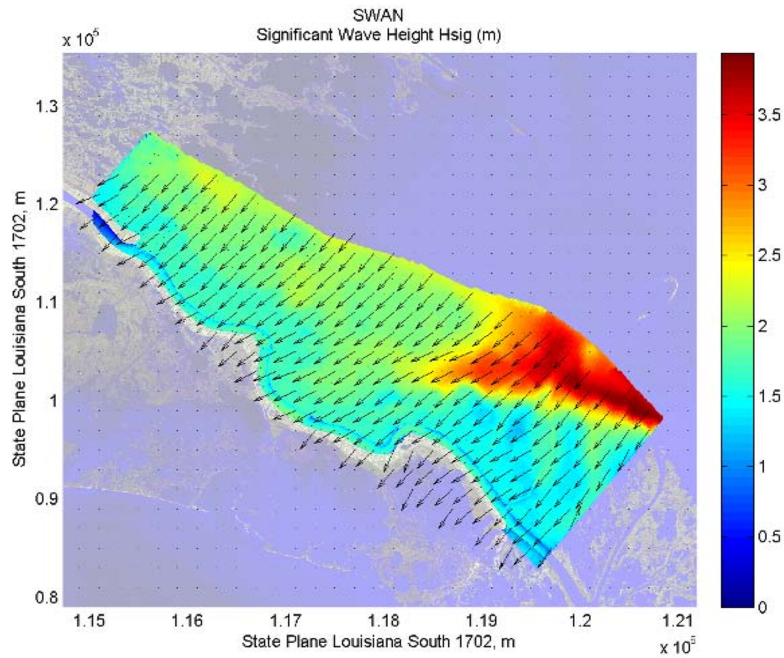


Figure 16 – Scenario 1 – Wind Speed = 77 mph at 35 deg - Significant Wave Height (m) and Mean Wave Period (a)



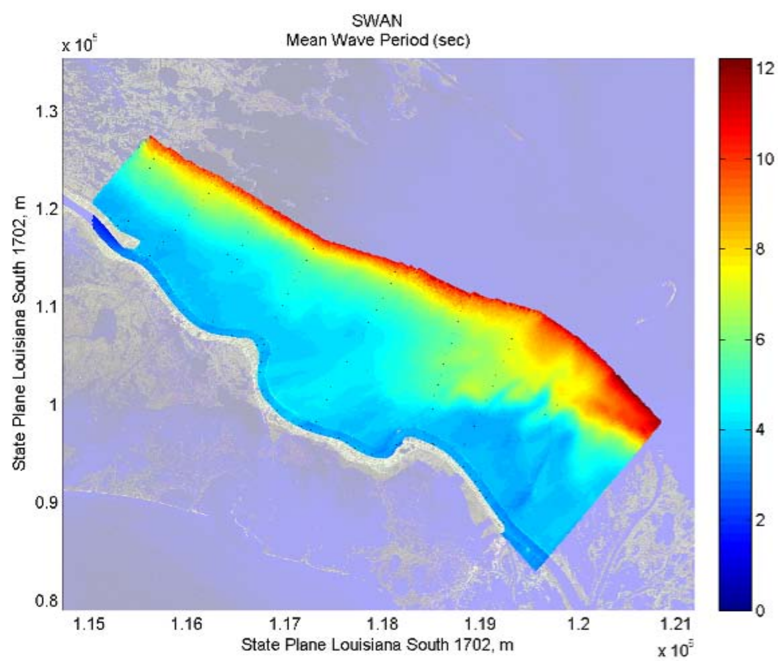
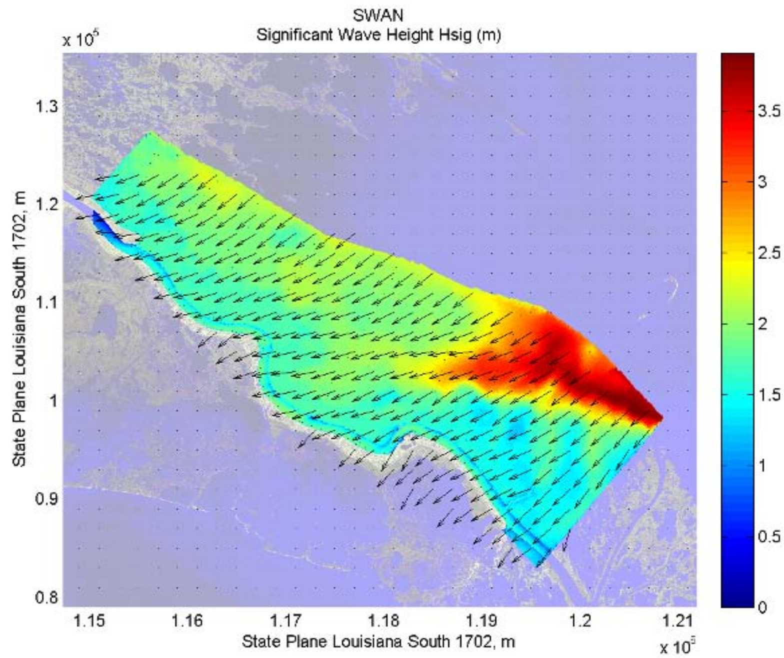


Figure 17 – Scenario 2 – Wind Speed = 77 mph at 40 deg - Significant Wave Height (m) and Mean Wave Period (s)



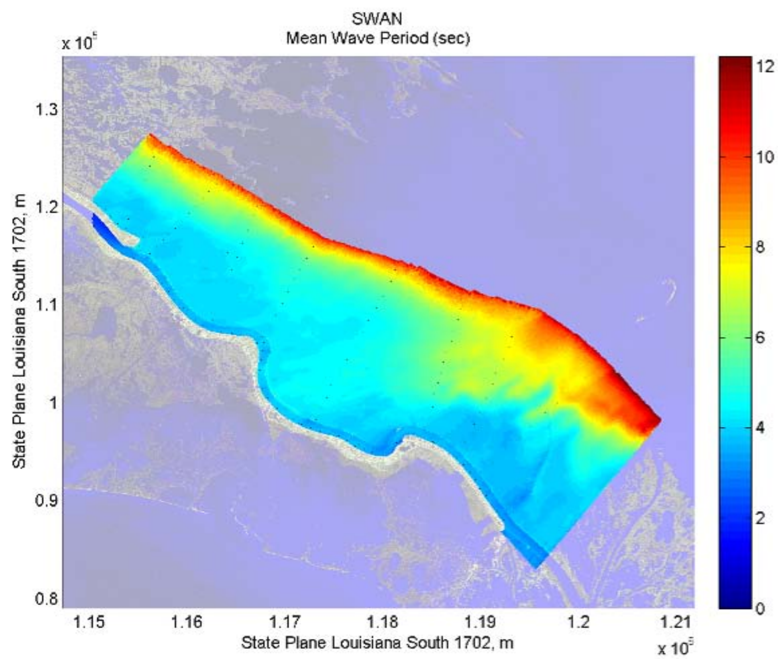
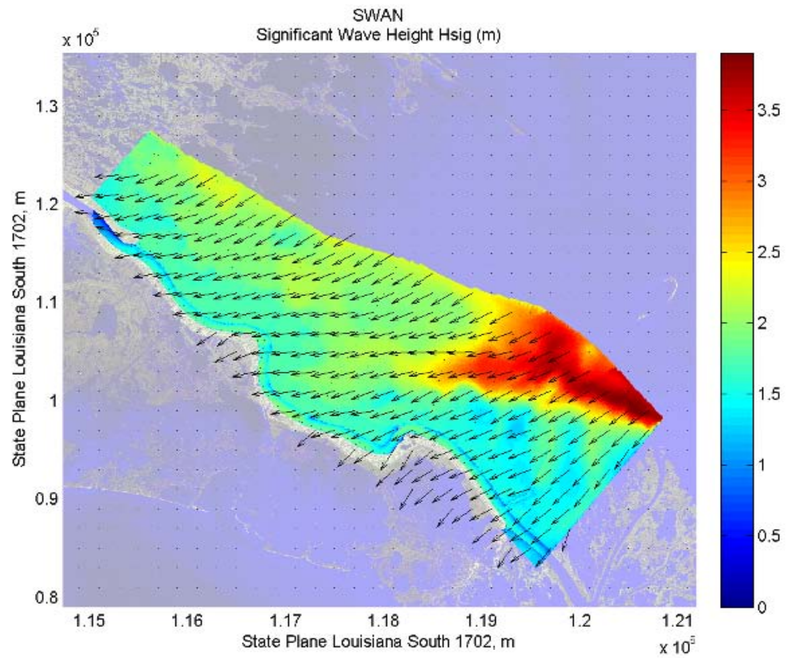


Figure 18 – Scenario 3 – Wind Speed = 77 mph at 50 deg - Significant Wave Height (m) and Mean Wave Period (s



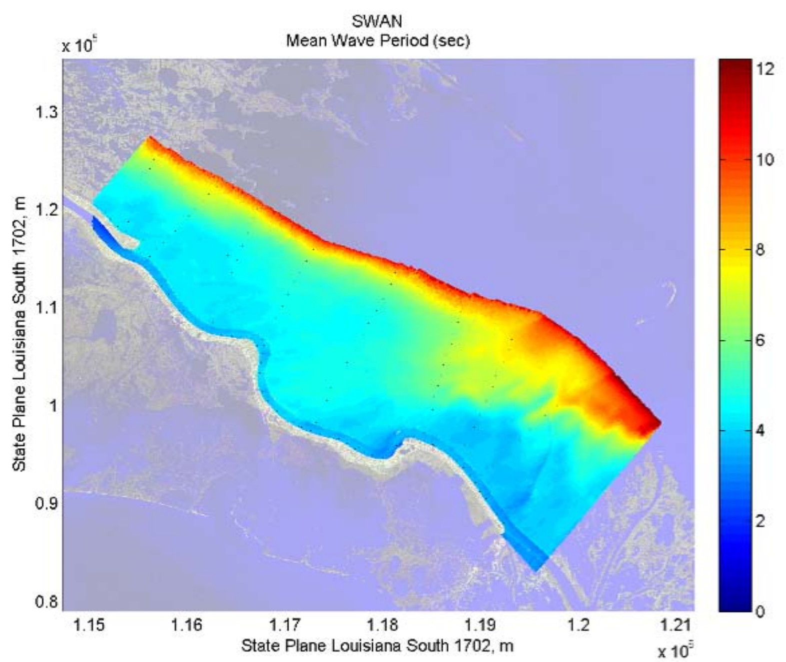
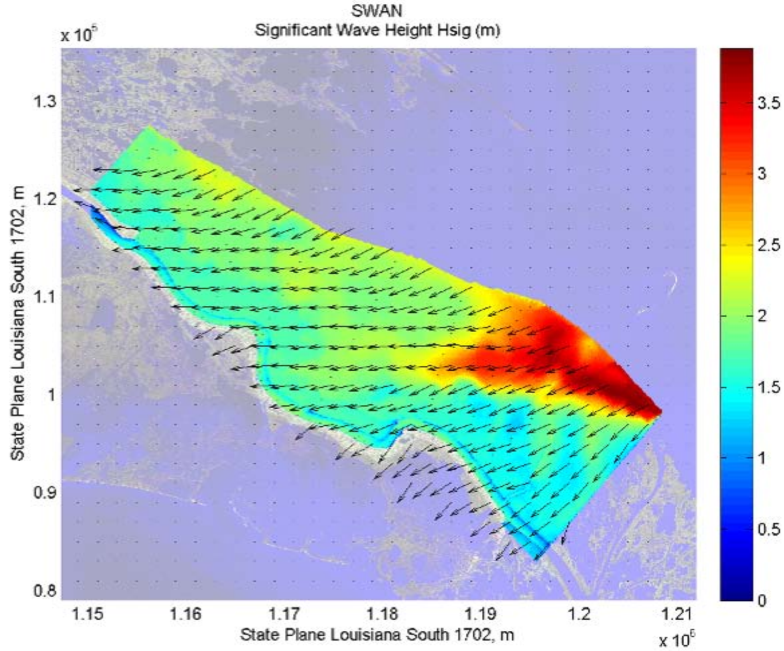


Figure 19 – Scenario 4 – Wind Speed = 77 mph at 55 deg - Significant Wave Height (m) and Mean Wave Period (s)



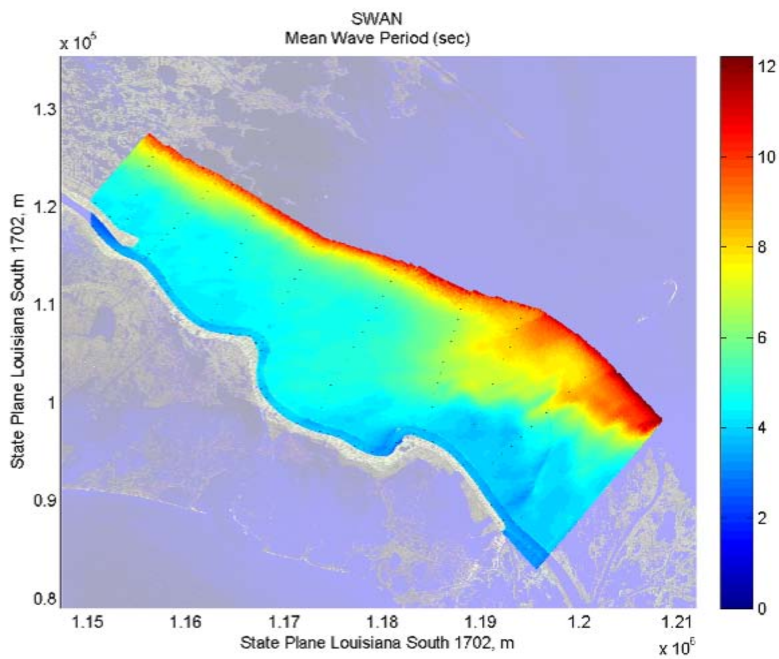
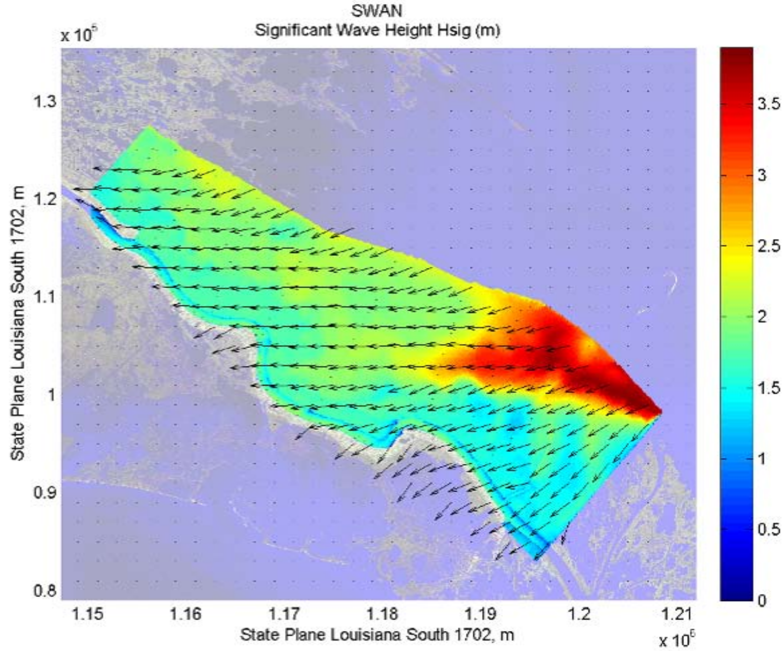


Figure 20 – Scenario 5 – Wind Speed = 77 mph at 60 deg - Significant Wave Height (m) and Mean Wave Period (s)



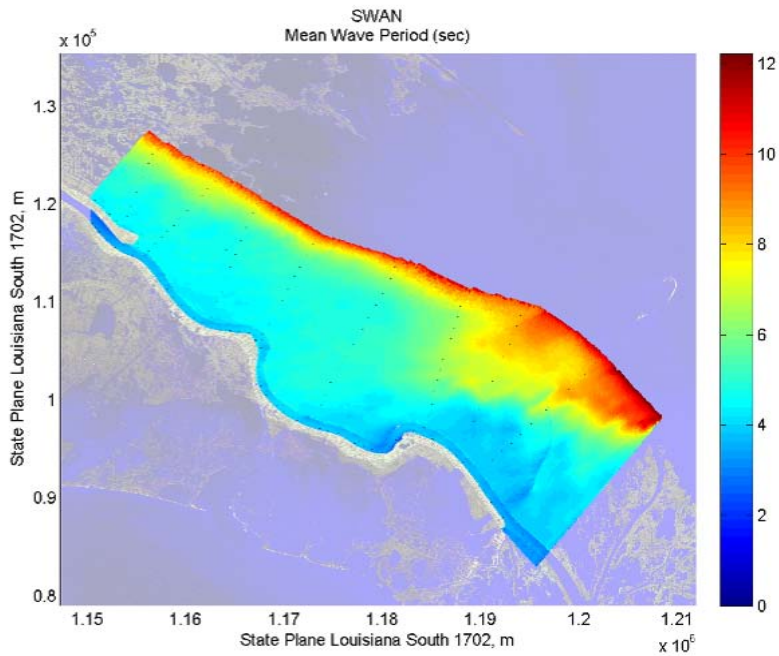
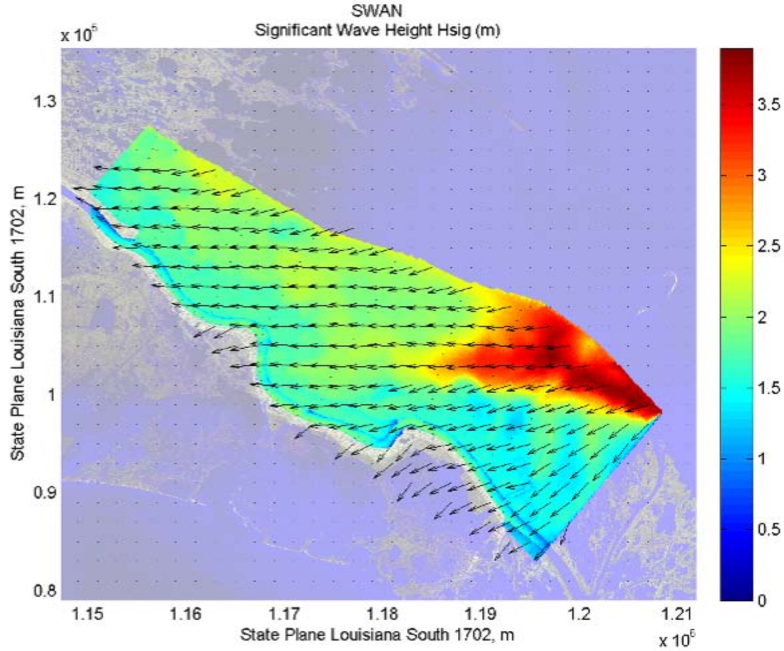


Figure 21 – Scenario 6 – Wind Speed = 77 mph at 65 deg - Significant Wave Height (m) and Mean Wave Period (s)



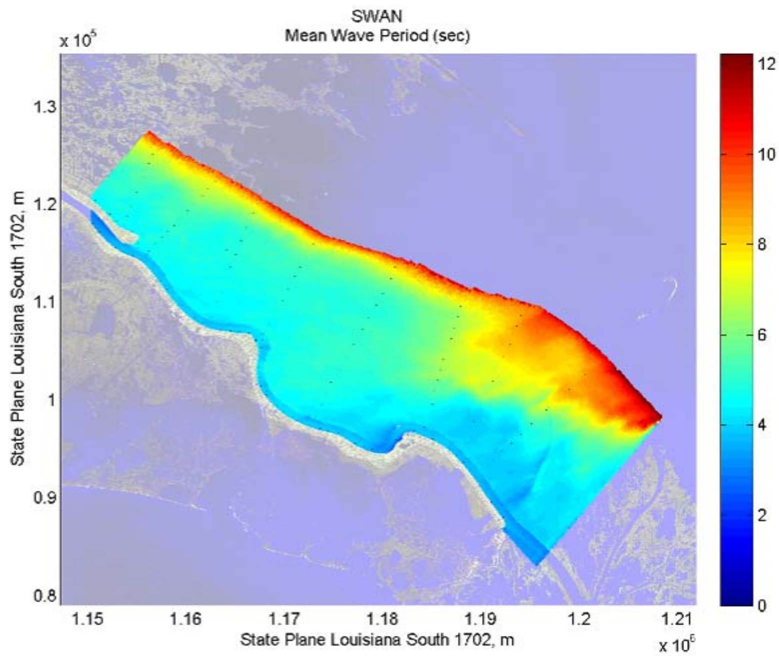


Figure 22 – Scenario 7 – Wind Speed = 77 mph at 70 deg - Significant Wave Height (m) and Mean Wave Period (s)

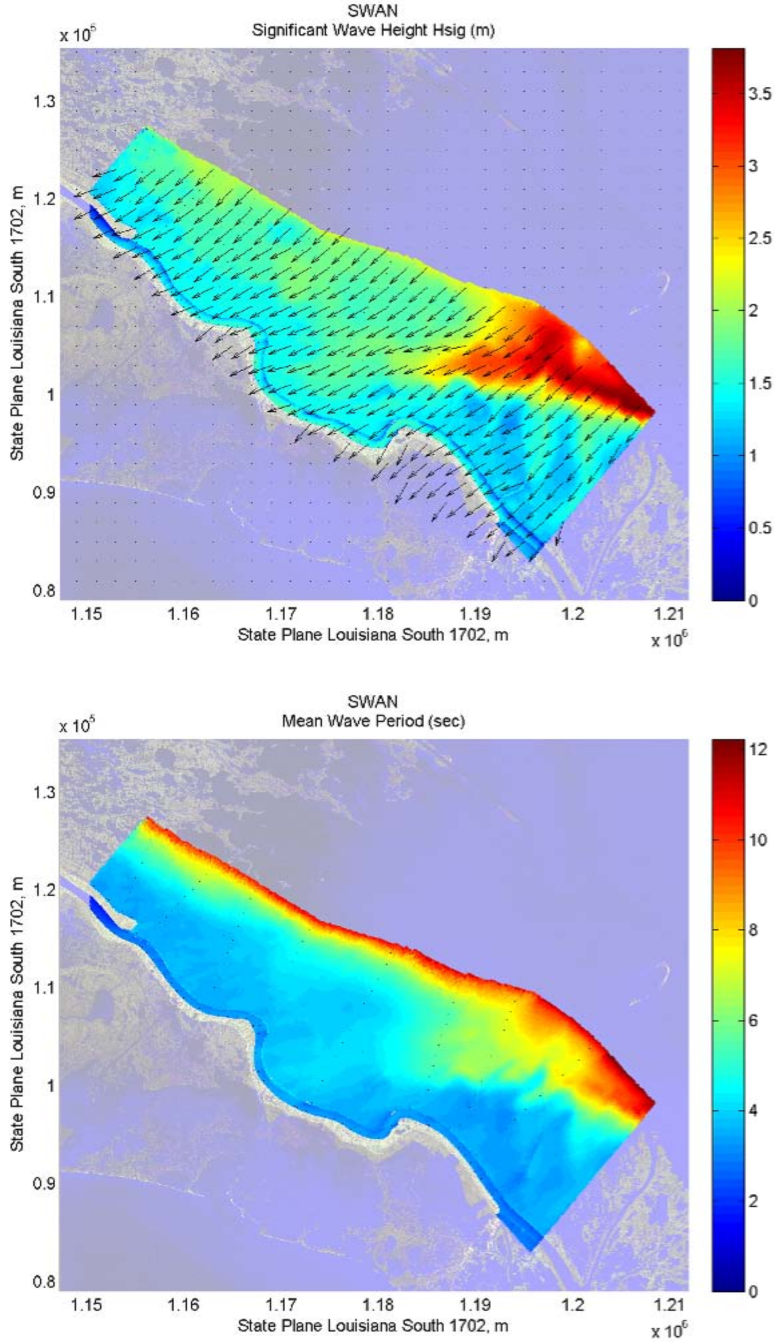
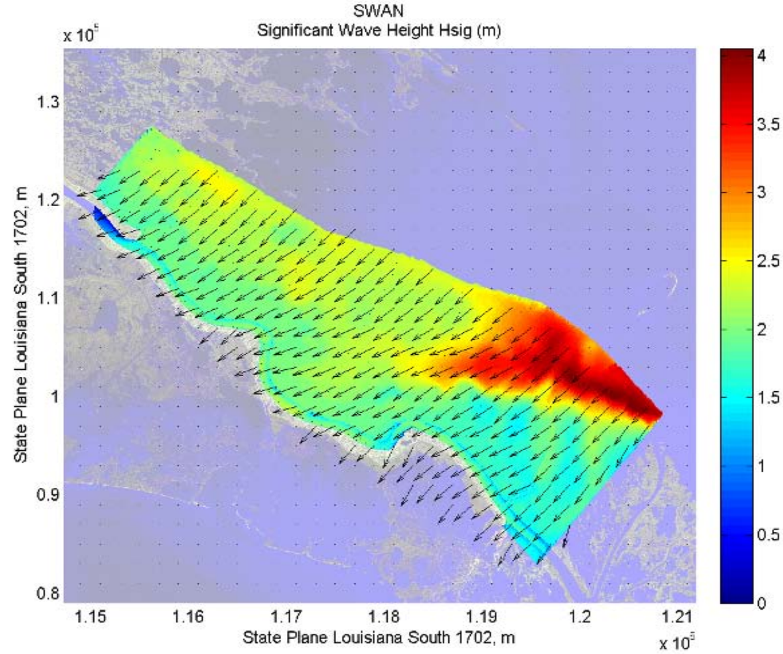


Figure 23 – Scenario 8 – Wind Speed = 77 mph at 45 deg with water level amplification factor = 0.85



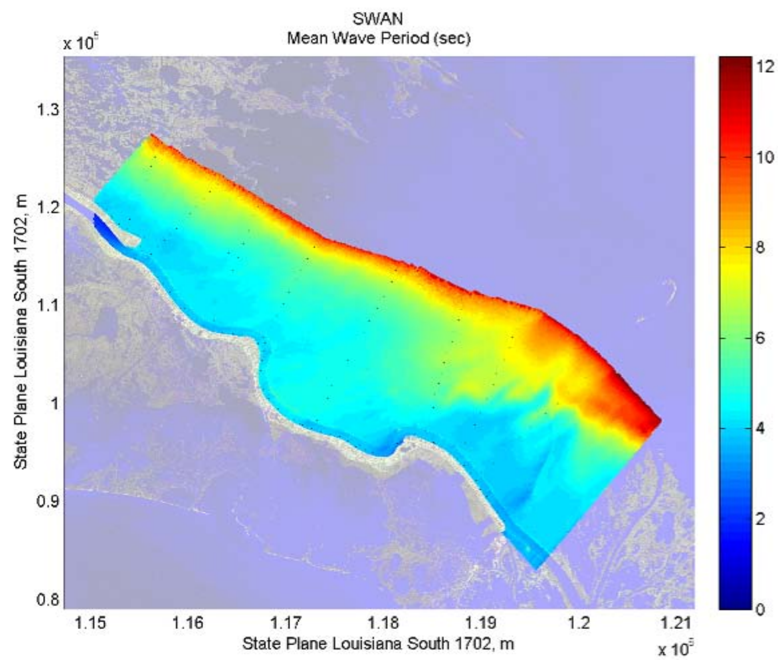
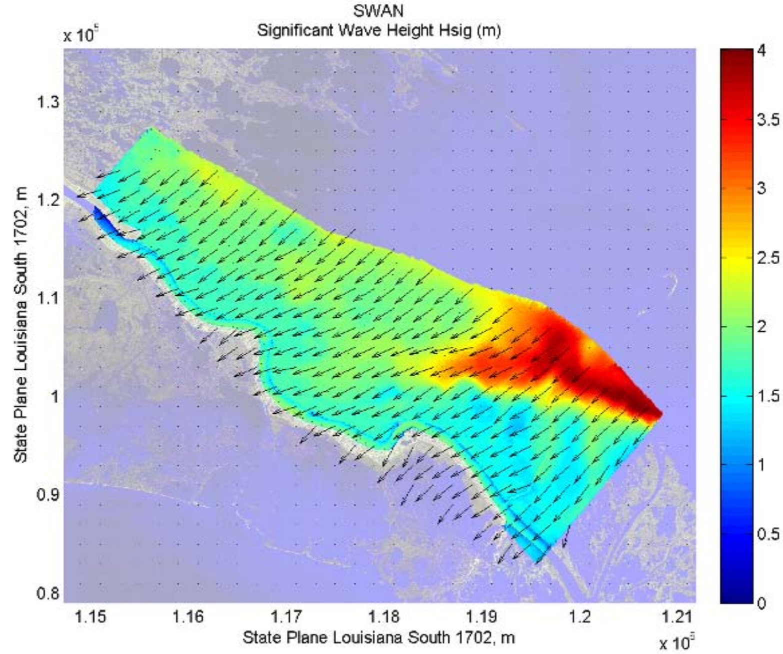


Figure 24 – Scenario 9 – Wind Speed = 77 mph at 45 deg with water level amplification factor = 1.15



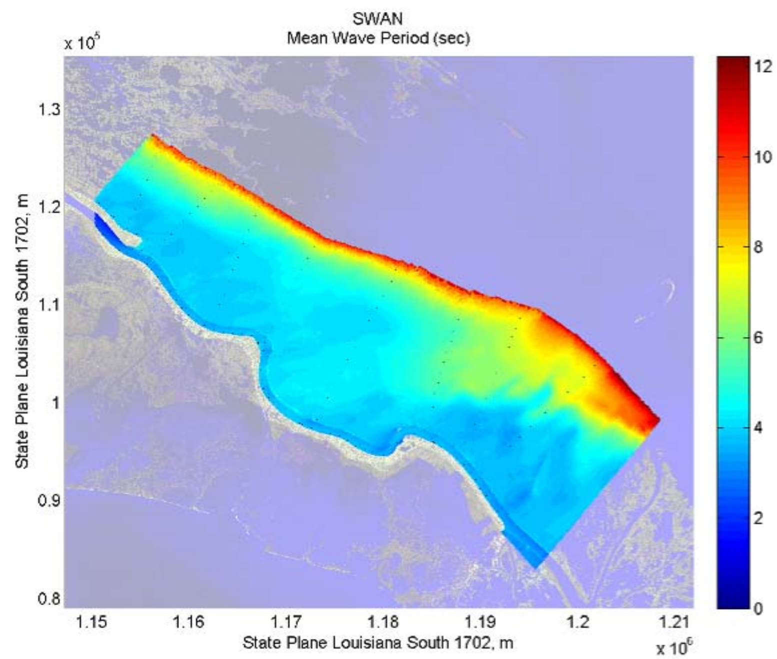
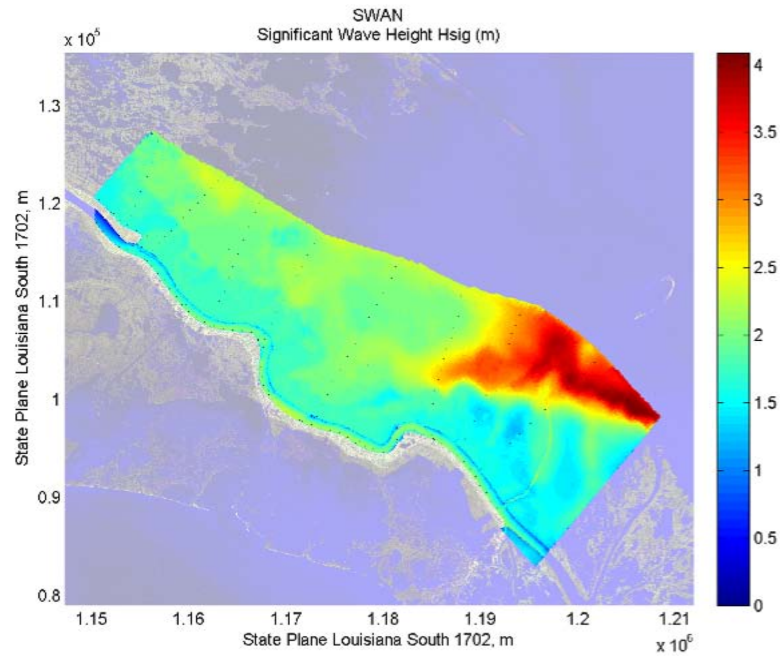


Figure 25 – Scenario 10 – W peed = 88 mph at 45 deg - Significant Wave Height (m) and Mean Wave Period (s)



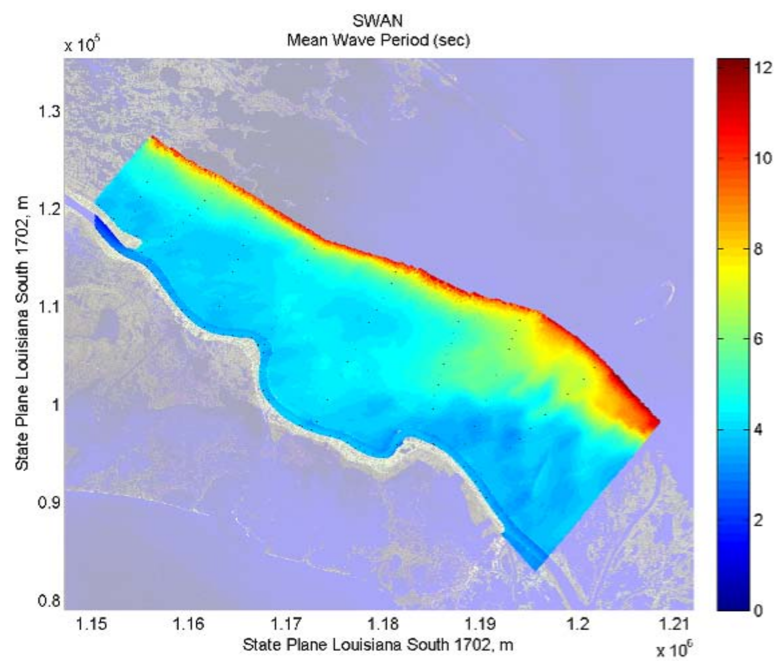
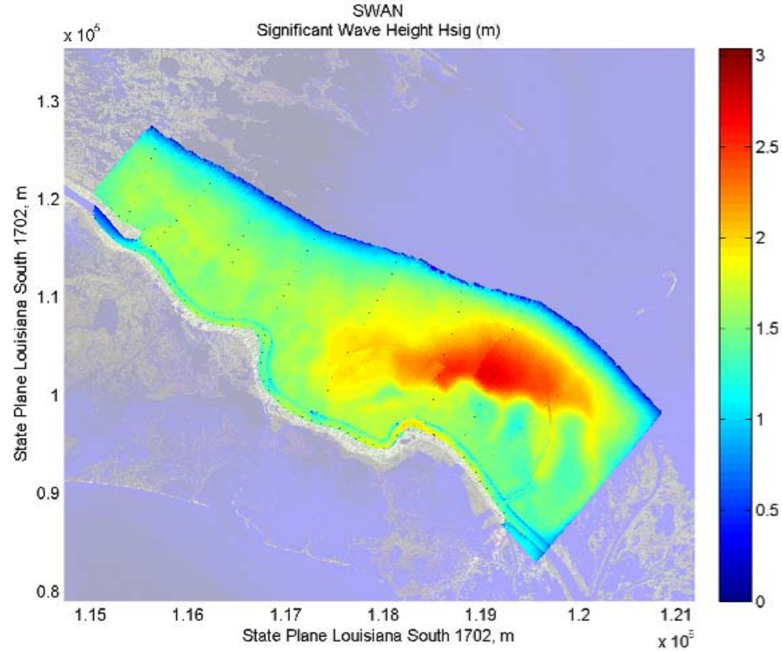


Figure 26– Scenario 11 – Wind Speed = 99 mph at 45 deg - Significant Wave Height (m) and Mean Wave Period (s)



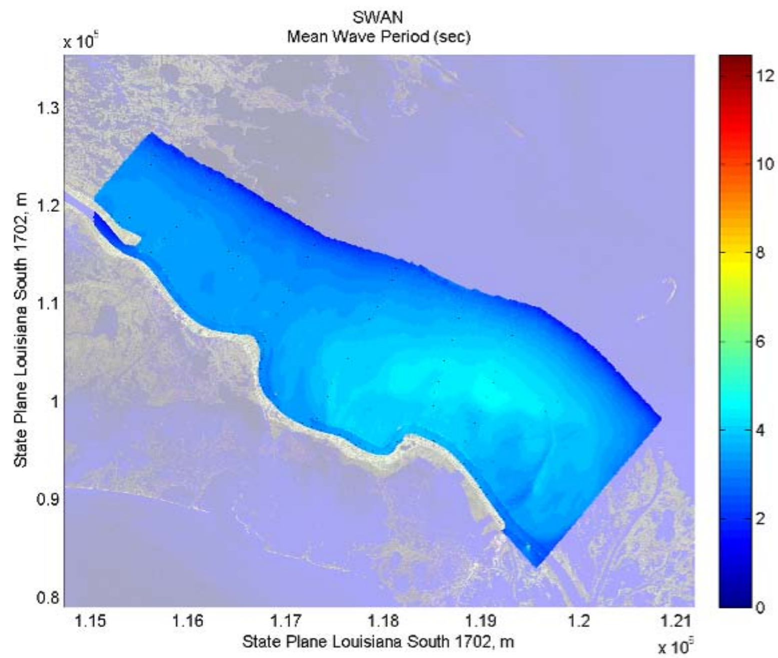


Figure 27 – Scenario 12 – Wind Speed = 77 mph at 45 deg with no offshore boundary conditions

Appendix K2 - SWAN Print File Example

```
            Execution started at 20091022.113511
            ---------------------------------------
                       SWAN
            SIMULATION OF WAVES IN NEAR SHORE AREAS
                  VERSION NUMBER 40.72ABCD
            ---------------------------------------

$************************************** HEADING *************************************************
PROJ 'UNST' 'MS'
$ Serie                              : Mississippi River SWAN Model
$ Scenario          : MS below mile 44
 $****************************************** MODEL INPUT *****************************************
$Set Maximum Error in pre-processing to 2     Nautical convention for wind and wave direction used
 SET MAXERR = 3  NAUTICAL
$Set mode to Stationary and two dimensional
 MODE STAT TWOD
 $Unstructured Grid
CGRID UNSTRUCTURED CIRCLE 36 0.025 0.800
Number of meshes in sigma-space: MSC-1 =  36
READGRID UNSTRUCURED ADCIRC
** Error           : number of cells around vertex is smaller than 4 or larger than 10
** Message         : The grid contains solely acute triangles
The unstructured grid contains solely triangles generated by SMS/ADCIRC
Number of vertices         = 100643
Number of cells            = 199596
  Number of internal cells = 197908
  Number of boundary cells =   1688
Number of faces            = 300238
  Number of internal faces = 298550
  Number of boundary faces =   1688
The minimum gridsize =    19.26639
The maximum gridsize =   359.94916

$ set Wind and WL
INP  wlevel  unstruct
READ wlevel FAC= 1.0  '100yrSurface.txt'
 $ SET LEVEL = 4.572
  WIND 34.42 45
 $          x0        y0       alp    mx   my     dx      dy
 $INP  friction   1308990.    499717.    141.   682   743   200.    200.  EXC = -999.
 $read friction  FAC= 1.0  '..\shared\fric\madsen_katrina_fric050.txt' IDLA=3  FREE

$****************************************** BOUNDARY CONDITIONS **********************************
 BOUN SHAPE JON MEAN DSPR POWER
$Side 1 Constant Wave Spectra defined by following parameters:  [hs] [per] [dir] [dd]
 BOUN SIDE 1 CON PAR 3.566 11.30 45 31.5
 BOUN SIDE 2 CON PAR 2.911 10.75 45 31.5
 BOUN SIDE 3 CON PAR 2.256 10.20 45 31.5
 BOUN SIDE 4 CON PAR 2.256 9.90 45 31.5
 BOUN SIDE 5 CON PAR 2.134 9.60 45 31.5
 $****************************************** PHYSICA *********************************************
 GEN3
 BREAKING
 $FRICTION
 $DIFFRACTION
 $TRIADS
  $************************************** NUMERICAL PARAMETERS ****************************************
 NUM ACCUR 0.02 0.02 0.02 99. STAT MXITST=100
 $************************************* OUTPUT POINTS ************************************************
 POINTS 'Output1' FILE '..\Output_Points.dat'
 $ ************************************ TABLE OUTPUT *********************************************
TABLE 'Output1' HEAD 'tab\Output1.tab' XP YP DEP SETUP HS TPS TMM10 TM01 TM02 TMBOT DIR WLEN STEEP WIND
 $ ************************************ BLOCK OUTPUT *********************************************
 BLOCK 'COMPGRID' NOHEAD 'mat\MS.mat' LAYOUT 3 XP YP  WIND WATLEV DEPTH DEP SETUP HS RTP TMM10 TM02 DIR
DSPR DIR STEEP
BLOCK 'COMPGRID' NOHEAD 'mat\MS2.mat' LAYOUT 3 XP YP  DISSip DISBot DISSUrf DISWcap HSWEll FrCoef
  $* ************************************** SP1 OUTPUT ABSOLUTE *************************************
 SPECOUT  'Output1'              SPEC1D ABSOLUTE 'sp1\Output1_abs.sp1'
 $ ************************************ SP1 OUTPUT RELATIVE *************************************
$ ************************************* SP2 OUTPUT ABSOLUTE *************************************
 SPECOUT  'Output1'              SPEC2D ABSOLUTE 'sp2\Output1_abs.sp2'
 $ ************************************ SP1 OUTPUT RELATIVE *************************************
```

```
COMPUTE
** Warning         : Bottom friction not on, UBOT not computed

-----------------------------------------------------------------
               COMPUTATIONAL PART OF SWAN
-----------------------------------------------------------------

Gridresolution       : MSC            37 MDC           36
                  : MTC            1
                  : NSTATC          0 ITERMX        100
Propagation flags    : ITFRE           1 IREFR          1
Source term flags    : IBOT            0 ISURF          1
                  : IWCAP           1 IWIND          3
                  : ITRIAD          0 IQUAD          2
Spectral bin        : df/f    0.1011E+00 DDIR    0.1000E+02
Physical constants   : GRAV    0.9810E+01 RHO     0.1025E+04
Wind input          : WSPEED   0.3442E+02 DIR    -0.1350E+03
Tail parameters      : E(f)    0.4000E+01 E(k)    0.2500E+01
                  : A(f)    0.5000E+01 A(k)    0.3000E+01
Accuracy parameters  : DREL    0.2000E-01 NPNTS   0.9900E+02
                  : DHOVAL  0.2000E-01 DTOVAL  0.2000E-01
                  : GRWMX   0.1000E+00
Drying/flooding      : LEVEL   0.0000E+00 DEPMIN  0.5000E-01
The nautical  convention for wind and wave directions is used
Scheme for geographic propagation is BSBT
Scheme spectral space: CSS     0.5000E+00 CDD     0.5000E+00
Current is off
Quadruplets          : IQUAD           2
                  : LAMBDA  0.2500E+00 CNL4    0.3000E+08
                  : CSH1    0.5500E+01 CSH2    0.8330E+00
                  : CSH3   -0.1250E+01
Maximum Ursell nr for Snl4 :    0.1000E+02
Triads is off
Bottom friction is off
W-cap Komen (`84)    : EMPCOF   0.2360E-04 APM     0.3020E-02
Battjes&Janssen (`78): ALPHA    0.1000E+01 GAMMA   0.7300E+00
Set-up is off
Diffraction is off
Janssen (`89,`90)    : ALPHA    0.1000E-01 KAPPA   0.4100E+00
Janssen (`89,`90)    : RHOA     0.1280E+01 RHOW    0.1025E+04

1st and 2nd gen. wind: CF10     0.1880E+03 CF20    0.5900E+00
                  : CF30    0.1200E+00 CF40    0.2500E+03
                  : CF50    0.2300E-02 CF60   -0.2230E+00
                  : CF70    0.0000E+00 CF80   -0.5600E+00
                  : RHOAW   0.1249E-02 EDMLPM  0.3600E-02
                  : CDRAG   0.1230E-02 UMIN    0.1000E+01
                  : LIM_PM   0.1300E+00

Number of active points = 100635 (fillings-degree:  99.99 %)


Settings of 2nd generation mode as first guess are used:
ITER     1    GRWMX   0.1000E+23    ALFA     0.0000E+00
IWIND    2    IWCAP    0           IQUAD     0
ISURF    1    IBOT     0           ITRIAD    0

iteration    1; sweep   1
iteration    1; sweep   2
iteration    1; sweep   3
iteration    1; sweep   4
iteration    1; sweep   5
iteration    1; sweep   6
iteration    1; sweep   7
iteration    1; sweep   8
iteration    1; sweep   9
iteration    1; sweep  10
iteration    1; sweep  11
iteration    1; sweep  12
iteration    1; sweep  13
iteration    1; sweep  14
     not possible to compute accuracy, first iteration
User-defined settings of 3rd generation mode is re-used:
ITER     2    GRWMX   0.1000E+00    ALFA     0.0000E+00
```

IWIND 3 IWCAP 1 IQUAD 2
ISURF 1 IBOT 0 ITRIAD 0

iteration 2; sweep 1
iteration 2; sweep 2
iteration 2; sweep 3
iteration 2; sweep 4
iteration 2; sweep 5
iteration 2; sweep 6
iteration 2; sweep 7
iteration 2; sweep 8
iteration 2; sweep 9
iteration 2; sweep 10
iteration 2; sweep 11
iteration 2; sweep 12
iteration 2; sweep 13
iteration 2; sweep 14
accuracy OK in 55.60 % of wet grid points (99.00 % required)

iteration 3; sweep 1
iteration 3; sweep 2
iteration 3; sweep 3
iteration 3; sweep 4
iteration 3; sweep 5
iteration 3; sweep 6
iteration 3; sweep 7
iteration 3; sweep 8
iteration 3; sweep 9
iteration 3; sweep 10
iteration 3; sweep 11
iteration 3; sweep 12
iteration 3; sweep 13
iteration 3; sweep 14
accuracy OK in 48.23 % of wet grid points (99.00 % required)

iteration 4; sweep 1
iteration 4; sweep 2
iteration 4; sweep 3
iteration 4; sweep 4
iteration 4; sweep 5
iteration 4; sweep 6
iteration 4; sweep 7
iteration 4; sweep 8
iteration 4; sweep 9
iteration 4; sweep 10
iteration 4; sweep 11
iteration 4; sweep 12
iteration 4; sweep 13
iteration 4; sweep 14
accuracy OK in 47.20 % of wet grid points (99.00 % required)

iteration 5; sweep 1
iteration 5; sweep 2
iteration 5; sweep 3
iteration 5; sweep 4
iteration 5; sweep 5
iteration 5; sweep 6
iteration 5; sweep 7
iteration 5; sweep 8
iteration 5; sweep 9
iteration 5; sweep 10
iteration 5; sweep 11
iteration 5; sweep 12
iteration 5; sweep 13
iteration 5; sweep 14
accuracy OK in 51.28 % of wet grid points (99.00 % required)

iteration 6; sweep 1
iteration 6; sweep 2
iteration 6; sweep 3
iteration 6; sweep 4
iteration 6; sweep 5
iteration 6; sweep 6
iteration 6; sweep 7
iteration 6; sweep 8

iteration 6; sweep 9
iteration 6; sweep 10
iteration 6; sweep 11
iteration 6; sweep 12
iteration 6; sweep 13
iteration 6; sweep 14
accuracy OK in 56.04 % of wet grid points (99.00 % required)

iteration 7; sweep 1
iteration 7; sweep 2
iteration 7; sweep 3
iteration 7; sweep 4
iteration 7; sweep 5
iteration 7; sweep 6
iteration 7; sweep 7
iteration 7; sweep 8
iteration 7; sweep 9
iteration 7; sweep 10
iteration 7; sweep 11
iteration 7; sweep 12
iteration 7; sweep 13
iteration 7; sweep 14
accuracy OK in 61.25 % of wet grid points (99.00 % required)

iteration 8; sweep 1
iteration 8; sweep 2
iteration 8; sweep 3
iteration 8; sweep 4
iteration 8; sweep 5
iteration 8; sweep 6
iteration 8; sweep 7
iteration 8; sweep 8
iteration 8; sweep 9
iteration 8; sweep 10
iteration 8; sweep 11
iteration 8; sweep 12
iteration 8; sweep 13
iteration 8; sweep 14
accuracy OK in 67.50 % of wet grid points (99.00 % required)

iteration 9; sweep 1
iteration 9; sweep 2
iteration 9; sweep 3
iteration 9; sweep 4
iteration 9; sweep 5
iteration 9; sweep 6
iteration 9; sweep 7
iteration 9; sweep 8
iteration 9; sweep 9
iteration 9; sweep 10
iteration 9; sweep 11
iteration 9; sweep 12
iteration 9; sweep 13
iteration 9; sweep 14
accuracy OK in 77.15 % of wet grid points (99.00 % required)

iteration 10; sweep 1
iteration 10; sweep 2
iteration 10; sweep 3
iteration 10; sweep 4
iteration 10; sweep 5
iteration 10; sweep 6
iteration 10; sweep 7
iteration 10; sweep 8
iteration 10; sweep 9
iteration 10; sweep 10
iteration 10; sweep 11
iteration 10; sweep 12
iteration 10; sweep 13
iteration 10; sweep 14
accuracy OK in 84.88 % of wet grid points (99.00 % required)

iteration 11; sweep 1
iteration 11; sweep 2
iteration 11; sweep 3

iteration 11; sweep 4
iteration 11; sweep 5
iteration 11; sweep 6
iteration 11; sweep 7
iteration 11; sweep 8
iteration 11; sweep 9
iteration 11; sweep 10
iteration 11; sweep 11
iteration 11; sweep 12
iteration 11; sweep 13
iteration 11; sweep 14
accuracy OK in 90.50 % of wet grid points (99.00 % required)

iteration 12; sweep 1
iteration 12; sweep 2
iteration 12; sweep 3
iteration 12; sweep 4
iteration 12; sweep 5
iteration 12; sweep 6
iteration 12; sweep 7
iteration 12; sweep 8
iteration 12; sweep 9
iteration 12; sweep 10
iteration 12; sweep 11
iteration 12; sweep 12
accuracy OK in 93.79 % of wet grid points (99.00 % required)

iteration 13; sweep 1
iteration 13; sweep 2
iteration 13; sweep 3
iteration 13; sweep 4
iteration 13; sweep 5
iteration 13; sweep 6
iteration 13; sweep 7
iteration 13; sweep 8
iteration 13; sweep 9
iteration 13; sweep 10
iteration 13; sweep 11
iteration 13; sweep 12
iteration 13; sweep 13
iteration 13; sweep 14
accuracy OK in 95.52 % of wet grid points (99.00 % required)

iteration 14; sweep 1
iteration 14; sweep 2
iteration 14; sweep 3
iteration 14; sweep 4
iteration 14; sweep 5
iteration 14; sweep 6
iteration 14; sweep 7
iteration 14; sweep 8
iteration 14; sweep 9
iteration 14; sweep 10
iteration 14; sweep 11
iteration 14; sweep 12
iteration 14; sweep 13
iteration 14; sweep 14
accuracy OK in 96.52 % of wet grid points (99.00 % required)

iteration 15; sweep 1
iteration 15; sweep 2
iteration 15; sweep 3
iteration 15; sweep 4
iteration 15; sweep 5
iteration 15; sweep 6
iteration 15; sweep 7
iteration 15; sweep 8
iteration 15; sweep 9
iteration 15; sweep 10
iteration 15; sweep 11
iteration 15; sweep 12
iteration 15; sweep 13
iteration 15; sweep 14
accuracy OK in 97.08 % of wet grid points (99.00 % required)

iteration 16; sweep 1
iteration 16; sweep 2
iteration 16; sweep 3
iteration 16; sweep 4
iteration 16; sweep 5
iteration 16; sweep 6
iteration 16; sweep 7
iteration 16; sweep 8
iteration 16; sweep 9
iteration 16; sweep 10
iteration 16; sweep 11
iteration 16; sweep 12
iteration 16; sweep 13
iteration 16; sweep 14
accuracy OK in 97.45 % of wet grid points (99.00 % required)

iteration 17; sweep 1
iteration 17; sweep 2
iteration 17; sweep 3
iteration 17; sweep 4
iteration 17; sweep 5
iteration 17; sweep 6
iteration 17; sweep 7
iteration 17; sweep 8
iteration 17; sweep 9
iteration 17; sweep 10
iteration 17; sweep 11
iteration 17; sweep 12
iteration 17; sweep 13
iteration 17; sweep 14
accuracy OK in 97.75 % of wet grid points (99.00 % required)

iteration 18; sweep 1
iteration 18; sweep 2
iteration 18; sweep 3
iteration 18; sweep 4
iteration 18; sweep 5
iteration 18; sweep 6
iteration 18; sweep 7
iteration 18; sweep 8
iteration 18; sweep 9
iteration 18; sweep 10
iteration 18; sweep 11
iteration 18; sweep 12
iteration 18; sweep 13
iteration 18; sweep 14
accuracy OK in 98.11 % of wet grid points (99.00 % required)

iteration 19; sweep 1
iteration 19; sweep 2
iteration 19; sweep 3
iteration 19; sweep 4
iteration 19; sweep 5
iteration 19; sweep 6
iteration 19; sweep 7
iteration 19; sweep 8
iteration 19; sweep 9
iteration 19; sweep 10
iteration 19; sweep 11
iteration 19; sweep 12
iteration 19; sweep 13
iteration 19; sweep 14
accuracy OK in 98.38 % of wet grid points (99.00 % required)

iteration 20; sweep 1
iteration 20; sweep 2
iteration 20; sweep 3
iteration 20; sweep 4
iteration 20; sweep 5
iteration 20; sweep 6
iteration 20; sweep 7
iteration 20; sweep 8
iteration 20; sweep 9
iteration 20; sweep 10
iteration 20; sweep 11

iteration 20; sweep 12
iteration 20; sweep 13
iteration 20; sweep 14
accuracy OK in 98.67 % of wet grid points (99.00 % required)

iteration 21; sweep 1
iteration 21; sweep 2
iteration 21; sweep 3
iteration 21; sweep 4
iteration 21; sweep 5
iteration 21; sweep 6
iteration 21; sweep 7
iteration 21; sweep 8
iteration 21; sweep 9
iteration 21; sweep 10
iteration 21; sweep 11
iteration 21; sweep 12
iteration 21; sweep 13
iteration 21; sweep 14
accuracy OK in 99.01 % of wet grid points (99.00 % required)

STOP

Appendix K3 - SWAN Variable Description

Mean absolute wave period presented in this report as TM_10 is defined by the following equation for TMM10:

TMM10 Mean absolute wave period (in s) of $E(\omega,\theta)$, defined as

$$T_{m-10} = 2\pi \frac{\int\int \omega^{-1} E(\omega,\theta) d\omega d\theta}{\int\int E(\omega,\theta) d\omega d\theta} = 2\pi \frac{\int\int \omega^{-1} E(\sigma,\theta) d\sigma d\theta}{\int\int E(\sigma,\theta) d\sigma d\theta}$$

TM01 Mean absolute wave period (in s) of $E(\omega,\theta)$, defined as

$$T_{m01} = 2\pi \left(\frac{\int\int \omega E(\omega,\theta) d\omega d\theta}{\int\int E(\omega,\theta) d\omega d\theta} \right)^{-1} = 2\pi \left(\frac{\int\int \omega E(\sigma,\theta) d\sigma d\theta}{\int\int E(\sigma,\theta) d\sigma d\theta} \right)^{-1}$$

Significant wave heights presented in this report are defined using the following equation:

HSIGN Significant wave height, denoted as H_s in meters, and defined as

$$H_s = 4\sqrt{\int\int E(\omega,\theta) d\omega d\theta}$$

where $E(\omega,\theta)$ is the variance density spectrum and ω is the absolute radian frequency determined by the Doppler shifted dispersion relation. However, for ease of computation, H_s can be determined as follows:

$$H_s = 4\sqrt{\int\int E(\sigma,\theta) d\sigma d\theta}$$

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Appendix L

Future Conditions

(Authors: J. Smith, ERDC and J. Atkins, Ayres Associates)

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APPENDIX L – FUTURE CONDITIONS

(Author: Jane Smith, ERDC and John Atkinson, Ayres Associates)

This appendix describes the effect of sea level rise and wave characteristics using ADCIRC and STWAVE (version 06/14/2007). The text below was provided by Jane Smith from ERDC and John Atkinson from Ayres Associates.

Sea level rise and subsidence are significant issues in the design of flood protection for southeast Louisiana. Flood walls, in particular, can not be easily raised, so future sea level rise must be considered in the initial design. The purpose of this analysis is to estimate the impact of sea level rise on 100-yr surge and waves for the design of the flood defenses.

The sea level rise analysis consisted of 27 storm simulations. Nine storms were selected from the 2010 simulations and each was run with 1 ft, 2 ft, and 3 ft increase in water level. No other changes to input were made (same offshore waves, same land cover specification, same model parameters, etc.). The nine storms selected were storms 005, 009, 015, 017, 024, 036, 053, 067, and 126. These storms were chosen to target 100-year water levels in various areas. Table 1 summarizes the approximate water level recurrence interval averaged over the target reaches for each storm.

Table 1. Approximate Water Level Return Periods for Selected Storms

Storm	Target Area/Approximate Water Level Recurrence (yrs)							
	South Shore Pontchartrain	Orleans E. and No. St. Bernard	St. Bernard So. and Caenarvon	Plaq. East	Plaq. West	West Bank	Golden Meadow	Morganza to the Gulf
005	25	25	45	25	25	65	80	200
009	70	65	200	60	60	250	550	1600
015	75	77	250	75	125	125	100	30
017	75	85	300	100	250	350	760	35
024	115	230	90	220	220	20	30	20
036	80	225	25	800	160	15	20	20
053	75	175	400	200	120	130	200	50
067	15	15	20	20	30	70	50	110
126	60	85	230	90	60	80	550	130

To summarize the results, Eleven reaches are defined: South Shore of Lake Pontchartrain (SSP), East Orleans (EO), St. Bernard North (SBN), St. Bernard South (SBS), Caenarvon (C), Plaquemines East (PE), Plaquemines West (PW), South West Bank (SWB), North West Bank (NWB), Golden Meadow (GM), and Morganza to the Gulf (MtG). These areas are illustrated in Figure 1.

The selection of only nine storms that give approximate 100-yr water levels provides estimates of the impact of sea level rise, but is not a rigorous analysis. For example, land cover classifications were not changed in the analysis. Vegetation types would change as water level increases, but if the increase is slow enough and sediment is available, the marsh elevation may also adjust to the change in water level. Manning-n values were not adjusted in this analysis because of the uncertainty in the values for higher sea level and so the results at each water level could be directly compared. Sea level was increased over the entire domain, which means that local impacts of subsidence are probably over estimated. The impacts of increasing sea level are two fold, the surge wave (which propagates at a speed, $c = \sqrt{gd}$, were g is acceleration of gravity and d is water depth) propagates faster, and the depth-limited wave height increases (also increasing wave setup). In general, we expect sea level rise to increase water levels more than linearly (water level increase > sea level rise), but the complex, shallow geometry and bathymetry of Southeast Louisiana alters this trend depending on the relative speed of the storm and the surge propagation (and the relative phasing of the two).

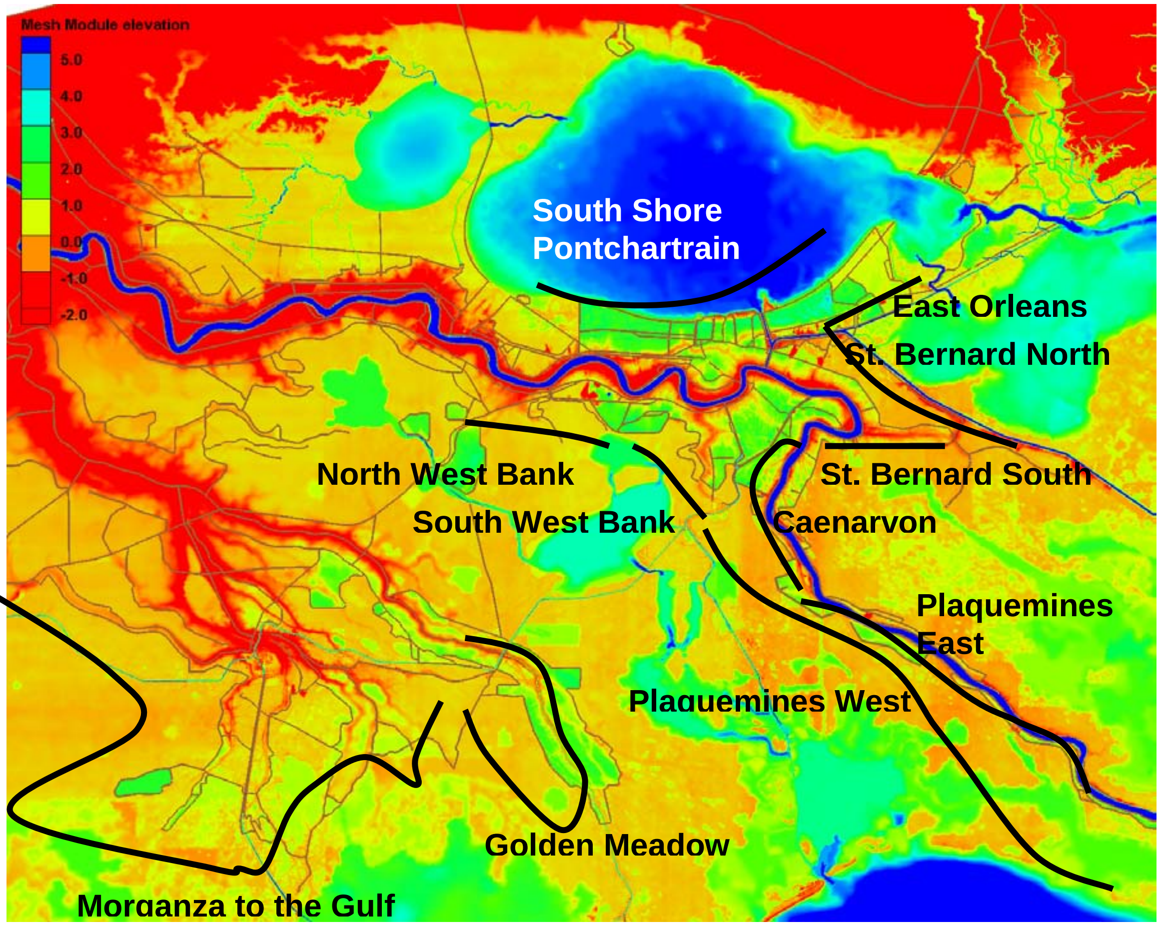

Figure 1. Reach Definitions overlaid on the ADCIRC gird (depths in meters).

Surge Results

The water level results are provided in tabular and graphic form. Tables 2-4 provide the range of maximum water level increase (in feet) for 1, 2, and 3 ft of sea level rise, respectively. The increases are calculated as the difference between the maximum water level at each grid point for the sea level rise run and the maximum water level for the base JPM run, calculated for each of the nine storms. The highlighted values are the storms at approximately the 100-yr water level for that reach (50-200 yr).

Figure 2 plots the relative water level increase (water level increase normalized by the sea level rise). The first trend to note is that the relative increase for a given storm and location, decreases as sea level rise increases. For example, storm 036 at Caenarvon generates a multiplier of 3.5 for 1 ft sea level rise, 3 for 2 ft sea level rise, and 2.5 for 3 ft sea level rise. The second trend to note is that the West Bank, St. Bernard South, and Caenarvon areas are highly variable in response (multipliers of 0.6 to 4.5). This is due to complexity of these areas and the interplay of "pockets" that catch the surge and the interaction of the storm track and river levees.

<u>South Shore of Lake Pontchartrain</u>. The SSP reach has the most consistent response to sea level rise. The multiplier is 1.0 to 1.5 (1 would be a linear response, 1 ft sea level rise = 1 ft increase is water level) with an average value of 1.3 for the target storms. The increased depth

decreases the friction, allowing more water to pile up on the shore. Waves will also increase, but that probably has minimal effect on the setup in Pontchartrain.

Back Levees of East Orleans and St. Bernard North. The response in EO and SBN has slightly more variation than SSP, with a multiplier of 1.1 to 1.6. This area forms a small pocket in the funnel area, but the reach is not as complex or shallow as areas to the south and west. The multipliers for the storms near the 100-yr water level are 1.1 to 1.6 in EO and 1.2 to 1.6 in SBN, with average values of 1.2 and 1.3, respectively.

St. Bernard South and Caenarvon. This reach is complex and shallow, and the results are highly variable with multipliers of 0.7 to 4.5. The large responses correspond to the storms with some of the smallest maximum surges (storms 24 and 36). These storms have tracks that cross through Breton Sound, east of this area. As the storms pass, the larger water depth allows the surge to move in faster, as well as decreasing the frictional resistance. The "catchers mitt" of Caenarvon amplifies the surge for these storms. Storms 009, 015, 017, 053, and 126 produce the largest surge in these areas (20-25 ft) and the sea level rise multipler for these storms is 0.6 to 1.3 for St. Bernard South and 0.6 to 2.0 for Caenarvon. Storms 009 and 024 produce the 100-yr water levels and these storms indicate multipliers of 0.7 to 2.3 for SBS and 0.7 to 4.5 for C with average values of 1.4 and 2.1, respectively.

Plaquemines East and West. These reaches are large with a lot of spatial variability, but the multipliers are less variable than the adjoining reaches. The multipliers for the target storms are 1.3 to 2.0 for Plaquemines East. For the Plaquemines West reach, the range of multipliers for the target storms is 1.4 to 3, with average values of 1.5 and 1.9, respectively.

West Bank. This reach is also complex and shallow. The multipliers range from 1.0 to 3.6. Storms 005, 015, 053, 067, and 126 are near the 100-yr level for the West Bank. The multipliers for these storms are large 1.3 to 3.6 for SWB and 1.0 to 2.9 for NWB. The largest numbers tend to be hot spots (small areas) and not large areas of high multipliers. The average multipliers for the target storms are 2.5 for SWB and 2.1 for NWB.

Golden Meadow and Morganza to the Gulf. Multipliers in this reach are similar to the West Bank, but not as variable. Multipliers range from 1.0 to 2.5. The surges tend to be most amplified on the northeast corner of Golden Meadow and in the pocket regions. The multipliers for the storms near the 100-yr water level are 1.4 to 2.3 for Golden Meadow and 1.5 to 2.0 for Morganza to the Gulf, with average values of 1.8 and 1.7, respectively.

Table 2. Increases in Peak Water Level for 1 ft Sea Level Rise (increase in feet)

	Storm 005	Storm 009	Storm 015	Storm 017	Storm 024	Storm 036	Storm 053	Storm 067	Storm 126
SSP	1.3	1.3	1.3	1.3	1.3	1-1.3	0.9-1	1.2	0.9-1
EO	1-1.1	1.3-1.6	1-1.3	1-1.3	1	1.3-1.6	1.1-1.2	1	1-1.2
SBN	1-1.3	1.6	1-1.3	1-1.6	1-1.3	1-1.3	1.1-1.3	1.1	1-1.3
SBS	1-1.2	1	1	1-1.3	1.6-2.3	2-3	0.9-1	1.4-1.9	0.9-1
C	1-2.2	1	1-1.6	1.3-2	4-4.5	3.3-3.6	0.8-1.3	2-2.4	1-1.5
PE	0.5-1.3	0.9-1.8	0.8-2	0.8-1.7	0.8-1.5	0.6-1.8	0.6-1.3	0.7-1.1	0.9-1.5
PW	1-1.5	1-1.9	1-1.4	0.7-2	0.7-2	0.7-3	1-2	0.9-1.6	0.9-1.4
SWB	1.3-2.7	2-3	2	2-2.3	1-1.3	1	2-3.6	1.6-2.1	1.4-3.2
NWB	1.5-1.9	1.5-2	1.6	1.6-2	1	1	1.9-3	1.1-1.7	1.4-2.8
GM	1-1.8	1-1.8	1-2.3	0.5-2.6	0.8-1.8	0.7-1.9	0.9-1.7	1.3-2	0.5-1.6
MtG	1-1.8	1-1.5	1-1.8	1-1.6	0.7-1.6	1	1-1.7	1-2	0.8-1.6

Table 3. Increases in Peak Water Level for 2 ft Sea Level Rise (increase in feet)

	Storm 005	Storm 009	Storm 015	Storm 017	Storm 024	Storm 036	Storm 053	Storm 067	Storm 126
SSP	2.5	2.6-2.8	2.6	2.6	2.6-3	2.3-2.6	1.9-2.3	2.4	1.9-2.4

EO	2-2.2	2-2.3	2.3-2.6	2-2.6	2.3-2.6	2.3-3	2.3	2	2.1-2.3
SBN	2-2.6	2.3-2.6	2.3-2.6	2-3	2.3	2.3	2.1-2.5	2.2	2.1-2.5
SBS	2.2-2.6	1.6	2	1-2	3-4	4-5	1.7	2.5-3.5	1.7-1.8
C	2.6-3.6	1.6	1-2.3	1-2.3	4-6.5	5-6	1.5-2.6	4-4.5	1.6-2.7
PE	1.3-3	1.6-3.3	1.6-3.3	1.5-3.3	1.5-2.9	1.3-3	1.2-2.5	1.3-2.3	1.7-2.9
PW	2-3	2-3.5	2-3.3	1.8-3.3	1.3-5.8	0.5-5.6	2-4	1.7-3.1	1.9-2.9
SWB	3-4.6	4-5	3.5-4.3	5	3-4	2	3.8-6.2	3.2-5	2.6-5.9
NWB	3-3.6	3-3.6	3-5.7	4-4.3	2	2	3.3-4.9	3-4.6	3.1-4.6
GM	2-3.3	1.5-3.5	2-4.3	1-4.9	1.5-3.3	1.5-3.3	1.6-3.2	2.5-3.3	1-3.1
MtG	2-3.4	2-2.9	2-3.2	2-3	2-3.2	2-2.8	2-3.2	2-3.6	1.6-3.1

Table 4. Increases in Peak Water Level for 3 ft Sea Level Rise (increase in feet)

	Storm 005	Storm 009	Storm 015	Storm 017	Storm 024	Storm 036	Storm 053	Storm 067	Storm 126
SSP	3.8	4-4.3	4	4.3	3.3-4.3	3.3-4	3-3.6	3.7	3-3.7
EO	3-3.2	3.3	3.3-3.6	3.3	3.3-3.6	3.5-4.5	3.3	3	3.1-3.3
SBN	3-3.7	3.3-3.6	3.3-4	3.6-4.6	3.3-3.6	3.3-3.6	3.3-3.5	3.3	3.1-3.6
SBS	3-3.5	2	2.3	2.6-3	4-5	4.6-6.2	2.2-2.4	3.6-4.6	2.3-2.4
C	3.5-4	2	1.6-2.6	1.6-3.3	6.6-7.2	6.5-7.5	2-3.3	5-5.9	2.2-3.6
PE	2-4	2.6-4	2.4-4.3	2.3-4.4	2.2-3.8	2-4	1.8-3.5	2-3.3	2.3-3.9
PW	3-4.3	3-5.2	2.9-5.2	2.7-5	1.8-7.8	2-7.2	3-6	2.6-5.2	3-4.6
SWB	4-6.5	5-5.3	7-7.5	6.6-7.2	5-6.6	3	5.6-8.5	5-6.9	4.8-7.8
NWB	4-5	5.3	5.6-6.2	6-6.2	3.3-4	3	4.3-6.2	5-5.9	3.9-5.9
GM	3-5	2-4.8	2-5.6	1.5-6.5	2.4-4.6	2.4-4.7	2.1-4.6	3.5-4.3	1.3-4.3
MtG	3-5	2.7-4.2	3-4.4	3-4.3	2.5-3.8	3-3.8	3-4.7	3-4.6	2.5-4.3

Recommended Multipliers. The recommended multipliers are provided in Table 5. These multipliers are the averages of the upper ranges of the multipliers for the target storms for each reach, including 1, 2, and 3 ft sea level rise simulations. The increase in surge is estimated as the sea level rise times the multiplier.

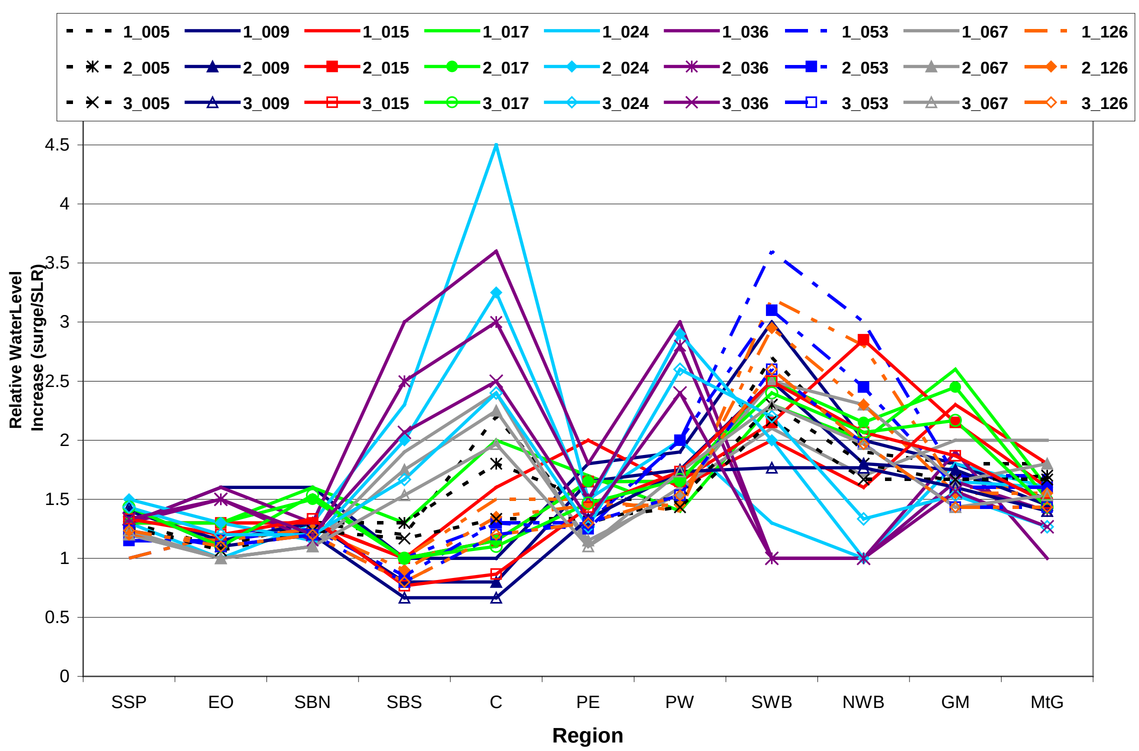

Figure 2. Relative Water Level Increases by Reach (legend provides the sea level rise (1, 2, and 3 ft) and storm number).

Table 5. Recommended Surge Multipliers for Sea Level Rise		
	Range	Surge Multiplier
Lake Pontchartrain	1.0-1.5	1.3
East Orleans	1.1-1.6	1.2
North St. Bernard	1.2-1.6	1.3
South St. Bernard	0.7-2.3	1.4
Caenarvon	0.7-4.5	2.1
Plaquemines East	1.3-2.0	1.5
Plaquemines West	1.4-3.0	1.9
South West Bank	1.3-3.6	2.5
North West Bank	1.0-2.9	2.1
Golden Meadow	1.4-2.3	1.8
Morganza to the Gulf	1.4-2.0	1.7

Wave Results

The wave results are also provided in tabular and graphical form. Tables 6-8 provide the range of maximum wave height increase (in feet) for 1, 2, and 3 ft of sea level rise, respectively. Figure 3 shows the increases graphically. The increases are calculated as the difference between the maximum wave height at each grid point for the sea level rise run and the maximum wave height for the base JPM run, calculated for each of the seven storms. The highlighted values are the storm at approximately the 100-yr water level for that reach.   The increases in wave height are generally less than 1 ft for East Orleans, St. Bernard North, and the West Bank. Pontchartrain, St. Bernard South, Caenarvon, Plaquemines, Golden Meadow, and Morganza to the Gulf had wave height increases up to 2-3 ft. The rate of increase in wave height is less for the larger values of sea level rise.

Figure 4 shows the wave height increase relative to surge increase (wave height increase normalized by the water level increase for the same sea level rise). The range of relative values is approximately 0.1 to 0.8. The ratios tend to decrease with increased sea level rise. The average relative values for the target storms in each reach are: Pontchartrain 0.41, East Orleans 0.15, St. Bernard North 0.16, St. Bernard South 0.45, Caenarvon 0.50, Plaquemines East 0.65, Plaquemines West 0.40, South West Bank 0.11, and North West Bank 0.15, Golden Meadow 0.24, and Morganza to the Gulf 0.43. The larger values are typically in the more exposed reaches (areas with less fronting marsh and deeper depths).

<u>South Shore of Lake Pontchartrain</u>.   The SSP reach has fairly consistent increase in wave height for sea level rise: 0.6 ft for 1 ft sea level rise, 1.0 ft for 2 ft sea level rise, and 1.5 ft for 3 ft sea level rise. The ratio of wave height increase to water level increase for the target storms varies from 0.23 to 0.60, with an average value of 0.43. The values are relatively high because an increase in surge results in a direct increase in depth-limited wave height in most areas.

<u>Back Levees of East Orleans and St. Bernard North</u>. The EO and SBN behave relatively consistently with increases in wave height of 0.1 to 1.2 ft for EO and 0.1 to 1.0 ft for SBN. The ratios of wave height increase to water level increase are all less than 0.4, with average values for the target storms of 0.13 (range of 0.06 to 0.31) for EO and 0.17 (range of 0.04 to 0.38) for SBN.

<u>St. Bernard South and Caenarvon</u>. This reach is complex and shallow, and the results are highly variable with wave height increases of 0.1 to 2.1 ft for SBS and 0.5 to 3.0 ft for C. The large responses correspond to the storms with the smallest maximum surges (storms 24 and 36). These storms have tracks that cross through Breton Sound, east of this area. As the storms pass, the larger water depth allows large waves to propagate into the area, as well as decreases the frictional resistance. The average ratio of wave height increase to water level

increase is relatively large in this area, 0.45 (range of 0.4 to 0.5) for SBS and 0.50 (range of 0.42 to 0.63) for C.

Plaquemines East and West. The wave height increases in these areas are similar to St. Bernard South and Caenarvon. The wave height increases are 0.4 to 2.8 ft for PE and 0.4 to 2.9 ft for PW. The maximum increases in wave height in the Plaquemines East reach were typically at the north end of this reach, between Phoenix and Davant. The average ratio of wave height increase to water level increase is 0.58 (range 0.38 to 0.78) for the target storms for PE. For the Plaquemines West reach, the maximum increases in wave height were typically between Empire and Buras or near Myrtle Grove. The average ratio of wave height increase to water level increase is 0.41 (range 0.23 to 0.69) for the target storms for PE.

West Bank. This reach is also complex and shallow. The wave height increases are 0.1 to 1.0 ft. The ratio of wave height increase to water level increase is 0.03 to 0.3 for the target storms with average values of 0.11 for SWB and 0.15 for NWB.

Golden Meadow and Morganza to the Gulf. These reaches include complex levee geometries (pockets) and bathymetry, but are more exposed than the west bank. The wave height increases are up to 2.0 ft along Golden Meadow and up to 3.0 ft along Morganza to the Gulf. The average ratio of wave height increase over surge increase for the target storms is 0.27 (range 0.14 to 0.42) for Golden Meadow and 0.37 (range 0.23 to 0.5) for Morganza to the Gulf.

Table 6. Wave Height Results for 1 ft Sea Level Rise (increase in feet)

	Storm 005	Storm 009	Storm 015	Storm 017	Storm 024	Storm 036	Storm 053	Storm 067	Storm 126
SSP	0-0.2	0.1-0.3	0.3-0.7	0.4-0.7	0.2-0.7	0.3-0.7	0.2-0.6	0-0.2	0.1-0.7
EO	0-0.2	0.1	0.1-0.4	0.1-0.2	0.1-0.3	0.1-0.2	0-0.1	0.3-0.4	0-0.1
SBN	0-0.3	0-0.6	0.1	0.1-0.2	0.1	0.1-0.2	0-0.4	0-0.4	0-0.2
SBS	0-0.1	0.1-0.4	0.3-0.5	0.4	0.3-1.1	0.1-0.7	0.2-0.3	0.2-0.3	0.2-0.3
C	0.2-1	0.2-0.6	0.2-0.7	0.6-0.9	1.0-2.0	0.1-0.7	0-0.8	0.3-0.5	0-1.2
PE	0-0.9	0.2-1.4	0.1-1.3	0.2-1.5	0.4-0.5	0.3-0.9	0-1.0	0-0.4	0.2-0.6
PW	0-0.4	0-0.5	0.1-1.1	0.1-0.8	0-0.8	0.1-0.7	0.1-0.8	0-0.4	0-0.6
SWB	0-0.1	0-0.1	0-0.2	0-0.2	0-0.3	0-0.2	0-0.5	0-0.4	0-0.3
NWB	0-0.2	0-0.1	0-0.2	0-0.2	0-0.3	0-0.2	0-0.3	0-0.3	0-0.2
GM	0.2-0.7	0-0.8	0-0.4	0-0.8	0-0.1	0-0.1	0-0.6	0.3-0.5	0-0.5
MtG	0.2-0.7	0.3-1.0	0-0.6	0-0.4	0-0.1	0-0.1	0-0.4	0.4-1.0	0-0.5

Table 7. Wave Height Results for 2 ft Sea Level Rise (increase in feet)

	Storm 005	Storm 009	Storm 015	Storm 017	Storm 024	Storm 036	Storm 053	Storm 067	Storm 126
SSP	0-0.4	0.5-1.0	0.6-1.2	0.5-1.2	0.4-1.1	0.5-1.2	0.3-1.1	0-0.3	0.2-1.1
EO	0-0.5	0.1-0.3	0.2-0.5	0.2-0.3	0.4-0.5	0.2-0.4	0.1-0.3	0.6-0.8	0-0.2
SBN	0-0.6	0.0-0.9	0.1-0.2	0.1-0.2	0.2-0.3	0.2-0.4	0-0.6	0-0.5	0-0.2
SBS	0-0.1	0.1-0.8	0.5-0.6	0.6-0.7	1.0-1.6	0.8-1.6	0.3-1.3	0.3-0.5	0.4-1.2
C	0.2-1.4	0.3-1.4	0.6-1.2	0.6-1.6	1.0-2.8	0.8-1.6	0-2.0	0.3-0.7	0-2.0
PE	0.2-1.1	0.3-2.2	0.3-1.8	0.4-2.4	0.5-1.0	0.5-1.7	0-1.2	0.1-0.6	0-1.2
PW	0.1-0.7	0-1.2	0.2-1.8	0.3-1.7	0.3-2.0	0.4-1.6	0.4-1.6	0-0.8	0-1.2
SWB	0.1-0.3	0-0.5	0-0.5	0.1-0.5	0-0.7	0.1-0.6	0-0.8	0.1-0.4	0-1.0
NWB	0.2-0.5	0-0.5	0-0.5	0.1-0.5	0-0.7	0.1-0.6	0-0.5	0.2-0.6	0-0.3
GM	0.4-1.2	0.3-1.0	0-0.6	0-1.5	0-0.2	0-0.1	0-1.0	0.3-0.7	0-1.0
MtG	0.4-1.6	0.8-2.0	0-0.9	0-0.7	0-0.1	0-0.2	0-0.9	0.5-1.5	0-1.2

Table 8. Wave Height Results for 3 ft Sea Level Rise (increase in feet)									
	Storm 005	Storm 009	Storm 015	Storm 017	Storm 024	Storm 036	Storm 053	Storm 067	Storm 126
SSP	0-0.5	0.7-1.2	0.7-1.3	0.8-1.4	1.0-1.7	0.8-1.4	0.4-1.7	0-0.4	0.3-1.6
EO	0-0.6	0.3-0.4	0.2-0.5	0.2-0.4	0.4-1.0	0.5-0.7	0.3-0.7	1.0-1.2	0-0.2
SBN	0-0.8	0.0-1.0	0.1-0.2	0.1-0.2	0.4-1.0	0.5-0.7	0.1-0.8	0-0.6	0-0.3
SBS	0-0.1	0.4-1.0	0.7-0.8	0.8-1.0	1.0-2.1	1.0-2.0	0.6-1.7	0.4-0.6	0.5-1.4
C	0.2-1.5	0.4-2.0	0.6-1.2	1.0-1.6	1.0-3.0	1.0-2.0	0-2.9	0.6-0.9	0-2.5
PE	0.2-1.2	0.5-2.6	0.4-2.0	0.5-2.8	0.6-1.5	0.8-2.0	0-1.8	0-1.0	0-1.5
PW	0.1-1.0	0.1-2.4	0.3-2.5	0.5-2.9	0.5-2.6	0.5-2.0	0.5-2.5	0-1.1	0-2.0
SWB	0.2-0.4	0.1-0.8	0.1-0.7	0.1-0.8	0.1-0.8	0.2-0.7	0-1.5	0.2-0.8	0-1.2
NWB	0.2-0.7	0.1-0.8	0.1-0.7	0.1-0.8	0.1-0.8	0.2-0.7	0-1.0	0.4-1.0	0-0.5
GM	0.6-1.4	0.3-1.7	0-1.0	0-2.0	0-0.3	0-0.1	0-1.8	0.3-0.8	0.3-1.5
MtG	0.7-2.4	1.0-3.0	0-1.0	0-1.0	0-0.3	0-0.2	0-1.4	0.6-1.5	0-1.6

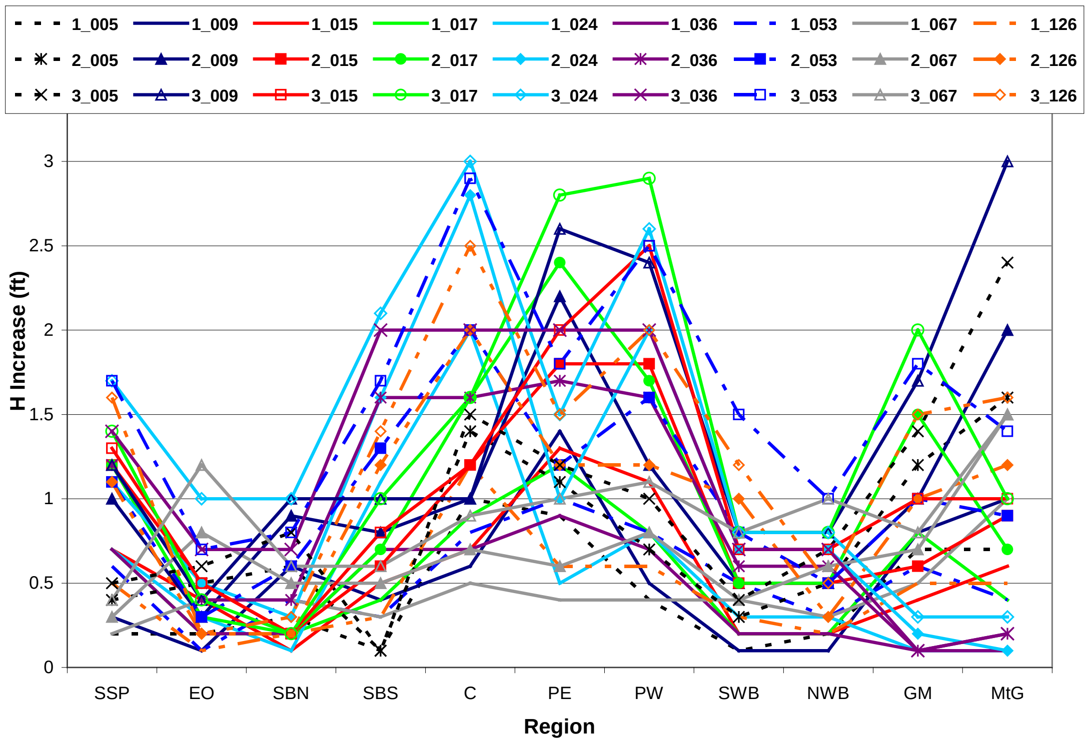

Figure 3. Wave height increase (feet).

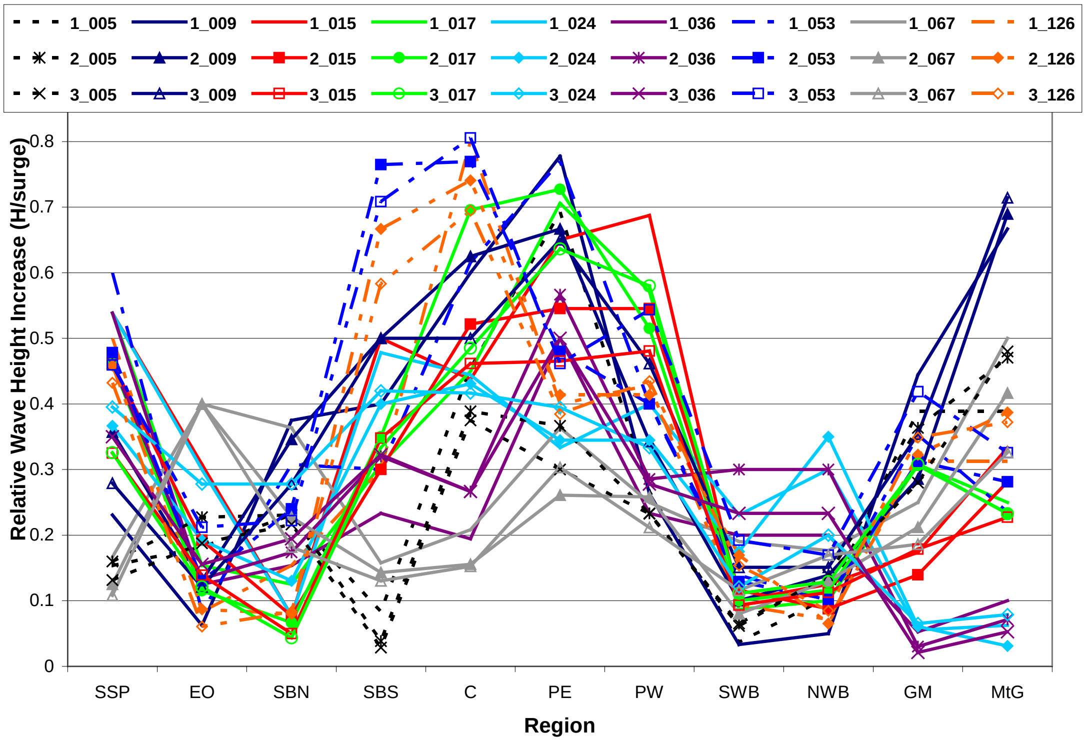

Figure 4. Normalized wave height increase (H increase/water level increase). Recommended Wave Height Increases.

The recommended wave height values are given in Table 9. The values are the averages of the upper ranges of the heights and ratios for the target storms for each reach, including 1, 2, and 3 ft sea level rise simulations. The increase in wave height for a region is estimated by first determining the water level change (sea level rise times the multiplier in Table 5) and then multiplying it times the right-hand column in Table 9 (e.g., for Lake Pontchartrain a 2 ft sea level rise would be multiplied by 1.3 to give a water level increase of 2.6 ft, and then the wave height increase would be 0.43 * 2.6 ft = 1.1 ft).

Table 9. Recommended Wave Height Response to Sea Level Rise

	1 ft SLR	2 ft SLR	3 ft SLR	ΔH/Δwater level
Lake Pontchartrain	**0.6 ft**	**1.0 ft**	**1.5 ft**	**0.43**
East Orleans	**0.2 ft**	**0.3 ft**	**0.4 ft**	**0.13**
North St. Bernard	**0.3 ft**	**0.4 ft**	**0.5 ft**	**0.17**
South St. Bernard	**0.8 ft**	**1.2 ft**	**1.6 ft**	**0.45**
Caenarvon	**1.3 ft**	**1.9 ft**	**2.0 ft**	**0.50**
Plaquemines East	**1.1 ft**	**1.8 ft**	**2.1 ft**	**0.58**
Plaquemines West	**0.7 ft**	**1.2 ft**	**2.5 ft**	**0.41**
South West Bank	**0.3 ft**	**0.6 ft**	**0.7 ft**	**0.12**
North West Band	**0.3 ft**	**0.5 ft**	**0.7 ft**	**0.13**
Golden Meadow	**0.6 ft**	**0.9 ft**	**1.3 ft**	**0.27**
Morganza to the Gulf	**0.7 ft**	**1.3 ft**	**1.7 ft**	**0.37**

Appendix M

CEMVN-ED-H Memorandum for Record, Subject: West Bank and Vicinity-Algiers Canal Design Levee Grade (13 Jan 2014)

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CEMVN-ED-H 13 Jan 2014
MEMORANDUM FOR RECORD

SUBJECT: West Bank and Vicinity – Algiers Canal Design Levee Grade

1. **References.**
   a. WBV-90 GIWW West Closure Complex, Detention Stage and Pump Station Capacity Selection Document, 11 March 2009
   b. Email from Nancy Powell (Chief, Hydraulics Branch, New Orleans District), 23 August 2009, specifying that a design grade of +8.2ft NAVD88 (2004.65) meets the HSDRRS overtopping criteria
   c. Overtopping output graphs for WB30lv and WB30fw from Robert Bass

2. **Purpose.** The purpose of this memorandum is to provide the documentation behind the 23 August 2009 decision to allow +8.2ft NAVD88 (2004.65) to be used as the minimum levee and floodwall elevation on the entire length of Algiers Canal. For purposes of this MFR, any statements relating to Algiers Canal encompass the entire detention basin (which also includes Harvey Canal).

3. **Original DWSE and Design Levee Grade**. The original Design Water Surface Elevation (DWSE) for Algiers Canal was +5.8ft NAVD88 (2004.65). The justification for this elevation is provided in Reference a. The Authorized Water Surface Elevation (AWSE) was +5.3ft NAVD88 (2004.65), uncertainty was +0.5ft, and resulted in a DWSE of +5.8ft NAVD88 (2004.65). The AWSE was computed from HEC-RAS modeling. Based upon this DWSE, the detention basin levee and floodwall design grade was documented to be +8.5ft NAVD88 (2004.65) with a 1:4 side slope for levees (using rounding to the nearest 0.5ft). The justification for this elevation is also provided in Reference a.

4. **Minimum Hydraulic Design Elevation.** The standard practice of MVN H&H is to provide a minimum hydraulic section and design elevation in 0.5ft increments. The lowest computed height that met HSDRRS criteria was +8.2ft NAVD88 (2004.65), determined in support of the March 2009 documentation (Reference a). The minimum hydraulic 1% design elevation for the WCC Detention Basin provided consistent with H&H standard practice is 8.5ft, although 8.2ft was the actual computed elevation.

5. **Background/Discussion.** In mid-late 2009, Hydraulics and Hydrologic Branch was asked to determine if an elevation of +8.2ft NAVD88 (2004.65) with a minimal levee footprint would meet HSDRRS criteria on Algiers Canal due to height deficiencies that existed along the Algiers Canal levee system. An email from Nancy Powell (Reference b) documented that an elevation of +8.2ft NAVD88 (2004.65) meets the overtopping criteria for existing conditions. Additionally as this is a detention basin, where relative sea level rise is not an issue, the +8.2ft NAVD88 (2004.65) elevation is valid for both levees (with a side slope of 1:3 to minimize levee footprint) and floodwalls. This MFR provides the technical basis for that prior decision/documentation.

6. **Technical Basis for +8.2ft NAVD88 (2004.65).** As stated in paragraphs 4 and 5, the +8.2 ft NAVD88 (2004.65) elevation for levees with a 1:3 slope and floodwalls meets the maximum allowable HSDRRS overtopping criteria. The probabilistic overtopping calculation output from the van der Meer's wave runup and overtopping equation for the levee is shown in graph WB30njplv and the probabilistic overtopping calculation output from the Franco-Franco overtopping equation for the floodwall is shown in WB30njpfw (Reference c). Included on these graphics are the Hydraulic design parameters used in these equations; Surge, Wave Height, and Wave Period and the q50 and q90 overtopping rate.

7. **Decision.** The 2009 re-examination of the hydraulic design of the Algiers Canal levees and floodwalls (Reference b) remains valid. The minimum levee and floodwall design elevation along the entire length of Algiers Canal is +8.2ft NAVD88 (2004.65) with a 1:3 side slope for levees.

JULIE Z. LeBLANC, P.E.
Chief, Hydraulics and Hydrologic Branch

CEMVN-ED 11 March 2009

MEMORANDUM FOR Chief PPPMD, Attn: Kevin Wagner, Sr. Project Manager

SUBJECT: Transmittal of Detention Stage and Pump Station Capacity Selection Document for WBV 90 GIWW West Closure Complex

1. This memorandum transmits the Detention Stage and Pump Station Capacity Selection Document for WBV 90 GIWW West Closure Complex, West Bank and Vicinity Hurricane and Storm Damage Risk Reduction in Orleans, Jefferson, and Plaquemines Parishes. The decision in this document affects the GIWW WCC as well as levees, floodwalls, and fronting protections along Harvey and Algiers Canal.

2. This Selection Document has been through ITR and the ITR certification is attached.

Walter O. Baumy

WALTER O. BAUMY, JR., P.E.
Chief, Engineering Division

Reference a

WBV-90
GIWW West Closure Complex

Detention Stage and Pump Station Capacity Selection Document

CEMVN-ED

March 11, 2009

WBV-90: Gulf Intracoastal Waterway West Closure Complex Detention Stage and Pump Station Capacity Selection Document

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WBV-90: Gulf Intracoastal Waterway West Closure Complex Detention Stage and Pump Station Capacity Selection Document

1.0 Introduction

The Project Delivery Team through the Alternative Evaluation Process (AEP) selected the Gulf Intracoastal Waterway (GIWW) - West Closure Complex (WCC) as the recommended alternative to provide risk reduction for West Bank and Vicinity Hurricane Projects located along Harvey and Algiers Canal in Plaquemines, Orleans, and Jefferson Parishes. The WCC will be located southwest of the confluence of the Harvey and Algiers Canals as shown in Figure 1.

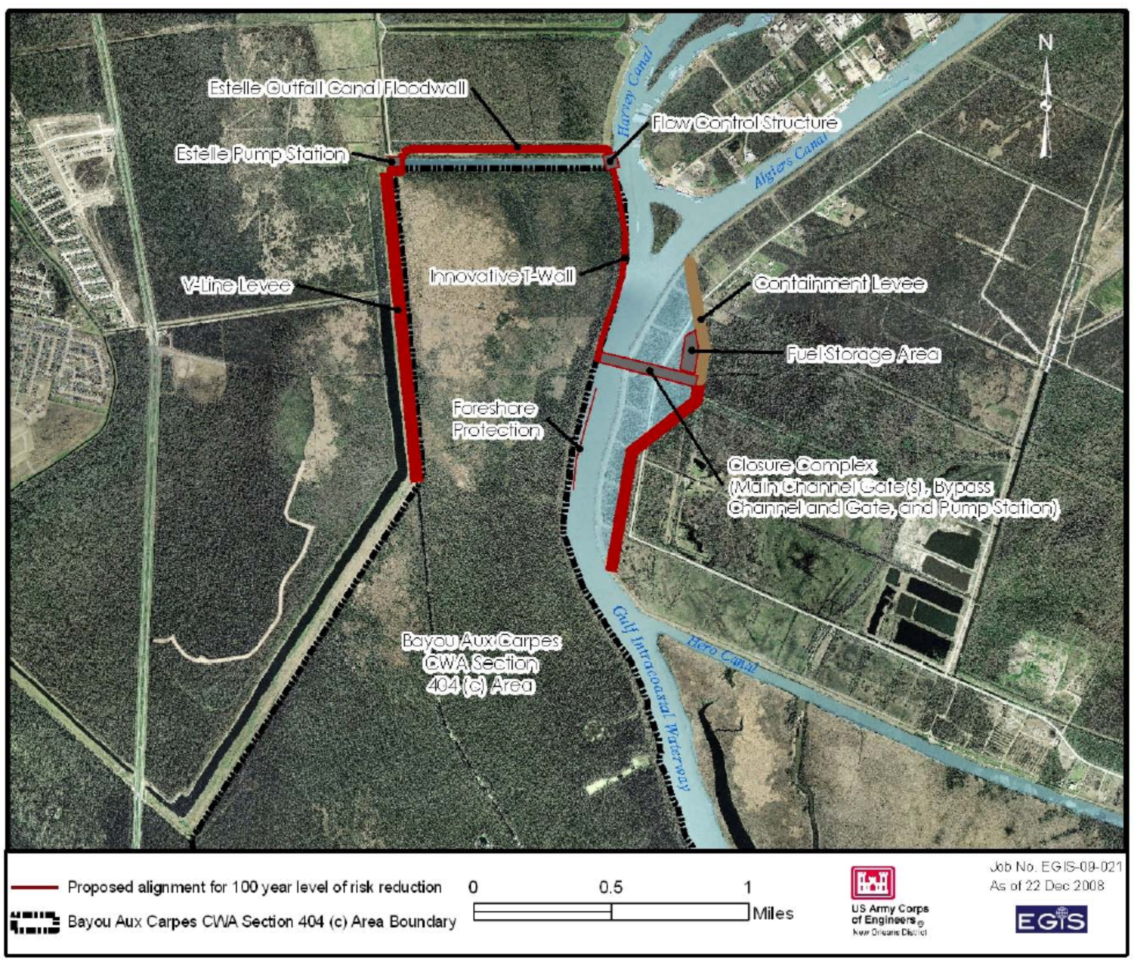


Figure 1. West Closure Complex (WCC)

The WCC will include the following features: navigation gates, 20,000 cfs pumping station and dredging of Algiers Canal. The complex will be designed to provide risk reduction from a hurricane event that will produce a 1% exceedence surge elevation and associated waves (commonly referred to as the 100 Year Design Storm). During a hurricane event the new navigation gates will be closed to prevent storm surge from entering the Harvey and Algiers Canals. Closure of the gates during a storm event will remove 26 miles of levees, floodwalls, fronting protections, and closure gates that parallel the banks of the Harvey and Algiers Canals from the "front line" of the Hurricane and Storm Damage Risk Reduction System (HSDRRS). When the gates are closed the Harvey and Algiers Canals will act as a detention basin for storm runoff pumped into the canals from several locally operated drainage pump stations located along the canals. During these events the new 20,000 cfs pumping station will evacuate water from the detention basin into Bayou Barataria.

For information on alternatives considered, the evaluation process to select the preferred alternative, the environmental impact of the alternatives and the decision document approving the GIWW-WCC reference the following documents:

- **Engineering Alternatives Report (EAR) –** EAR-W-19 Sector Gate South Innovation Study, Volume I – Report; September 10, 2008
- **Individual Environmental Report (IER) & Decision Document-** IER 12
- **Project Description Document (PDD) –** PDD 9 – Harvey Algiers

The purpose of this paper is to establish the design criteria for the area that will act as a detention basin behind WCC when the gates are closed. These criteria will include: hydraulic elevations during a storm event, WCC pump station capacity, design elevation for levees and floodwalls, and backflow prevention requirements.

2.0 Executive Summary

The design of all elements of the detention basin and the drainage pump station considered conditions that could reasonably be expected to occur during the lifetime of the system. Hurricane wind, waves, storm surge, rainfall events, settlement and consolidation of levees, live and dead loads on all structures in accordance with USACE Guidelines and applicable codes, foundation pile capacities, seepage, barge impact, pump capacities versus various head conditions and loss of electrical power were addressed in the design to minimize risk.

Based on these considerations the following table summarizes the design elevations and recommendations for the detention basin. The analysis and justification for these numbers can be found in the appropriate sections of this report.

Table 1. Summary of Design Recommendations

Detention Basin Design Rainfall:	10 Percent (%) exceedence extratropical, 24-hour rainfall
WCC Pump Station Capacity:	20,000 cfs
Channel Improvements – Algiers Canal:	Increase bottom depth by 3.0 feet (max) by dredging from Belle Chasse Tunnel to Hero Cutoff
Channel Improvements – Harvey Canal:	None required
Detention Basin Levee and Floodwall Design Grade:	El. 8.5 with a 1:4 slope for levees
Design Water Surface Elevation (DWSE):	El. 5.8
Threshold for Backflow Suppression	Invert El. 7.0
Structural Superiority for Applicable Detention Basin Components	1.0 above Design Grade (i.e. El. 9.5)

Note: All elevations in feet, NAVD 88 (2004.65)

3.0 Goals and Objectives

Hydrology and Hydraulics

- Establish the appropriate detention basin design rainfall for conditions where the WCC gate is closed.
- Determine the range of water elevations in the canals for the closed gate condition.
- Determine the appropriate combination of WCC pump station capacity and detention basin features that result in safe operations during a hurricane event.

Geotechnical

- Determine safe water elevations based on stability analysis of existing levees in accordance with Factors of Safety (FOS) and criteria as defined in the HSDRRS Design Guidelines.
- Determine requirements for channel improvements for the Harvey and Algiers Canal that achieve an optimum balance between the WCC pump station capacity and the detention basin features.

Levees, Floodwalls, Closure Gates, and Fronting Protections

- Provide a detention basin system with risks equal to or less than that afforded for the GIWW WCC.
- Establish the design elevation for the levees and associated structural features located along Harvey and Algiers canals.
- Develop structural superiority requirements for structural features associated with the detention basin.
- Establish when backflow prevention is required at the locally owned and operated pumping stations within the detention basin.

Navigation

- Provide safe navigation for all possible operating conditions. Identify conditions when navigation needs to be suspended.
- Determine if the navigation canals have sufficient capacity to safely pass navigation and to serve as a detention basin without additional canal improvements.

Quality

- Determine a combination of pump station capacity at WCC with adequately sized levees, floodwalls, and fronting protection features that minimize risk.

Time & Cost

- Produce a design that minimizes both time and cost in the acquisition of additional rights-of-way parallel to the detention basin levees and floodwalls and thus contribute to accelerating completion dates of these projects.
- Improve construction duration and cost by taking advantage of the reduced design elevations of levees, floodwalls, etc. along Harvey and Algiers Canals that result from construction of WCC.

Partnership with Locals

- Acknowledge the importance of having a communication plan for cooperation with Parishes that have pumping stations discharging into the Harvey and Algiers Canal

4.0 Background

The WCC, located southwest of the confluence of the Harvey and Algiers Canals, will reduce the risk of flooding from a 1% exceedence surge elevation and associated waves. The detention basin, located on the protection side of the WCC, will be bordered by approximately 26 miles of levees/floodwalls, two navigation/waterway control structures, and a new drainage pump station.

There are nine interior drainage pump stations that discharge directly into the Harvey and Algiers Canals (see Appendix C for a map). With the WCC closed, evacuation of the discharge from the parish pump stations will be blocked from flowing into Bayou Barataria. Consequently, a new pump station will be necessary to evacuate water from the detention basin to Bayou Barataria, maintaining the existing drainage pattern. The required capacity of the new WCC pump station is dependent upon the detention basin design rainfall and storage capacity within the detention basin. The detention basin storage capacity is dependent on the channel capacity of the two canals and the height of the levees/floodwalls and the WCC features.

A combination of hydraulic modeling and geotechnical analyses was used to evaluate the pump station and detention basin storage capacity.

5.0 Hydrology and Hydraulic Analysis

5.1 Selection of Design Rainfall

The detention basin and pump station must be capable of providing protection from rainfall associated with tropical events when the WCC gates are closed. EM 1110-2-1413 (Hydrologic Analysis of Interior Areas) reads “If a local storm drainage system is in existence, then the minimum facility should pass the local system design event with essentially no increase in interior flooding.” The minimum facility was equated to the design event for which the interior pump station(s) or drainage structures have been sized. The interior drainage system and pump stations for Jefferson and Orleans Parishes that pump into the Harvey and Algiers Canals have been designed for the 10% exceedence 24-hour extratropical rainfall event under the Southeast Louisiana (SELA) authorization. A feasibility study is presently underway to look at interior flooding for the Belle Chasse area of Plaquemines Parish, which includes the area of the parish on the protected side of the WCC. For the detention basin design, the 10% exceedence 24-hour extratropical rainfall event (9.1 inches) was selected as the minimum facility required.

For the Interagency Performance Evaluation Team (IPET) engineering and operational risk and reliability analysis, the IPET team developed rainfall totals associated with the 152 storms that comprise the storm set for IPET, Louisiana Coastal Protection and Restoration study (LACPR), and the 1% design elevation Advanced Circulation Model (ADCIRC) modeling. This rainfall was also considered in the selection of the detention basin design rainfall. The rainfall totals were estimated based on National Aeronautic and Space Administration (NASA) data that correlates rainfall intensity and volume to hurricane characteristics such as wind speed, radius to maximum winds and pressure. Joint Probability Method with Optimal Sampling process (JPM-OS) probability analysis was applied to the rainfall totals to determine the tropical rainfall with a 1% chance of occurring in any given year.

In addition, the rainfall totals associated with storms that produced a surge elevation in the range of the 1% surge elevation in the Harvey and Algiers Canal area were averaged to determine the rainfall total associated with the storms producing a 1% surge elevation. This total was calculated to be 9.1 inches.

5.2 Parish Drainage Pump Stations

The nine (9) existing and functioning parish drainage pump stations (DPS) within the confines of the WCC and their capacities are listed below.

Table 2. Existing Local Drainage Pump Stations and Capacities

Drainage Pump Station	Capacity (cfs)
S&WB # 11	1,750
S&WB # 13	4,650
Belle Chasse # 1	3,550
Belle Chasse # 2	990
Planters	2,360
Hero	3,900
Cousins # 1, 2 & 3	6,520
Estelle # 1	550
Estelle # 2	1,140

Future SELA pump requirements have been considered in the analysis. Under SELA, there are plans to expand the capacity of New Orleans Sewerage and Water Board Pump Station No. 13 by 2,000 cfs to a total capacity of 6,650 cfs.

5.3. Detention Basin Analysis

HEC-RAS and HEC-HMS modeling was performed to evaluate different detention basin design grades, Algiers Canal channel capacities, and WCC pump capacities. The Federal Emergency Management Agency (FEMA) flood insurance study HEC-RAS and HEC-HMS models of the interior areas for which the parish pump stations pump into the Harvey and Algiers Canals were acquired and modified to include the future SELA pump requirements. The 10% exceedence, 24-hour extratropical rainfall was applied uniformly to the HEC-HMS interior models, and runoff from these models was used in the HEC-RAS interior models to calculate the output flow hydrographs for each of the nine pump stations that drain into the two canals. The output flow hydrographs from each station were then used as input hydrographs for a separate HEC-RAS model of the Harvey and Algiers Canals. This HEC-RAS model was developed from a HEC-RAS model being used for the Donaldsonville to the Gulf of Mexico feasibility study. This model includes the Harvey Canal Sector gate, which was closed during the model runs. In the HEC-RAS canal model, rainfall is also uniformly applied to the canals. The "pump-on" elevation for the WCC pump station was assumed to be between El. 2.0 and El. 2.5 NAVD 88 (2004.65).

The modeling effort consisted of iterations; a WCC pump station size was assumed, the HEC-RAS canal model run with the 10% exceedence, 24-hour rainfall, and maximum detention basin still water levels (SWLs) determined from model results. Target SWLs, in one foot increments, were provided for use in performing the model runs and developing the different design scenarios. The target SWL elevations used were El. 5.0, 6.0, 7.0, and 8.0 NAVD 88 (2004.65), with the upper end of the Algiers Canal being the governing location (i.e., highest point in the water surface profile). Multiple model runs were made to determine the WCC pump station size for each target SWL. The table in Appendix B shows the scenarios used to develop the recommended design water surface elevation (DWSE).

Using the results of the geotechnical analysis as screening criteria for target SWLs, the HEC-RAS canal model runs showed that a 23,750 cfs pump would be required at the southern location to maintain a maximum still water level of 5.9 ft in the detention basin.

To improve its hydraulic efficiency, Algiers Canal channel capacity was increased by enlarging the channel from Belle Chasse tunnel southward to the Hero Cut off, a distance of 25,966 linear feet (4.91 miles), to a depth approximately three feet below the current invert. With the larger channel, the HEC-RAS canal model results showed the WCC pump station could be reduced to 20,000 cfs and maintain a maximum still water level of 5.3 ft in the detention basin.

Both scenarios will meet the hydraulic goals for the WCC. Consideration of cost, levee cross section for stability, and real estate requirements show that the 20,000 cfs WCC pump station, with Algiers Canal enlarged, a DWSE of 5.8 ft (5.3 ft plus 0.5 ft for uncertainty), and levee and floodwall elevations in the 8.5 ft range is the most effective plan to meet the overall objectives of the project.

To finalize the design grade for the levees and floodwalls, considerations for risk and uncertainty, wind driven waves, subsidence and sea level rise, and settlement of levee lifts were addressed.

5.4 Waves

During a tropical event, tropical winds will create waves in the detention basin and can cause wave overtopping. A wave analysis was performed to calculate a design wave height and wave period that could occur in the detention basin. STWAVE modeling performed for the 1% design elevations did not have sufficient resolution to be used in Harvey and Algiers Canals. An analysis of a fetch-limited wave growth for an enclosed canal was performed using Bretschneider's Formula and a design water surface elevation inside the detention area of 5.8 feet. The wind speed was assumed to be 112.9 ft/sec, or 77 mph, which is the maximum sustained winds for a 1% storm. The width of the Harvey and Algiers Canals is about 500 feet in the perpendicular direction. From Bretschneider's Formula, the calculated wave height is 1.5 feet and the peak period 2.5 seconds with this fetch (and infinite duration).

These values do not incorporate the effects of wave reflection within an enclosed body. Wave reflection can dampen or amplify the wave heights. On the other hand, wave damping due to the presence of objects is not considered. Based on engineering judgment and experience, a wave height of 1.5 feet was used along with a wave period of 2.5 seconds in the Harvey and Algiers Canals.

5.5 Uncertainty

For the volume of water that will be pumped into the detention basin, using a uniformly distributed rainfall in modeling is a conservative assumption that accounts for some uncertainty in the modeling. The uniform rainfall distribution results in the pump stations beginning to pump at the same time. During a tropical event, rain does not fall across the project area in a uniform manner at the same time. From historical pump data and discussions with the local pump operators, it is known that all pump stations do not start pumping at the same time.

The uncertainty in the water surface elevation is a function of the accuracy of the geometry input, flow input, and the quality of gage and pump records for calibration. An additional increment of 0.5 ft is reasonable and appropriate for the level of modeling performed. Adding this value to the maximum still water level of 5.3 ft results in a design water surface elevation (DWSE) of 5.8 ft for the recommended alternative.

The levee/floodwall design process is documented in the HSDRRS Design Guidelines. For the detention basin analysis, a standard deviation of 0.5 ft was applied. Standard deviation values of 10% of the average significant wave height and 20% of the peak period were used (Smith, pers. comm.). In absence of data, all uncertainties are assumed to be normally distributed.

5.6 Maximum Allowable Wave Overtopping

The authorized design still water is based on the water level, with model accuracy uncertainty included, that was calculated for a 10% exceedence rainfall event and considers the 20,000 cfs pump station operating and the sector gate(s) closed. For the design still water, wave height and wave period, the maximum allowable average wave overtopping is 0.1 cfs/ft at 90% level of assurance and 0.01 cfs/ft at 50% level of assurance for grass-covered levees. For the design still water, wave height and wave period, the maximum allowable average wave overtopping is 0.1 cfs/ft at 90% level of assurance and 0.03 cfs/ft at 50% level of assurance for floodwalls with appropriate protection (armoring) on the protected side.

Using the methodology documented in the HSDRRS Design Guidelines, for the recommended plan, the wave overtopping calculated meets the maximum allowable average wave overtopping requirements in the detention basin. The maximum rates calculated were 0.013 cfs/ft for the 90% level of assurance and 0.0012 cfs/ft for the 50% level of assurance.

5.7 Subsidence and Sea Level Rise

Previous design efforts included in the *Elevations for Design of Hurricane Protection Levees and Structures* report, dated October 2007, included an additional 2 feet added to surge elevations in the West Bank and Vicinity project area to account for the increase in surge elevations resulting from global subsidence and sea level rise. The gate elevation at the closure structure, +16 feet, has these two factors built into the required elevation.

There are several other design elevations which were checked to see if they should be adjusted for subsidence and sea level rise including still water level, the water level when the gate is closed, and the levee height in the detention area.

Historical data shows that subsidence in the project area has been occurring at the rate of 0.25 feet per 50 years. Since the detention levee and floodwall heights are designed using the DWSE, the water level and the levee and floodwall height are relative to each other. Levees and the detention area channel should experience the same rate of global subsidence; therefore, the relation between the water surface elevation in the canal and the levee and floodwall elevations will not change in the future.

If the levees are maintained at El. 8.5 throughout the project life, an additional 0.25 feet of storage may be available in the detention channel in the future. This could be used to account for future storage requirements due to increase in runoff volume over and beyond what has been projected in the SELA project. Based on the 1999 GDM and current levee crown elevations, as a result of settlement of the levees, one additional levee lift with some maintenance in limited areas is needed for the levees along the Harvey and Algiers Canals to remain above El. 8.5 through 2057.

The present gate closure criterion for the WCC gates is when the stage during a tropical event is expected to reach 3.0 ft. Normal water levels in the GIWW, Harvey and Algiers Canals will likely increase due to subsidence and sea level rise. This will result in the gates being closing earlier, remaining closed longer, and closing more frequently in the future.

5.8 Resiliency of the Detention Basin

To check the resiliency of the levee and floodwall design elevations in the detention area, the HEC-HMS and HEC-RAS interior models were run with the 1% chance exceedence extratropical rainfall total of 13.2 inches in 24 hours. The output hydrographs were used in the HEC-RAS canal model with the 20,000 cfs WCC pump station in place, the Algiers Canal enlarged, and the Harvey Canal Sector Gate and WCC gates closed. The resulting peak water level was 8.0 feet; this is below the 8.5 foot elevation of the levees and floodwalls. (Note this is in contrast to the design conditions of a 1% exceedence surge elevation and associated waves with concurrent 10% exceedence 24-hour rainfall event of 9.1 inches. The resiliency check considers the rarer scenario during which the 1% exceedence rainfall accompanies a gate closure.)

5.9 Hydraulic Design Criteria

The DWSE is the stage or water level used in deterministic analyses such as the geotechnical and structural stability analyses and seepage analysis. The authorized water surface elevation (AWSE) and its associated uncertainty at the selected confidence limit are used to determine this stage. For this analysis, the AWSE is 5.3 ft, and the DWSE is 5.8 ft.

In geotechnical analysis, the stability analyses require both the DWSE and the low water surface elevation. The probable theoretical low water elevation with the sector gates open was determined from the ADCIRC model results for the 152 storm set for an ADCIRC output point near the WCC. This value is -2.5 NAVD88 (2004.65). Note, these ADCIRC model runs do not consider the WCC gate complex in place.

The low water elevation with the sector gates closed was calculated to be -1.5 NAVD88 (2004.65) and is indicative of a drawdown inside the protected area due to a hurricane not on the critical path. This water elevation can be controlled through closing the gates and pump operations during extreme low water events. The criteria for gate closure and pump operations will be dictated by the water control plan.

5.10 Navigation

The Harvey and Algiers Canals are primarily navigation channels that must continue to safely and efficiently convey traffic throughout the year when there is no tropical event.

All scenarios investigated were carefully checked to assure the integrity of the navigation channels was maintained. This included sizing gates on the closure structures such that they had minimal impact to existing and authorized infrastructure and environment. Also, where the Algiers Canal will be dredged, the dredging will impact minor/minimal risk as the goals are compatible: more storage capacity along with depths that are deeper than what is required for navigation. Additionally, a separate study found that while a small increase in shoaling is expected to occur, maintenance dredging will not be required for several decades.

6.0 Geotechnical Analysis

6.1 General

The geotechnical analysis included checks of both existing conditions and proposed improvements of features along Harvey and Algiers Canals. These preliminary design calculations included global stability and seepage that are documented primarily in prior reports noted in Section 1.0. Engineers considered several critical load cases. The detention basin experiences its highest water with gates closed and pumps in operation with the design water surface elevation (DWSE) governed by the routing of rainfall runoff from the local pump stations through the canals to the WCC pump station. Three different DWSE conditions were examined during the course of work as suggested by the hydraulics analyses. Engineers also examined stability under the low water conditions and checked the effect of proposed canal dredging on seepage.

6.2 Global Stability

The Algiers Canal levees and floodwalls were initially analyzed for two proposed design heights and two water elevations. The two conditions analyzed were as follows. These initial analyses did not include canal dredging.

Table 3. Initial Design Parameters for Geotechnical Analyses

Top of Detention Basin	DWSE	Low Water
Elev 7.0	5.0	-1.0
Elev 10.0	8.0	-1.0

Calculations indicated that levees would need to be widened and stabilized with berms in order to meet the required Factors-of-Safety for global slope stability as specified by HSDRRS Design Criteria. The required additional rights-of-way were estimated to be 22.5 acres of additional land for DWSE = 8.0, as opposed to an additional 9.5 acres for DWSE = 5.0. Considering the sensitive and time-consuming process required to obtain rights-of-way in the residential and industrial areas adjacent to the canal, the alternative requiring the least quantity of real estate acquisition was considered most favorable. Thus, a pump station configuration that could maintain the DWSE as close to El. 5.0 as feasible became preferable and guided the final recommendation, as further defined by the hydraulic modeling.

The results of the hydraulic analysis including considerations for changing future conditions and uncertainty resulted in the preferred design conditions summarized in the table below.

Table 4. Final Design Parameters for Geotechnical Analyses

Top of Detention Basin	DWSE	Low Water
Elev 8.5	5.8	-1.0.

Two critical project reaches (WBV-47-and WBV-49) were analyzed for the detention basin height of 8.5 (see project map in Appendix C). The analyses of the existing levee cross-sections on these reaches consisted of both stability and seepage analyses performed per the latest HSDRRS Design Guidelines. For project reach WBV-47, a small reach of levee near the Algiers Lock would require a stability berm outside the existing ROW. For WBV-49, a reach of approximately one mile (from Station 443+25 to Station 496+00) would require a stability berm outside the existing ROW, approximately 30 feet to meet the current criteria. In addition, if the WBV-47 levee is constructed with additional height of overbuild as is common for earthen construction, calculations indicate the need for a levee setback. At this location, an overbuild elevation of 9.7 could trigger the need for a levee setback to maintain a HSDRRS minimum FOS =1.4 for low water.

Hydraulic modeling illustrated the need to dredge part of the Algiers Canal to improve its hydraulic efficiency. The preliminary analysis indicated the need to dredge about 3 feet below the current invert of the Algiers Canal from the Belle Chasse Tunnel southward to Hero Cutoff. Slope stability of the existing levees adjacent to the portion of the canal to be dredged was analyzed for low water at El. -1.0. Analysis of WBV-49 showed the dredged section stability has a FOS of more than 1.8 using Spencer's method, which is greater than the required minimum FOS of 1.4. Preliminary analysis of WBV-6a.1 indicated that slope stability into the dredged channel also meets the minimum HSDRRS Design Guidelines for slope stability.

More recent hydraulic computer modeling has shown that the low water elevation for the open gate condition could be as low as elevation -2.5 for certain rare events. A preliminary review of WBV-47 levee stability analysis toward the canal indicates the slope stability factor-of-safety could drop below the Spencer Method minimum required FOS of 1.4 for this extreme low water condition. Engineers will need to further examine and take appropriate actions to protect the integrity of the system under this load condition. Because this potential condition occurs during tropical storm events when navigation will likely be halted, one preliminary solution will be to include a requirement in the O&M manual to close the WCC gates under this scenario to maintain a higher water level in the detention basin.

6.3 Seepage

Seepage along the Algiers and Harvey Canals has been evaluated in accordance with the latest HSDRRS Design Criteria. Results are presented in the Algiers and Harvey Canals geotechnical reports that are part of the documents listed in Section 1.0. The seepage analysis was based on existing canal geometry, DWSE = 8.0 and a detention basin elevation of 10.0. The initial seepage analysis did not include dredging part of Algiers Canal between Belle Chasse Hwy and Hero Cutoff.

Geotechnical engineers revisited the seepage analysis of the initial geotechnical reports to assess the possible impacts of dredging on seepage. The only project reach for which calculations indicate a seepage concern was WBV-6a. The initial seepage analysis for this reach is presented in a geotechnical report titled “WBV-6a Algiers Canal west side levee between Belle Chasse Highway and Hero Cutoff,” prepared by URS consultants and dated January 2008. The report details the check of four seepage cases for the three soil reaches identified within WBV-6a. Foundation conditions there include two sand layers: a shallow thin sand layer 2 to 3 feet in thickness between approximate elevations -12 and -17, and a deeper 20-foot-thick sand stratum between elevations -17 and -40.

Two of the seepage cases were for shallow sands between El. -12 to -15 and El. -13 to -17. In both cases the shallow sands intercept the existing canal bottoms, so dredging the channel would reduce the entrance distance. As the original seepage calculation considered DWSE = 8.0, engineers determined that the small decrease in entrance distance would be more than compensated if the detention basin DWSE is set to the lower water elevation of 5.8. This analysis used the HSDRRS Design Guidelines minimum requirements of Factor of Safety = 1.6 at design water surface elevation and FOS = 1.3 at Project Grade. Excess heads at the levee toe were calculated using the analytical procedures and design values in DIVR 1110-2-400 (1998) blanket theory. The criteria also require FOS for heave of at least 1.20. The FOS calculated in the Soils Report of January 2008 are 5.9 and 8.4 for the shallow sands for water elevation of 10.0 and 8.0, respectively. Using the lower water elevations of 8.5 to top of the detention basin and DWSE of 5.8 and reducing the entrance distance due to dredging, the calculated seepage FOS was found to increase slightly to 6 and 9 respectively.

The other two cases examined were for deeper sands. Deep sands between El. -17 to El. -37 are located between Station 980+00 to 1038+43 (Belle Chasse Highway to Whitney Barataria Pumping Station). The deep sands between El. -20 to El. -40 are also found from Station 1111+06 to Station 1230+00 (near the Hero Cutoff). Dredging the Algiers Canal to between El. -18 and El. -19.5 will be close to or will penetrate into these deeper sands. For analysis purposes it was assumed that dredging would penetrate the sand layer. The January 2008 report assigned a different permeability for the two deeper sands. For the dredged channel conditions, engineers assumed the entrance distance for seepage would decrease and so analyses were made using a shorter entrance distance of 250 feet. For the sands between El. -20 to El. -40 with a permeability of 50×10^{-4} cm/sec, the FOS for seepage exceeds the HSDRRS Design Guidelines minimum requirements (FOS of 1.6 for DWSE 5.8 and FOS of 1.3 for top of protection). The second deeper sand between El. -17 to El. -37 with a permeability of 200×10^{-4} cm/sec also results in FOS for seepage above the current criteria.

The Factors of Safety for heave for all the sand layers are above the required FOS of 1.2 for the lower entrance distances.

In addition, the January 2008 report reviewed the data collected from piezometers installed along the Algiers Canal for the existing conditions (without dredging). Of the 5 piezometers only two indicated a correlation with the canal stage. An analysis of the two piezometers, assuming a correlation with the canal stage at elevations 8.0 and 10.0, resulted in FOS for seepage (exit gradient) and heave above the minimum required FOS.

In summary, based on the above analyses including seepage analyses to date for DWSE El. 5.8 and a detention basin height of El. 8.5 for WBV-6a and WBV-49, the proposed dredging will not decrease the seepage FS or heave FS below the minimum requirements of the HSDRRS Design Criteria.

6.4 Settlement

Geotechnical engineers examined settlement of the levees based on the 1999 General Design Memorandum for this project and current levee crown elevations. It has been estimated that one additional levee lift with some maintenance in limited areas would be required to maintain the levees along the Harvey and Algiers Canals at or above El. 8.5 through 2057. The additional levee lift and/or maintenance lifts are included in the project life cycle costs.

6.5 Proposed Retention Basin Criteria

There is on-going discussion of adopting specific design criteria for risk reduction features that are not directly exposed to hurricane loading. The levees, walls and gates along Algiers and Harvey Canals meet this definition since they will serve as part of a detention basin when the WCC gates are closed. These revised criteria would take into account the less critical role of features in the secondary line of protection by allowing less stringent FOS for design. Such criteria applied to Algiers and Harvey Canals would have a positive effect on the calculated stability of all features. However, these criteria are not being applied at this time and all engineering analyses to date apply the standard HSDRRS Design Guidelines. Implementation of the revised criteria will be evaluated during final design.

7.0 Recommendations

The 10% exceedence, 24-hour extratropical rainfall event from NOAA TP 40 is recommended for use as the design event in developing the detention stages and design pump station capacity. As described above, this event reflects the design rainfall used for the interior storm drainage pump stations and roughly equals the design rainfall for a 1% exceedence tropical event as determined from JPM-OS.

Based on the preceding analyses, the recommended WCC pump station capacity is 20,000 cfs for use with the associated DWSE and detention stage of El. 5.8 NAVD 88 (2004.65) and the required 3 feet of dredging in the Algiers Canal from the Belle Chasse tunnel to Hero Cutoff. The recommended dredging reduces the required WCC pump station pumping capacity by 3,750 cfs, which reduces both initial construction costs and long term operation and maintenance. The combination of the 20,000 cfs pump station and the DWSE of 5.8 allows for a lower design grade (discussed below), which reduces the scope of the twenty five projects comprising the detention basin. In some instances, the lower DWSE of the detention basin all but eliminates the need for further construction. Because El. 5.8 is slightly above the highest stage on record for the area, there is a high confidence that existing levees can safely withstand the design loads with minimal additional work.

7.1 Recommended Design Grade

Selection of a design grade of El. 8.5 NAVD 88 (2004.65) is recommended. This design grade builds upon the 10% exceedence, 24-hour extratropical rainfall DWSE and accounts for risk and uncertainty, waves (satisfying overtopping criteria), subsidence and sea-level rise. Settlement analyses using existing data indicate one additional lift will be required to maintain levees at this elevation through 2057. Because levees and floodwalls behind the WCC are not directly exposed to storm surge, subsidence and sea level rise effects are not factors for the design grade on the detention basin features. No increase in elevation of the detention levees and floodwalls is required over the design life of the project and the 2007 elevation is equal to the 2057 elevation, both of which are El. 8.5. Furthermore, a check of the 1% exceedence, 24-hour extratropical rainfall with gates open resulted in a maximum stage in the canals of El. 6.7, below the proposed DWSE of 8.5. This provides assurance that the detention basin system will perform even under the more extreme rainfall. Existing levee crowns range between El. 8.5 and 10.0, so attaining El. 8.5 will not take a large measure of effort. Where existing levees exceed El. 8.5, the additional elevation will function as overbuild and will settle down to El. 8.5 over the project life.

7.2 Recommended Structural Superiority

Selection of structural superiority of one foot (1’-0”) above the recommended design grade is recommended for all fronting protections. The HSDRRS Design Guidelines specifies providing 2’-0” of structural superiority on projects that would prove especially difficult or expensive to retrofit or rebuild should additional height be required to maintain the 1% design grade. The current practice is to apply structural superiority to all pump station fronting protections. While the fronting protections in the detention basin will not be on the “front line” of the HSDRRS, some structural superiority is warranted. To determine the amount of structural superiority to apply to these projects, the following factors have been considered:

(1) Sea level rise: As stated earlier in this document, sea-level rise will not affect the functionality of the structures as a detention basin, thus no sea-level rise component needs to be included in the structural superiority number.
(2) Subsidence: An additional 0.25’ should be added to account for expected future subsidence.
(3) Settlement: During the future timeframe, settlement is not anticipated to exceed 0.5’ based upon the area’s history and the anticipated pile lengths for the structures requiring superiority, thus an anticipated 0.5’ will be added to assure design grade is maintained.
(4) Uncertainty: To account for potential uncertainties in the general investigation conducted to develop the expected sea-level rise, subsidence, and settlement, an additional 0.25’ will be added.
(5) Secondary line of protection: While structural superiority is warranted, the pump station fronting protections being examined here will not directly bear the force of a hurricane storm surge and associated waves. Water elevations in the detention basin are for the most part controlled.

The resulting recommendation is to add 1’-0” of structural superiority.

Similarly, it is the intent of a proposed DIVR to require a type of structural superiority (also requiring 2’-0”). A request for deviation from the DIVR will be required using the same reasoning: that the fronting walls will be subjected to controlled stages and do not, therefore, require as much additional height. Furthermore, the DIVR’s primary intent of preventing erosion at critical structures is addressed by standard armoring practices in use throughout the HSDRRS.

7.3 Recommended Backflow Suppression Threshold

Selection of El. 7.0 NAVD 88 (2004.65) as the threshold for providing backflow suppression is recommended. Where interior pump station discharge tube inverts fall below this elevation, physical backflow suppression either in the form of sluice gates or butterfly valves/gate valves will still be required.

This recommendation is based on an examination of the 1% tropical events and the associated design rainfall, which is approximately equivalent to the historical 10% extratropical rainfall. Under this scenario, WCC gates are closed and the design water surface elevation (DWSE) in the detention basin per earlier discussions is El. 5.8 NAVD 88 (2004.65), a full 1.2 feet below the recommended threshold. Wave action will introduce the potential for some backflow, but the volume of water that would get through the protection under these conditions is anticipated to be very low. Operation of adjacent pumps in combination with the interior drainage storage capabilities can handle the overflow.

This threshold is intended to be a general recommendation only. Each pump station should be evaluated individually to determine the best backflow suppression plan dependent upon number of pumps present, number of pumps with discharge inverts below or near the recommended threshold, and other appropriate physical and operational factors. Hydraulic analysis may be used to justify allowing variances where discharge inverts fall below El. 7.0.

8.0 Estimated Impacts to Detention Basin Projects

Based on preliminary design efforts, even with construction of the WCC and implementation of the above recommendations, some improvements to the existing parallel protection along the Harvey and Algiers Canals will be required. Summarized below are the affected projects and the additional construction anticipated for each. In general, adoption of a DWSE of 5.8 eliminates unbalanced loads for most, if not all, floodwalls thus significantly impacting the final foundation designs.

This information is based on preliminary design efforts. Design assumptions will be checked and verified during design and documented in accordance with the current design criteria. Refer to Appendix A for the anticipated construction cost based on the recommended design elevations for each of the below projects.

WBV-14g.2 - A portion of this contract is part of the detention basin. The reach from the corner of the intersection of the Estelle Outfall Canal and Harvey Canal north toward New Estelle Pump Station will be constructed to the Detention Basin elevation. A reach of approximately 50 feet from the wall's intersection with the Water Control Structure at the end of the Estelle Outfall Canal will consist of T-Wall constructed to elevation +9.5. The T-wall will then transition to a levee section that will be constructed to elevation +8.5 and continue north until transitioning into the New Estelle Pump Station tie-in wall.

WBV-14a.2 - A T-Wall will be constructed on the protected side of the existing levee at two reaches along the levee reach between New Estelle Pump Station and the tie-in wall of the Harvey Sector Gate/Cousins Pump Station discharge walls. T-walls will be constructed to elevation +9.5 with a 20-foot offset from the existing levee reach in order to meet limited ROW constraints. The existing levee would provide flood protection during construction and stay in place.

WBV-38.2 - New pile-supported T-Walls will be constructed to elevation +10.5 and integrate the existing IWalls. The existing I-Walls will remain in place. The existing fronting protection wall will require sluice gates for backflow prevention.

WBV-46.2 - The existing sector gate at Lapalco Boulevard meets the detention basin design elevation requirements. New pile-supported T-Walls will be constructed to elevation +10.5 and integrate the existing floodwalls. The new T-Walls will require piles to support the new floodwalls and the existing I-Walls will be kept in place.

WBV-01, 2a, 2b - Harvey FWs Sector Gate to Hero P.S. - The existing walls are adequate as is. Consideration should be given to checking for lower elevations and changing foundation requirements; re-use materials on other projects.

WBV-3a - Harvey FWs Hero P.S. to Elmwood Marine - The existing walls are adequate as is.

WBV-3b - Harvey FWs Elmwood Marine to Algiers Canal - The existing walls are adequate as is except that impact barriers must be added to protect against barge impact. Consideration should be given to checking for lower elevations and changing foundation requirements; re-use materials on other projects.

WBV-6a.1 - This 10,000 LF stretch of levee is currently under construction to the previously-authorized level of risk reduction. Additional work to improve stability will include some re-shaping and possibly the construction of a small stability berm within the existing ROW. A series of gaps in the levee reach were originally planned to be vehicle gates providing access through the levee, however the lower required levee elevation allows these areas to become vehicle ramps over the levee. The ramps meet the approach and decline-slope requirements to fit within existing ROW on the protected and canal-side of the levee.

WBV-4.2, 5.2, 6.2 – These contracts previously consisted of a series of vehicle access gates along WBV 06a.1. With the lower design elevations many of the gate locations will now be ramps constructed under WBV 06a.1. Gates that will still be constructed are described below.

WBV-4.2 - Three 30-foot swing gates will be constructed. Floodgates will tie into floodwall sections that transition into the adjacent WBV-06a.1 levee on either side. Floodgates will be constructed to provide access to the Algiers Canal for private industry with the levee reach area. Limited footprints on the protected side and the Algiers Canal side of the alignment prevent vehicle and equipment ramps from being used at these locations. Locations where gates were previously required will now be incorporated as ramps under 6a.1.

WBV-5.2- Two 60-foot roller gates will be constructed. Floodgates will tie into floodwall sections that transition into the adjacent WBV-06a.1 levee on either side. Floodgates will be constructed to provide access to the Algiers Canal for private industry within the levee reach area. Limited footprints on the protected side and the Algiers Canal side of the alignment prevent vehicle and equipment ramps from being used at these locations.

WBV-6.2 - Approximately 2,750 linear feet of T-wall and 650 linear feet of levee would be constructed around the tunnel along with five vehicular gates (three on the east and two on the west) and two railroad gates (one on each side). The additional ROW required to construct the new floodwall on either side of the Algiers Canal would be approximately 18 acres.

WBV-47 - Minor reshaping of the levee section is required to increase the stability of the levee along nearly 20,000 LF. An additional 65 ft of permanent ROW would be required along an 8,700-linear foot stretch of levee to construct a protected side berm near the levee's tie-in to the Algiers Lock.

WBV-48 - No additional levee raising or reshaping is required. A levee raise of this levee reach was completed to elevation +10 in 2007. Additional geotechnical work will be completed to confirm stability of the levee for meeting the criteria of the detention basin.

WBV-49 - A stability berm will be constructed on the protected side along a 6,300-LF stretch. Due to houses adjacent to the existing ROW, a reinforced levee would need to be constructed for 2,700 LF.

WBV-07 - Planters P.S. - Fronting protection will be constructed to elevation +9.5 and consist of pile-supported T-walls that tie into the adjacent levee sections. Butterfly valves will prevent backflow. No fronting protection currently exists across the front of the pump station. The fronting protection walls will be constructed to bypass the existing I-Walls or A-frame walls. The existing walls that tie the pump station into the adjacent levee will be left in place.

WBV-08 - S&WB P.S. 13 - Fronting protection will be constructed to elevation +9.5 and consist of pile-supported T-walls that tie into the adjacent levee sections. Sluice gates will prevent backflow. No fronting protection currently exists across the front of the pump station. The fronting protection walls will be constructed to bypass the existing I-Walls or A-frame walls. The existing walls that tie the pump station into the adjacent levee will be left in place.

WBV-10 - BC #1 P.S. - Fronting protection will be constructed to elevation +9.5 and consist of pile-supported T-walls that tie into the adjacent levee sections. Butterfly valves will prevent backflow. No fronting protection currently exists across the front of the pump station. The fronting protection walls will be constructed to bypass the existing I-Walls or A-frame walls. The existing walls that tie the pump station into the adjacent levee will be left in place.

WBV-11 - BC #2 P.S. - Fronting protection will be constructed to elevation +9.5 and consist of pile-supported T-walls that tie into the adjacent levee sections. Butterfly valves will prevent backflow. No fronting protection currently exists across the front of the pump station. The fronting protection walls will be constructed to bypass the existing I-Walls or A-frame walls. The existing walls that tie the pump station into the adjacent levee will be left in place.

WBV-13 - S&WB P.S. 11 - Fronting protection will be constructed to elevation +9.5 and consist of pile-supported T-walls that tie into the adjacent levee sections. Butterfly valves will prevent backflow. No fronting protection currently exists across the front of the pump station. The fronting protection walls will be constructed to bypass the existing I-Walls or A-frame walls. The existing walls that tie the pump station into the adjacent levee will be left in place.

WBV-23 - New Estelle P.S. - The existing I-Wall tie-ins will be replaced with T-Wall or L-Walls. Butterfly valves will prevent backflow. Fronting protection currently in place does not need to be replaced.

WBV-44 – Whitney Barataria P.S. – The existing I-Wall tie-ins will be replaced with T-Wall or L-Walls. Butterfly valves will prevent backflow. Fronting protection currently in place does not need to be replaced.

9.0 Criteria

To provide a reliable system of protection, stringent criteria were used to evaluate the project components including the materials used in their manufacture. Design Criteria applied was in accordance with the HSDRRS Design Guidelines. This criteria was drawn from USACE Engineering Manuals, references that are widely used by practicing professionals and applicable codes.

Appendix A

Construction Cost Summary

Project by Contract Reach	HSDRRS Programmatic Cost Estimate- Base Construction Cost	PDD 9 Construction Base Cost
WBV-07	$11,494,937	$16,402,512
WBV-08	$13,915,096	$14,912,382
WBV-10	$15,623,560	$15,962,454
WBV-11	$5,392,756	$6,825,886
WBV-13	$4,932,811	$7,646,707
WBV-04.2	$11,639,630	$2,751,904
WBV-05.2	$8,713,182	$3,274,728
WBV-06.2	$9,565,116	$14,887,606
WBV-06a.1	$7,428,345	$226,035
WBV-06a.2	$208,068,937	$0
WBV-14a.2	$4,314,766	$23,195,261
WBV-14e.2	$55,198,153	$31,671,972
WBV-14g.1	$41,138,348	$2,525,120
WBV-14g.2	$2,322,485	$39,081,417
WBV-23	$10,059,759	$6,055,548
WBV-33	$14,767,146	$25,932,696
WBV-38.2	$16,500,284	$8,644,371
WBV-39b.2	$18,279,164	$0
WBV-44	$22,351,590	$7,421,298
WBV-46.2	$8,796,465	$9,902,802
WBV-47.1	$6,677,943	$951,811
WBV-47.2	$83,607,867	$0
WBV-48.2	$135,768,623	$0
WBV-49.1	$4,776,298	$1,235,531
WBV-49.2	$72,518,721	$0
WBV-90	$0	$566,457,137
Total	$793,851,982	$805,965,178

Appendix B

HEC-RAS Results for GIWW West Closure Complex, selected alternative

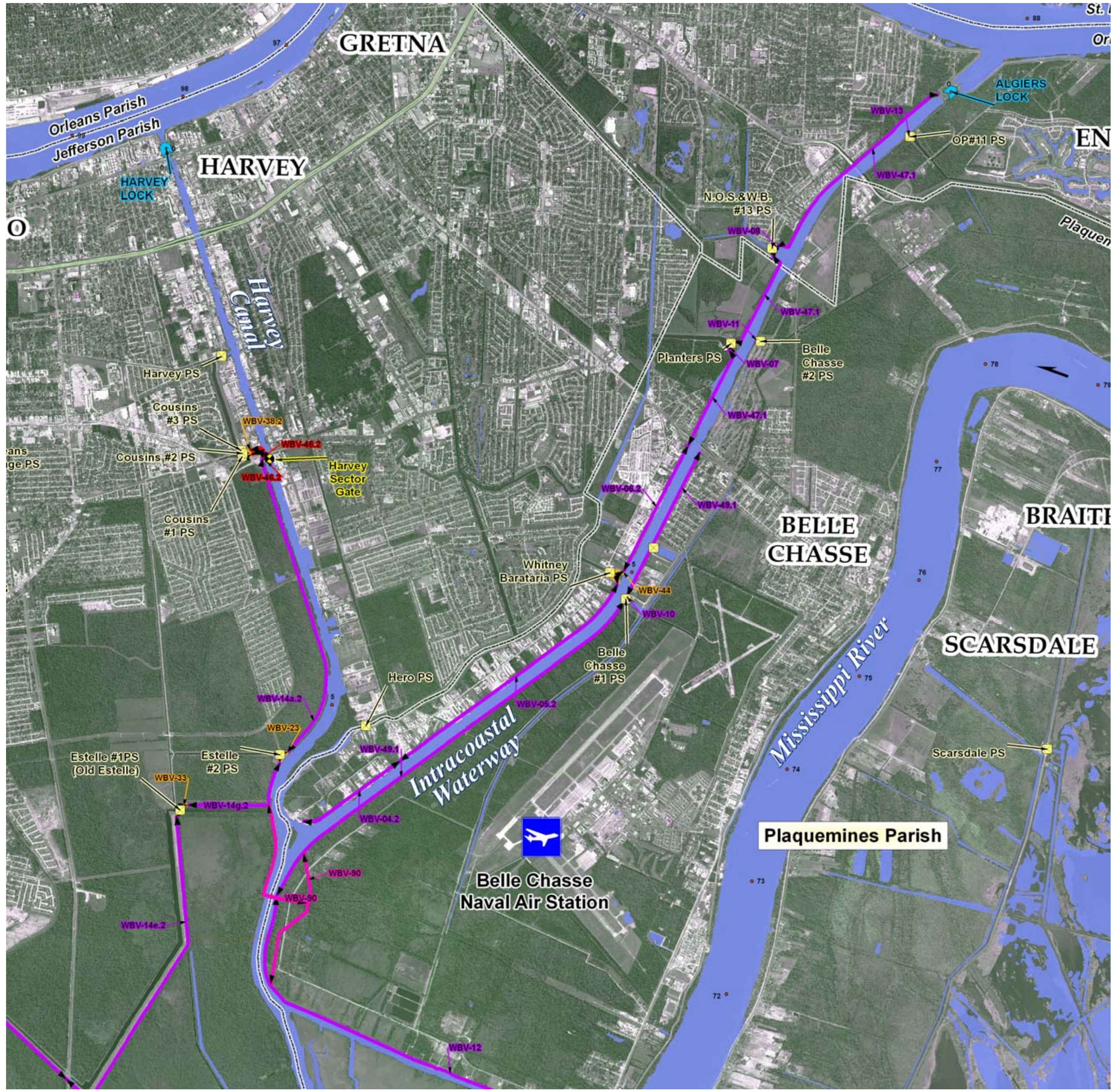
10-year rainfall events		
HEC-RAS Plan number:	**2**	**4**
Run name: **Description:** **Rainfall for Run**	**Chm,20k,10,noOE** Gates Closed, 20,000 cfs pump 10-year	**10chmOP-noOE** Gates Open, no Pumping 10-year
SGS (or ALG) Pump (cfs) **Pump on Elev range**	**20,000** **2 to 3**	**n/a** **n/a**
Algiers Peak Elevation (ft) **Harvey Peak Elevation (ft)**	**5.8** **3.9**	**5.8** **3.5**

100-year rainfall events		
HEC-RAS Plan number:	**11**	**10**
Run name: **Description:** **Rainfall for Run**	**Chm,20k,100,noOE** Gates Closed, 20,000 cfs pump 100-year	**100chmOP-noOE** Gates Open, no Pumping 100-year
SGS (or ALG) Pump (cfs) **Pump on Elev range**	**20,000** **2 to 3**	**n/a** **n/a**
Algiers Peak Elevation (ft) **Harvey Peak Elevation (ft)**	**8.0** **5.9**	**6.7** **4.4**

All elevations contain .5 ft for required uncertainty in modeling

Appendix C

Map showing the proposed location of the West Closure Complex (WBV-90) and other contracts along Harvey and Algiers Canals.



Appendix D

Abbreviations and Acronyms

3DSAD	Computer program
AASHTO	American Association Of State Highway And Transportation Officials
ACI	American Concrete Institute
AISC	American Institute Of Steel Construction
ASCE	American Society Of Civil Engineers
AWS	American Welding Society
BC	Belle Chasse
CEMVN	Corps Of Engineers -Mississippi Valley -New Orleans
CEMVN-ED-H	Corps Of Engineers -Mississippi Valley -New Orleans -Engineering Division Hydraulics And Hydrology Branch
CFS	Cubic Feet Per Second
CWALSHT	Computer program
DFIRMS	Digital Flood Insurance Rate Map
DIVR	Division Regulation
DPS	Drainage Pump Station
ED-F	Engineering Division -Foundation Branch
ED-H	Engineering Division -Hydraulics And Hydrology Branch
El.	Elevation
EM	Engineering Manuel
ENG	Engineering
EPA	Environmental Protection Agency
ERDC	Engineering Research And Development Center -Vicksburg, Mississippi
FOS	Factor Of Safety
FEMA	Federal Emergency Management Authority
FWs	Floodwalls
GDM	General Design Memorandum
GIWW	Gulf Intracoastal Waterway
HEC-HMS	Computer program
HEC-RAS	Computer program
HQUSACE	Headquarters -United States Army Corps Of Engineers -Washington, D.C.
HSDRRS	Hurricane and Storm Damage Risk Reduction System
I Wall	Steel Sheetpile Wall
IGE	Independent Government Estimate
IPET	Interagency Performance Evaluation Taskforce
ITR	Independent Technical Review
JPM-OS	Joint Probability Method With Optimal Sampling
Kt	Frictional Resistance Of Granular Soil On Piles In Tension
LWall	Pile Supported Concrete Wall Shaped Like An L

LF	Linear Feet
LSSRB	Louisiana Department Of Transportation And Development -State Specifications For Roads And Bridges
MOP	Method Of Planes
NASA	National Aeronautics And Space Administration
NAVD	North American Vertical Datum
NOAA TP40	National Oceanic And Atmospheric Administration Technical Paper 40
P.S.	Pump Station
Q-Case	Unconsolidated, Undrained Triaxial Compression Test
Rmax	Radius Of Maximum Wind
ROW	Right Of Way
S&WB	Sewerage And Water Board Of New Orleans
S-Case	Consolidated Drained Direct Shear Test
SELA	Southeast Louisiana
SOW	Scope Of Work
SWL	Still Water Level
T Wall	Pile Supported Concrete Wall Shaped Like An Inverted T
USACE	United States Army Corps Of Engineers
WBV	West Bank And Vicinity Hurricane Protection Project
WCC	West Closure Complex
WES	Waterways Experiment Station, Vicksburg, MS-Now Known As ERDC
WSE	Water Surface Elevation

COMPLETION OF INDEPENDENT TECHNICAL REVIEW
(ER-1110-1-12)

The District has completed the West Closure Complex Detention Stage and PS Capacity Selection Paper for WBV 90 GIWW-West Closure Complex for West Bank and Vicinity Hurricane and Storm Damage Risk Reduction System Project. Notice is hereby given that an independent technical review, that is appropriate to the level of risk and complexity inherent in the project, has been conducted as defined in the Quality Control Plan. During the independent technical review, compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This included review of: assumptions; methods, procedures, and material used in analyses; alternatives evaluated; the appropriateness of data used and level obtained; and reasonableness of the result, including whether the product meets the customer's needs consistent with law and existing Corps policy. The independent technical review was accomplished by an independent team. All comments resulting from ITR have been resolved.

Neil Schwanz
Geotechnical ITR Team Lead

10 Mar 09
Date

Patrick Foley
Hydrology & Hydraulics ITR Team Lead

Date

Kent Hokens
Structural ITR Team Lead

10 Mar 09
Date

CERTIFICATION OF INDEPENDENT TECHNICAL REVIEW

Significant concerns and the explanation of the resolution are as follows:

See attached for comments from ITR

As noted above, all concerns resulting from independent technical review of the project have been fully resolved.

Chief, Engineering Division

Date

COMPLETION OF INDEPENDENT TECHNICAL REVIEW
(ER-1110-1-12)

The District has completed the West Closure Complex Detention Stage and PS Capacity Selection Paper for WBV 90 GIWW-West Closure Complex for West Bank and Vicinity Hurricane and Storm Damage Risk Reduction System Project. Notice is hereby given that an independent technical review, that is appropriate to the level of risk and complexity inherent in the project, has been conducted as defined in the Quality Control Plan. During the independent technical review, compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This included review of: assumptions; methods, procedures, and material used in analyses: alternatives evaluated: the appropriateness of data used and level obtained; and reasonableness of the result, including whether the product meets the customer's needs consistent with law and existing Corps policy. The independent technical review was accomplished by an independent team. All comments resulting from ITR have been resolved.

Neil Shwanz
Geotechnical ITR Team Lead

Date

Patrick Foley
Hydrology & Hydraulics ITR Team Lead

10 March 09
Date

Kent Hokens
Structural ITR Team Lead

Date

CERTIFICATION OF INDEPENDENT TECHNICAL REVIEW

Significant concerns and the explanation of the resolution are as follows:

See attached for comments from ITR

As noted above, all concerns resulting from independent technical review of the project have been fully resolved.

Chief, Engineering Division

Date

Snapshot Report: Comment Submitters
Project: **West Closure Complex**
Review:**For the WCC Detention Stage and PS Capacity Selection ITR**
(sorted by Office, Last Name)

Engineering Control Branch											
Assigned Users (Last, First)	**Comments Authored**		**Evaluation**					**Backcheck**			
	Total	Withdrawn	Pending	Concur	Check	Info	Non-Concur	Pending	Closed	Open	
Ruppert, Timothy	0	0	0	0	0	0	0	0	0	0	
OFFICE TOTALS	0	0	0	0	0	0	0	0	0	0	
Geotechnical Section											
Assigned Users (Last, First)	**Comments Authored**		**Evaluation**					**Backcheck**			
	Total	Withdrawn	Pending	Concur	Check	Info	Non-Concur	Pending	Closed	Open	
Schwanz, Neil (view contributed)	4	0	0	4	0	0	0	0	4	0	
OFFICE TOTALS	4	0	0	4	0	0	0	0	4	0	
Hydraulics & Hydrology Branch											
Assigned Users (Last, First)	**Comments Authored**		**Evaluation**					**Backcheck**			
	Total	Withdrawn	Pending	Concur	Check	Info	Non-Concur	Pending	Closed	Open	
Foley, Patrick (view contributed)	11	0	0	5	0	0	6	0	11	0	
OFFICE TOTALS	11	0	0	5	0	0	6	0	11	0	
S-M-E-A Section											
Assigned Users (Last, First)	**Comments Authored**		**Evaluation**					**Backcheck**			
	Total	Withdrawn	Pending	Concur	Check	Info	Non-Concur	Pending	Closed	Open	
Hokens, Kent (view contributed)	3	0	0	1	2	0	0	0	3	0	
OFFICE TOTALS	3	0	0	1	2	0	0	0	3	0	
Grand Total:	18										

LEGEND

- Total = Withdrawn + Pending + Concur + Check + Info + Non-Concur
- Pending Backcheck = Total - Withdrawn - Closed - Open

NOTES

- Withdrawn = Comments withdrawn prior to evaluation (by someone other than the submitter).
- Comments deleted by the submitter prior to evaluation are not tracked.

Snapshot Report: Customers
Project: **West Closure Complex**
Review:**For the WCC Detention Stage and PS Capacity Selection ITR**
(sorted by Office, Last Name)

No customers have been assigned to this review.

Report Complete

Comment Report: All Comments
Project: West Closure Complex
Review: WCC Detention Stage and PS Capacity Selection ITR
Displaying 18 comments for the criteria specified in this report.

500 ms to run this page

Id	Discipline	DocType	Spec	Sheet	Detail
2358786	Structural	Other	n/a'	n/a	n/a

The requirements and justification for structure heights is very confusing. The required top of structure height is stated to be EL 8.5 in in Section 2.0 with 1 foot of structural superiority for fronting protection. Sections 7.1 and 7.2 confuse the issue as to what is included in these elevations. Is elevation 8.5 the required height of structures (without structural superiority) for the present or for the future condition? Section 7.1 seems to indicate that it is for the present condition, although it is seems to be written for levees only. Section 7.2 indicates that superiority is provided for items that are accounted for separately in the rest of the system in the calculation of the structure height for the future (2057) condition. For the surge protection flood walls, the required top of protection is established at the future (2057) elevation and includes height to account for regional subsidence and sea level rise to that date. Structural superiorty is added to this value for structures that are particularly difficult to construct such as pump station fronting protection and other structures requiring extensive dewatering. Structural superiority for the surge barriers is a contingency against future design changes that would require additional structure height for protection. These changes could be from a variety of sources such as reauthorization, changes in design assumptions, as well as subsidence. The requirements for top of barrier elevation and structural superiority in this document need to be clarifed.

Submitted By: Kent Hokens (651-290-5584). Submitted On: 07-Mar-09

Revised 07-Mar-09.

1-0 Evaluation **Check and Resolve**
The detention basin levees and floodwalls will be maintained at El. 8.5 for the life of the project (i.e. through 2057). Unlike the remainder of the system, the detention basin will not be exposed directly to storm surge, so the effects of sea-level rise and its associated uncertainties are not included in the selected design grade (see paragraph 5.7 for more on this issue) and as such no increase in elevation is required between 2007 and 2057 to account for the sea-level rise factor. As for subsidence, the entire area will subside equally on both sides of the detention levees, which means maintaining El. 8.5 will actually result in 0.25' of increased storage. Since paragraph 7.1 is not clear, the following statement will be added after the second sentence: "Due to the fact that these levees are not directly exposed to storm surge, subsidence and sea-level rise effects on the detention base levees and floodwalls are not factors in the selection of design grades. As such, no increase in elevation of the detention levees and floodwalls is required over the design life of the project. In other words the 2007 Elevation = 2057 Elevation = El. 8.5." Given that the calculation of wave runup and overtopping has resulted in higher required design grades for levees than floodwalls, the design grade of 8.5 was selected based strictly upon the detention basin levees, building in some conservatism for the floodwalls The superiority discussion in this document is centered on the objective of maintaining Elevation 8.5 for the design life of the difficult-to-construct structures in the detention basin and provides specific rationale justifying the selection of 1'-0" in lieu of the HSDRRS Design Guidelines 2'-0" recommendation.

Submitted By: Christopher Dunn (504-862-1799) Submitted On: 10-Mar-09

1-1 Backcheck Recommendation **Close Comment**
This answers the comment

Submitted By: Kent Hokens (651-290-5584) Submitted On: 10-Mar-09

Current Comment Status: **Comment Closed**

Id	Discipline	DocType	Spec	Sheet	Detail
2358796	Structural	Other	n/a'	n/a	n/a

Following up with my previous comment, the discussion of Design Grade in paragraph 7.1 is confusing and appears to result in a selection of design grade that is inconsistent with the rest of the system. This paragraph states that additional lifts will be required in the future to maintain a grade of 8.5 because of settlement. For the rest of the system, levees are being constucted to the required elevation plus overbuild for local settlement. Addiitonal lifts to provide height to account for regional subsidence and sea level rise is left for future work.

Submitted By: Kent Hokens (651-290-5584). Submitted On: 07-Mar-09

Revised 07-Mar-09.

1-0 Evaluation **Check and Resolve**
See response to 2358786.

Submitted By: Christopher Dunn (504-862-1799) Submitted On: 10-Mar-09

1-1 Backcheck Recommendation **Close Comment**
Comment is addressed

Submitted By: Kent Hokens (651-290-5584) Submitted On: 10-Mar-09

Current Comment Status: **Comment Closed**

Id	Discipline	DocType	Spec	Sheet	Detail
2358801	Structural	Other	n/a'	n/a	n/a

In Section 9.0, it's not clear why the HSDRRS design requirements were not simply referenced and clarificaitions or proposed deviations from them stated. As it is, the design criteria is a combination of a few specific items that are already covered in the HSDRRS and are incomplete for the entire basin design, and a laundry list of potential design codes and manuals that may or may not be complete. For instance, why is a bearing capacity value listed since it is covered by design manuals, dependent on load case, and very unlikely to be used since all structures will be pile founded? Also, the lateral earth pressure and dewatering paragraphs makes a general statement about common practices (rather than stating criteria) that seem out of place in this type of document. Also, identical references are listed in different places.

Submitted By: Kent Hokens (651-290-5584). Submitted On: 07-Mar-09

Revised 07-Mar-09.

1-0	Evaluation **Concurred** Will eliminate superfluous references and strictly reference HSDRRS Design Guidelines as an "umbrella" reference. Submitted By: Christopher Dunn (504-862-1799) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Comment is addressed Submitted By: Kent Hokens (651-290-5584) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359075	Geotechnical	Planning Report	n/a'	n/a	n/a

Coordinating Discipline(s): Geotechnical

Paragraph 3.0, Levees, Floodwalls, Closure Gates, and Fronting Protections, first bullet. Reference is made to risk. It seems intuitive that risk for this work is significantly less than that of the WCC since this is not a hurricane barrier. Was a risk assessment done?

Submitted By: Neil Schwanz (651-290-5653). Submitted On: 08-Mar-09

1-0	Evaluation **Concurred** A formal risk assessment has not been completed. Through the design process we have taken steps to reduce risk. This includes sizing the pump station and dredging of the canal to maintain the SWL Elevation. Also Operations manuals will dictate requirements to control the water level. Submitted By: Jennifer Vititoe (504-862-1252) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: Neil Schwanz (651-290-5653) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359076	Geotechnical	Planning Report	n/a'	n/a	n/a

Coordinating Discipline(s): Hydraulics,Hydraulics

Page 7, paragraph 5.2. Was analysis done without any restrictions on individual PS operations? Will an operations system requirements be needed?

Submitted By: Neil Schwanz (651-290-5653). Submitted On: 08-Mar-09

Revised 08-Mar-09.

1-0	Evaluation **Concurred** Analysis was performed considering all (9) locally owned pumps operating simultaneously and at full capacity. As mentioned in Section 5.5, per conversations with local pump operators it is known that this scenario does not occur. As necessary there will be coordination with local sponsors for pump operation. Submitted By: Jennifer Vititoe (504-862-1252) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: Neil Schwanz (651-290-5653) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359077	Geotechnical	Planning Report	n/a'	n/a	n/a

Coordinating Discipline(s): Hydraulics

Page 11, 5.9, second paragraph. In addition to DWSE and low water, the top of barrier elevation is also required.

Submitted By: Neil Schwanz (651-290-5653). Submitted On: 08-Mar-09

1-0	Evaluation **Concurred** It has been done and it will be done. Submitted By: Shung Chiu (504-862-1032) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: Neil Schwanz (651-290-5653) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359078	Geotechnical	Planning Report	n/a'	n/a	n/a

Coordinating Discipline(s): Geotechnical

Page 15, 6.5. Retention levee criteria was provided by HQ which is less restrictive than used in these analyses. Generally a design goal is to meet minimum criteria without significant overdesign. Why is revised criteria not being followed?

Submitted By: Neil Schwanz (651-290-5653). Submitted On: 08-Mar-09

1-0	Evaluation **Concurred** We follow the HQ provided criteria when the WCC gates are closed. However, we do evaluate the system with HSDRRS criteria when gates are opened at the present time. Submitted By: Shung Chiu (504-862-1032) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Closed without comment. Submitted By: Neil Schwanz (651-290-5653) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359096	Hydraulics	Planning Report	n/a'	n/a	n/a

Page 2, section 1.0. The proposed closure is off the navigation sailing line. Will this be a problem for navigation? Especially for the Algiers Canal.

Submitted By: Patrick Foley (651-290-5630). Submitted On: 08-Mar-09

Revised 08-Mar-09.

1-0	Evaluation **Non-concurred** The closure location shown in the report is not the final location and channel lines are now being developed. The navigation industry is being consulted throughout this project and has tested the alignment on the SHIP simulator model at ERDC. Submitted By: Heath Jones (504-862-2426) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Response adequate. No change in report needed. Submitted By: Patrick Foley (651-290-5630) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359101	Hydraulics	Planning Report	n/a'	n/a	n/a

Page 6, section 5.1. Why were interior flood control facilities larger than the minimum facilities not studied? EM 1110-2-1413 indicates minimum facilities are intended to be a starting point for the IFC analysis.and that for most projects they will be found inadequate due to high residual damages.. At a minimum, the report should show that residual damages with the recommended plan are minor.

Submitted By: Patrick Foley (651-290-5630). Submitted On: 08-Mar-09

Revised 08-Mar-09.

1-0	Evaluation **Non-concurred** This project has very little to no impact on residual damages unless the retention basin levees are overtopped. Residual damages due to a less frequent rainfall event would be due to the lack of capacity at the local interior pump stations and not a function of any of the project features for WCC. Submitted By: Heath Jones (504-862-2426) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Response adequate. No changes to report needed. Submitted By: Patrick Foley (651-290-5630) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359102	Hydraulics	Planning Report	n/a'	n/a	n/a

Page 6, section 5.1. It appears that the retention levees will have a higher probability of failure during an exterior 1% event than the main line levees. The coincident retention pond stage looks to be about 6.8 to 7.5 (see next comment). With wave action there could be signficant overtopping of the retention levees. The report should address the probability and consequences of the retention levees failing during the design event.,

Submitted By: Patrick Foley (651-290-5630). Submitted On: 08-Mar-09

Revised 08-Mar-09.

1-0	Evaluation **Non-concurred** The retention levees are no more likely to fail during a 1% exterior surge event because the surge will not be allowed to propogate into the retention basin because of the main line levees and gate. The probability and consequences of failure will be addressed in the PDD, but it out of the scope of this document Submitted By: Heath Jones (504-862-2426) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Response adequate. No change to report needed. Submitted By: Patrick Foley (651-290-5630) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359103	Hydraulics	Planning Report	n/a'	n/a	n/a

Page 6-7, Section 5.1. The last two paragraphs state two methods were used to calculate the retention pond level for the 1% chance precipitation, but the report isn't clear what the results were. Page 10 shows a 1% retention pond level of 7.3, page 15 shows 7.5, and appendix B seems to show 6.8.

Submitted By: Patrick Foley (651-290-5630). Submitted On: 08-Mar-09

Revised 08-Mar-09.

1-0	Evaluation **Concurred** The value should be 7.8 ft and will be changed in the report Submitted By: Heath Jones (504-862-2426) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Response adequate. Revised report checked and has been changed. Submitted By: Patrick Foley (651-290-5630) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359104	Hydraulics	Planning Report	n/a'	n/a	n/a

Page 7, section 5.2. The last sentence states the New Orleans Sewerage and Water Board Pump Station No. 13 might be increased by 2,000 cfs to a total capacity of 2,990 cfs, but the table shows the existing capacity for S&WB # 13 is 4,650 cfs.

Submitted By: Patrick Foley (651-290-5630). Submitted On: 08-Mar-09

1-0	Evaluation **Concurred** The value should be 6,650. Will change in report. Submitted By: Heath Jones (504-862-2426) Submitted On: 10-Mar-09

1-1	Backcheck Recommendation **Close Comment** Response adequate. Revised report checked and change has been made. Submitted By: Patrick Foley (651-290-5630) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359105	Hydraulics	Planning Report	n/a'	n/a	n/a

Page 8, section 5.3. The slope of the DWSE along the canals should be given. Appendix B shows the maximum water elevation for the Harvey Canal is only 3.2 vs. the maximum of 5.3 for the Algiers Canal. It appears the 5.8 DWSE based on the 5.3 water level was used for all locations along both canals. Why wasn't the sloping water level used?

Submitted By: Patrick Foley (651-290-5630). Submitted On: 08-Mar-09

1-0	Evaluation **Non-concurred** The PDT decided very early on (to be conservative)that we would not use the slope of the water surface and design the retention basin protection for that slope. If some or all of the pumps at the 20,000cfs pump station became inoperable during an event the retention basin would essentially become a level pool, so we treated the design in this manner Submitted By: Heath Jones (504-862-2426) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Response adequate. No change in report needed. Submitted By: Patrick Foley (651-290-5630) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359106	Hydraulics	Planning Report	n/a'	n/a	n/a

Page 9, section 5.5. The use of 0.5 feet for uncertainty seems like a reasonable value but should be justified. Was the HMS model run with lower loss coefficients run to estimate the possible increase in inflow and was the RAS model run with higher friction factors?

Submitted By: Patrick Foley (651-290-5630). Submitted On: 08-Mar-09

Revised 08-Mar-09.

1-0	Evaluation **Concurred** MVS ran a sensitivity analysis of the manning's n values in the HEC-RAS model and it was determined that the standard deviation originally assumed was appropriate and the 90% level of assurance SWE was slightly below 5.8'. Submitted By: Stacey Frost ((504)862-2993) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Response adequate. No change in report needed. Submitted By: Patrick Foley (651-290-5630) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359107	Hydraulics	Planning Report	n/a'	n/a	n/a

Page 9, section 5.6. The report should state what the computed overtopping is, not just that it meets the maximum allowable requirements.

Submitted By: Patrick Foley (651-290-5630). Submitted On: 08-Mar-09

1-0	Evaluation **Concurred** We will add to the report. Submitted By: Heath Jones (504-862-2426) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Response adequate. Submitted By: Patrick Foley (651-290-5630) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359108	Hydraulics	Planning Report	n/a'	n/a	n/a

Page 11, section 5.11. It is stated that based on a separate study maintenance dredging should not be required for several decades. Did this separate study include the proposed dredging of the Algiers Canal? If not, is more frequent dredging expected? Submitted By: Patrick Foley (651-290-5630). Submitted On: 08-Mar-09	
1-0	Evaluation **Non-concurred** Yes, it included the proposed dredging. And the conclusion was "The conclusion of the analysis is that, while a small increase in shoaling is expected to occur, the dredging requirements for the deepened channel will remain as they are presently." Submitted By: Heath Jones (504-862-2426) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Response adequate. No change in report needed. Submitted By: Patrick Foley (651-290-5630) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359109	Hydraulics	Planning Report	n/a'	n/a	n/a

Page 15, section 7.0. It is stated that the design rainfall roughly equals that from a 1% exceedance tropical event from JSM-OS. The design rainfall is not given in the report but is shown on TP 40 to be about 9.0 inches. Section 5.8 on page 10 states the 1% chance rainfall is 13.2 inches in 24 hours. These are not roughly equal and the resultant retention pond levels of 5.3 vs. 6.8 to 7.5 are also not roughly equal. The proposed levees at 8.5 will contain the stillwater 1% event but there could be significant overtopping from waves and possible backflow. Submitted By: Patrick Foley (651-290-5630). Submitted On: 08-Mar-09 Revised 08-Mar-09.	
1-0	Evaluation **Non-concurred** The design rainfall for the 10% non tropical event is 9.0 inches which is equal to the 1% tropical event rainfall. The runs which resulted in the SWL of 6.8 are for the 1% non tropical rainfall event which is 13.2 inches. It is acknowledged that overtopping from waves will occur for the higher rainfall total, but there is a small chance that the 1% non tropical rainfall event will occur while the gates are closed. Backflow suppression is being designed for the 10% event as well. It is acknowledged that backfow will occur for the 1% non tropical rain event, but again this is an unlikely event to have. Rainfall totals have been added to report. Submitted By: Heath Jones (504-862-2426) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Response adequate. The addition to the report of the rainfall amounts for the 10% non tropical event and the 1% tropical event clear up this concern. The revised report was checked and these rainfall amounts have been added. Submitted By: Patrick Foley (651-290-5630) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

2359111	Hydraulics	Planning Report	n/a'	n/a	n/a

Appendix B table shows alternatives that are not mentioned in the text. The text should explain what these alternatives are and why they were not selected ot they should be removed from Appendix B or a note added to the table explaining why they are shown, Submitted By: Patrick Foley (651-290-5630). Submitted On: 08-Mar-09 Revised 08-Mar-09.	
1-0	Evaluation **Concurred** Appendix B will be modified. It is out of date. Submitted By: Heath Jones (504-862-2426) Submitted On: 10-Mar-09
1-1	Backcheck Recommendation **Close Comment** Response adequate. Submitted By: Patrick Foley (651-290-5630) Submitted On: 10-Mar-09
	Current Comment Status: **Comment Closed**

Kirkeeng, Thomas A MVR

From: Powell, Nancy J MVN
Sent: Sunday, August 23, 2009 10:38 AM
To: Kirkeeng, Thomas A MVR
Cc: Lester, Barbara L MVR; Frost, Stacey U MVN
Subject: RE: Algiers Design Elevation

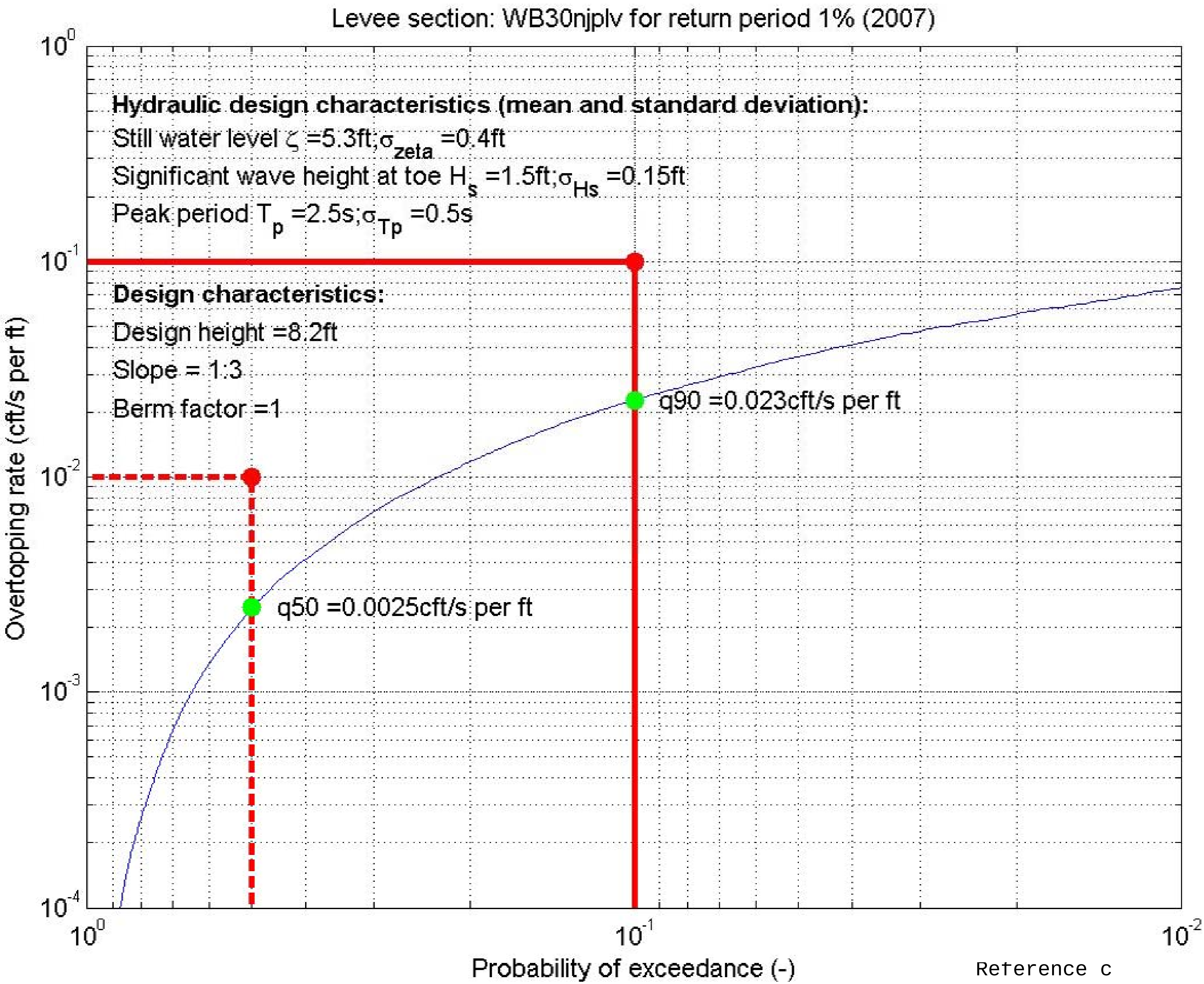
At the request of MVN Geotech Branch, MVN H&H Branch re-examined the hydraulic design of the WCC levees and floodwalls at an elevation of 8.2 ft NAVD88 2004.65 in lieu of 8.5 ft NAVD88 2004.65. Using a design elevation of 8.2 ft, with a slope of 1:3 for the levees, both a levee and floodwall at elevation 8.2 meet the overtopping criteria for existing conditions. As this is a detention basin, where relative sea level rise is not an issue, the elevation is also valid for future conditions.

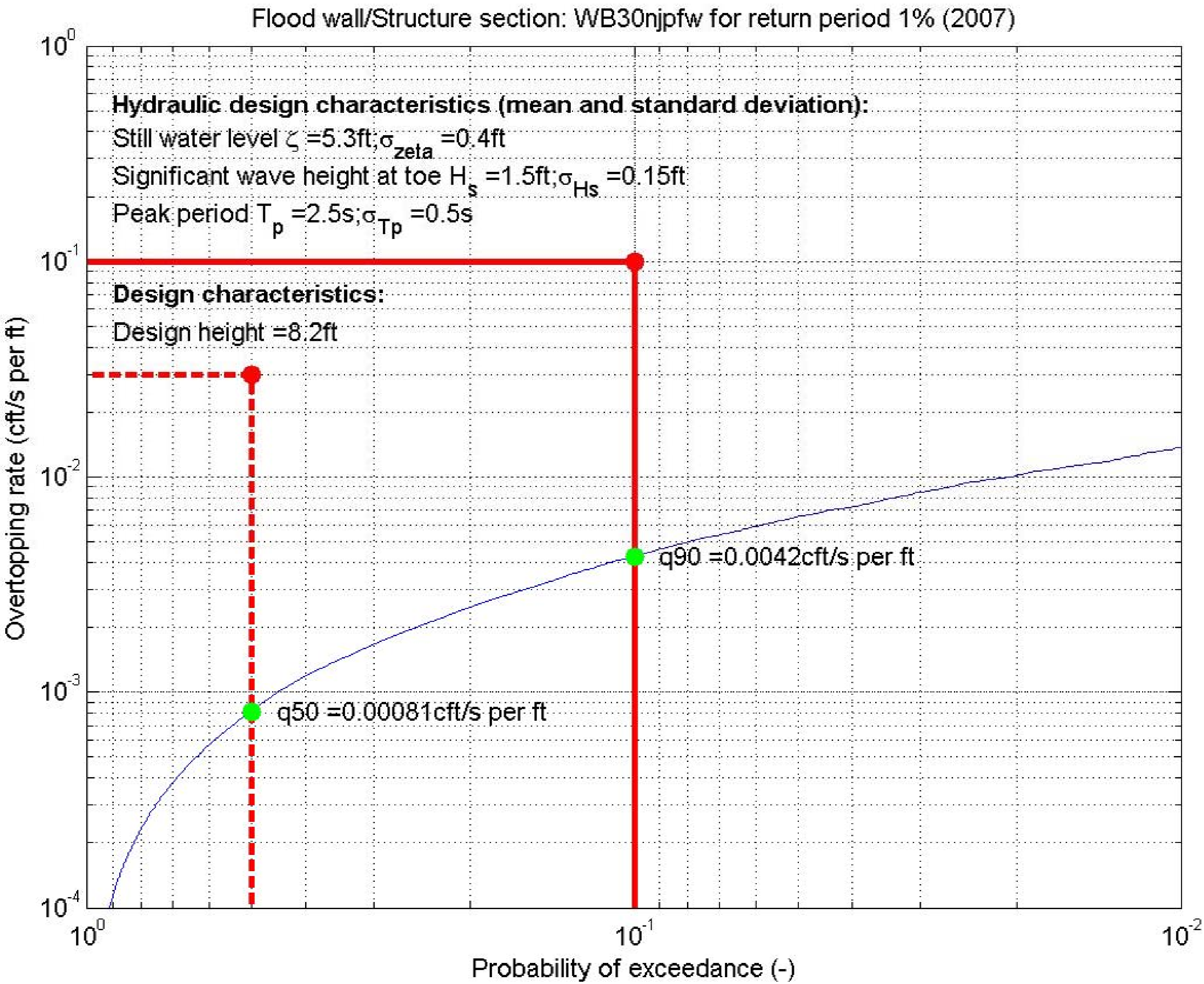
If you have any questions, please let me know.

Nancy J. Powell, P.E., D.WRE
Chief, Hydraulics and Hydrologic Branch
U.S. Army Corps of Engineers, New Orleans District
CEMVN-ED-H
PO Box 60267
New Orleans, LA 70160
Phone - (504) 862-2449
Fax - (504) 862-2471
email - nancy.j.powell@usace.army.mil

-----Original Message-----

Reference b


Levee section: WB30njplv for return period 1% (2007)
Hydraulic design characteristics (mean and standard deviation):
Still water level ζ =5.3ft;σzeta =0.4ft
Significant wave height at toe Hs =1.5ft;σHs =0.15ft
Peak period Tp =2.5s;σTp =0.5s
Design characteristics:
Design height =8.2ft
Slope = 1:3
Berm factor =1
q90 =0.023cft/s per ft
q50 =0.0025cft/s per ft
Overtopping rate (cft/s per ft)
Probability of exceedance (-)
10^0
10^-1
10^-2
10^-3
10^-4
Reference c



Flood wall/Structure section: WB30njpfw for return period 1% (2007)
Hydraulic design characteristics (mean and standard deviation):
Still water level ζ =5.3ft;σzeta =0.4ft
Significant wave height at toe Hs =1.5ft;σHs =0.15ft
Peak period Tp =2.5s;σTp =0.5s
Design characteristics:
Design height =8.2ft
q90 =0.0042cft/s per ft
q50 =0.00081cft/s per ft
Overtopping rate (cft/s per ft)
Probability of exceedance (-)

Appendix N

Wind Speed for 100- Year and 500- Year Event

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APPENDIX N – WIND SPEED FOR 100-YEAR AND 500-YEAR EVENT

For design purposes, the wave characteristics along the levees and floodwalls have to be known. A nearshore wave model (STWAVE) has been used for almost the entire system to estimate the wave characteristics. However, the model grid from STWAVE is too coarse to represent the waves in the canals, e.g. in the IHNC or Harvey Canal. In these regions, the empirical method from Brettschneider has been applied *Shore Protection Manual* (SPM, 1984).

The determination of the design wave height in the canals will depend upon the determination of the design wind speed. Estimating the 100-year wind speed will be paramount to determining the 100-year wave height. The method for estimating hurricane wind speeds for given return periods is presented in Coastal Engineering Technical Note (CETN) I-36 dated December 1985. This provides an estimate of the fastest-mile hurricane wind speed at 10 meters above ground over open terrain along the coast. This fastest mile wind speed is then converted to a duration of one hour, utilizing the method presented in the Corps of Engineers' (SPM, 1984).

The design wind speed was taken from CETN-I-36, *Estimates of Hurricane Winds for the East and Gulf Coasts of the United* States. The following are excerpts from that document.

> Extreme hurricane wind speeds cannot be predicted by extrapolating annual wind speed distributions. Batts, et. al. estimated hurricane winds indirectly from statistical distributions of hurricane climatologically characteristics and a mathematical model of the hurricane wind field. The model takes into account the position of the storm center relative to the point of interest, storm decay, wind speed reduction over land due o friction, and the effects of time averaging. The model gives the recurrence interval wind speeds as fastest-mile at 10 meters above ground over open terrain at the coastline and 124 miles inland. The model assumes a straight shoreline and a constant overland surface roughness.

Referring to Figure 1 of CETN-I-36, Station 650 was selected as representative of the study area. For different return periods, the estimated fastest mile wind speed at the coast are listed below:

Return Period (years)	At the Coast	At 200 km Inland
10	61	61
25	80	80
50	91	91
100	100	100
2000	130	130

Table L-1: Estimated Fastest Mile Wind Speed for Location 650 (Source: CETN-I-36)

For a return period of 100-years, the estimated fastest mile wind speed at the coast is 100 mph. At a distance of 124 miles inland, the estimated wind speed remains at 100 mph. This is due to the lack of ground obstruction to the wind. For the design purposes, the wind speed with a return period of 500-years must also be known (resiliency analysis). The wind speed with a return period of 500-year has been obtained by interpolation of the data in Table 1 resulting in 116 mph.

The fastest mile wind speed must now be converted to a time dependant average wind speed, preferably in hourly durations. The method to do this is outlined the SPM ,1984, pages 3-26 to 3-30.

- Fastest Mile Wind Speed during 100-year event = 100 mph
- Find: 1-Hour average wind speed
- Time to Travel 1-mile: t = (60 min/hr)(60 sec/min)/100 = 3600/100 = 36 sec
- Conversion Factor: $1.277 + 0.296 \tan h\ (0.9 \log_{10} 45/t) = 1.30$
- 1-Hour Average Wind Speed: 100/1.3 = 77 mph

Analogously, the 1-hour average wind speed during a 500-year event equals 88 mph.

Appendix O

Sea Level Rise- Impact Assessment of New Relative Sea Level Change Engineer Circular for Flood Risk Reduction Projects in the New Orleans Area (17 Feb 2011, Authors: S. Ayers, M.V. Ledden, M. Agnew)

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Executive Summary

The objective of this document is to perform an assessment of the hydraulic impact of relative sea level change (RSLC) on the Hurricane Storm Damage Risk Reduction System (HSDRRS) flood risk reduction projects in the New Orleans area following the new guidance provided in Engineering Circular (EC) 1165-2-211. This report addresses how RSLC has been included in flood risk reduction projects prior to this EC. It also presents the RSLC scenario according to the EC and assesses the impact of these scenarios for the current HSDRRS design. The entire analysis presented in this document has been based on limited field and ADCIRC modeling data. Moreover, the assessment has been carried out for generic sections of the various HSDRRS.

The effect of RSLC was treated differently in the various flood risk reduction projects prior to this EC. With respect to the HSDRRS program, RSLC has been based on the LCA study (2004) and was set at +1.0 ft in 2057. Following the new guidance from this EC, three different RSLC scenarios are presented in this report for the three areas in the HSDRRS; Lake Pontchartrain, West Bank, and Lake Borgne. These RSLC scenarios are based on; 1) a linear extrapolation of the historical rate, 2) a moderate acceleration of RSLC (NRC-1)*should this be NRC I, and 3) a severe acceleration of RSLC (NRC-3). It is concluded that the RSLC scenario from the HSDRRS design is comparable with the obtained historical rate at the West Bank (+1.0 ft in 2060) according to the new EC. The RSLC at Lake Pontchartrain (+1.3 ft in 2057) and Lake Borgne (+1.5 ft in 2057) is higher than the originally assumed scenario in the HSDRRS design.

The effect of RSLC on the 1% design surge level has been assessed using the existing information of in total 41 ADCIRC runs. A so-called amplification factor (defined as the 1% surge level increase/sea level change increase) has been determined using these ADCIRC runs. The obtained amplification factors are lower than originally applied in the HSDRRS design. Using these new amplification factors, the 1% design surge level changes have been determined based on the RSLC scenarios. It was found that the 1% surge level increase based on the historical RSLC rate is higher for the Lake Borgne and Lake Pontchartrain area (1.6 and 1.8 ft in 2057 respectively) compared with the original HSDRRS design (1.5 ft in 2057 for both areas). For the West Bank, the increase is lower for the historical RSLC scenario (1.6 ft versus 2.0 ft in 2057). The 1% surge level increase is higher for the accelerated RSLC scenarios (2.0 – 4.0 ft in 2057) compared with the original HSDRRS numbers.

The assessment of the HSDRRS program shows that the current HSDRRS hydraulic design for the levees and floodwalls will be sufficient for 50 years for Lake Pontchartrain and Lake Borgne or even beyond 2060 for the West Bank if the historical RSLC rate from the EC is considered. The hydraulic lifetime reduces to 35 years for the worst case RSLC scenario. All current 2060 design elevations for the levees and floodwalls are above the 1% surge level + 2.0 ft freeboard, with most conservative RSLC scenarios. That implies that - apart from just raising the system - various other options are also possible to cope with more severe RSLC scenarios. Various generic hydraulic solutions are discussed. It is recommended that a more in-depth multi-disciplinary analysis should be carried out to assess the options to cope with more severe RSLC scenarios on a reach-by-reach basis in the entire HSDRRS.

1.0 INTRODUCTION

USACE issued a new policy, in July 2009, to incorporate sea level change considerations in civil works programs (EC1165-2-211). The purpose of this new EC is stated as follows:

"This circular provides United States Army Corps of Engineers (USACE) guidance for incorporating the direct and indirect physical effects of projected future sea-level change in managing, planning, engineering, designing, constructing, operating, and maintaining USACE projects and systems of projects. Recent climate research by the Intergovernmental Panel on Climate Change (IPCC) predicts continued or accelerated global warming for the 21st Century and possibly beyond, which will cause a continued or accelerated rise in global mean sea-level. Impacts to coastal and estuarine zones caused by sea-level change must be considered in all phases of Civil Works programs."

The EC requires that these future sea level change projections are incorporated into every USACE coastal activity including; planning, engineering design, construction, and operating Projects as far inland as the extent of estimated tidal influence. To that end, the following aspects shall be considered in planning, engineering, and design studies:

a) How sensitive are the natural and human systems to climate change?
b) What alternatives are available for the entire range of possible future RSLC?
c) How sensitive are alternative plans and designs to the different RSLC scenarios, how it affects risk, and what design and operation and maintenance measures shall be implemented to minimize adverse impacts while maximizing beneficial effects?

The EC prescribes three different RSLC scenarios which shall be considered in this evaluation; low, intermediate, and high. The low scenario is determined based on observed historical rates in the area of interest. The intermediate and high scenarios should be based on the National Resource Council (NRC) Curves I and III, respectively with accelerated RSLC scenarios.

For a long time, flood risk reduction projects have been carried out in Southern Louisiana and around New Orleans in particular because of its flood prone nature. Since Hurricane Katrina in 2005, these efforts have been intensified and various programs are in place to study and design flood risk reduction measures in the area of interest. Examples are:

- Louisiana Coastal Protection and Restoration Project (LACPR) date
- Hurricane Storm Damage Risk Reduction System (HSDRRS) 2010
- Morganza to the Gulf date
- Larose to Golden Meadow date
- Atchafalaya Flow Line 1986 or 2010

Note that some of these studies are feasibility studies (e.g. LACPR), whereas others are detailed design and construction programs (e.g. HSDRRS). The various projects and programs have considered RSLC but in each study the approach has varied.

The objective of this document is to assess the impact of RSLC on these flood risk reduction projects in the New Orleans area based on the new guidance provided in EC. Questions which will be addressed are:

- How was the effect of RSLC treated in the various projects prior to this EC?
- What are the RSLC scenarios for the New Orleans area based on the new EC?
- What is the impact of these new EC scenarios on the design surge levels and design elevations?
- How sensitive are the plans for different RSLC scenarios and what alternatives are available?

This report will limit itself to the HSDRRS project in describing the sensitivity for the different RSLC scenarios and the available alternatives.

RSLC, also known as apparent subsidence, is defined as the relative lowering of the land surface with respect to sea level. There are five major factors that contribute to RSLC as depicted in the diagram and listed below.

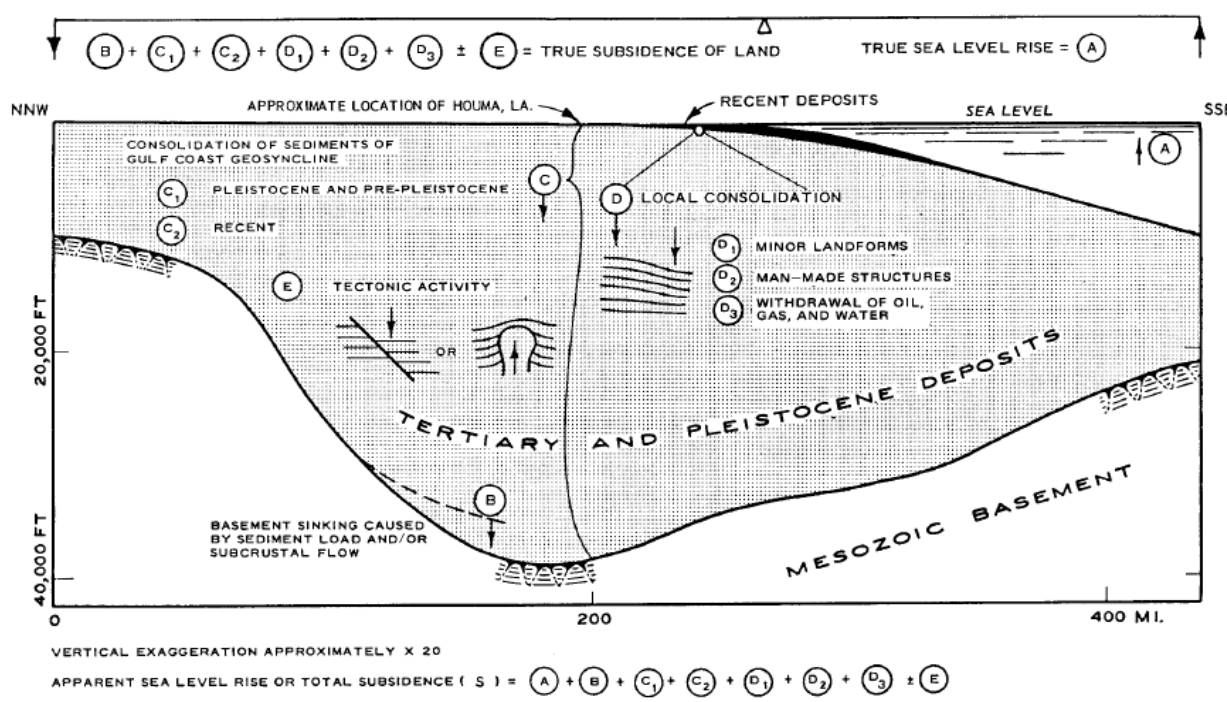
(A) Actual sea level rise

(B) Basement sinking caused by sediment load

(C) Consolidation of sediments of the Gulf coast geosynclines

(D) Local consolidation

(E) Tectonic activity.



Generalized cross section of Gulf coast geosynclines depicting components of apparent sea level rise (adapted from Kolb and Van Lopik 1958)

The outline of this document is as follows:

- **Chapter 2.0** Past Actions – Discusses how RSLC was incorporated into various studies prior to the development of this EC.
- **Chapter 3.0** Data and Data Analysis– Discusses the detailed determination of RSLC scenarios based on the most current EC (July 2009) within the New Orleans area.
- **Chapter 4.0** ADCIRC Model Results Analysis – Analyzes the impact of the different RSLC scenarios for the design surge levels (and the associated design elevations).
- **Chapter 5.0** HSDRRS Design Assessment – Discusses the sensitivity of the different RSLC scenarios for the HSDRRS program.
- **Chapter 6.0** Conclusions and Path Forward

2.0 PAST ACTIONS

2.1 MORGANZA TO GULF

RSLC was considered in the April 1994 Morganza to Gulf Reconnaissance Study, and the March 2002 Morganza to Gulf Feasibility Report. RSLC rates were determined through analysis of long term water surface elevation records.

For the New Orleans District Atchafalaya River Delta Study in the 1980s, ERDC conducted a literature review of the studies performed along the Gulf Coast and the Atchafalaya River Basin relating to RSLC. In these studies, RSLC rates varied significantly, ranging from 0.85 cm/year for a study by Baumann and Adams in the Amelia area to 1.62 cm/year for a study by Penland et. al., for the Atchafalaya coastal area.

ERDC also performed a statistical analysis of the long term water surface elevation records from the gages in the Atchafalaya River Basin and Morganza to Gulf areas. Based on multiple regression analyses on single and multiple stations, RSLC estimates were developed for existing (1980) and future conditions (2030) for the Atchafalaya Bay and a portion of the Morganza to the Gulf study area below the Bayou Black Ridge. The ERDC analysis showed a spatial and temporal variation in RSLC rates, where rates for the period 1962-2030 varied from 0.7 cm/year in the Houma area to 1.4 cm/year at the mouth of the Lower Atchafalaya River.

In a separate study, the New Orleans District analyzed water surface elevation data in the Lake Verret portion of the Morganza to the Gulf Study area, as well as data for the Gulf of Mexico at Biloxi, Intracoastal Waterway at Houma, and the Lower Atchafalaya River at Morgan City, to determine RSLC rates in the Lake Verret area. Some of the stations experienced noticeable subsidence during the period of record, and subsequently periodic datum corrections were incorporated back into the data so that RSLC estimates would not be erroneously biased low.

Next, the 50% exceedence water surface elevations for the months July thorough October were computed for each year of the gages' period of record. The months July through October were chosen as generally few flood events occur during this time of year. Plots of the 50% exceedence water surface elevation vs. time showed a gradual increase in elevation over time; the rate of change varied from 0.1 to 1.0 cm/year depending on location and time. Future subsidence for the area above Bayou Sorrel was estimated by Hydraulics and Hydrology Branch and Geotechnical Branch personnel. In 50 years, the area between Bayou Sorrel and Interstate 10 was expected to experience 0.8 ft of RSLC. Above Interstate 10, RSLC was expected to be 0.4 ft in 50 years. For rainfall modeling, the models were adjusted by lowering cross section and storage area elevations by the computed amount. For hurricane surge, RSLC was accounted for by lowering structure elevations in the economic analysis.

For the feasibility study, significant differences in surge heights for existing, base, and future conditions did not materialize in the results of the ADCIRC runs for these conditions. RSLC of the coastal area in the future did not produce increases in surge heights along the coastal reaches.

Because storm setup, a portion of the storm surge due to winds, is inversely proportional to the depth, ADCIRC results indicated that the storm surge would be slightly lower at the model boundaries in the future. Because of the consequences associated with accepting this premise, lower stages were not used for future conditions. Thus the same stages at the boundaries were used to drive the model and design levee heights for both base and future conditions.

2.2 ATCHAFALAYA FLOW LINE

The flow line for the Mississippi River & Tributaries (MR&T) Project Design Flood (PDF) in the Atchafalaya Basin Floodway System (ABFS) is currently being re-analyzed and the study is projected to be is this the Flow Line Study complete in 2010 . As part of this study, the future conditions of the ABFS is approximated in the geometry of a 1-dimensional hydraulic model and a 2-dimensional finite element model and the PDF is analyzed with these future geometry conditions. The 1-dimensional model domain covers the area from the Gulf Intracoastal Waterway (GIWW) to the Red River, while the 2-dimensional model covers the domain from the Wax Lake Outlet flow split near Myette Point downstream to the Gulf of Mexico including the Atchafalaya Bay.

Subsidence throughout the study domain was estimated in a separate geomorphic analysis. The future model geometry of the basin incorporated the findings of the geomorphic analysis through modification of the geometry over existing conditions geometry in the 1-dimensional and 2-dimensional models.

The currently approved flow line (1986) used a downstream tailwater stage of 5.0 ft NGVD at River Mile (RM) 152 near Eugene Island for all model analyses. In order to project year 2059 tailwater stages in the Atchafalaya Bay in accordance with EC 1165-2-211 guidelines, the following method was employed. The 5.0 ft NGVD stage that was used in the 1987 flow line report was projected forward to 2009 using the historical global eustatic sea level change rate of 1.7 millimeter (mm)/year. Then starting in year 2010, the three eustatic sea level scenarios were used to project future gulf stages in 2059. These three stages are then used as the Gulf boundary stage in the 2-dimensional model for future PDF simulations.

The conversion from NGVD to NAVD88 is a subtraction of 0.16 ft, according to Corpscon version 6.0.1 using Vertcon94 data files. Refer to **Table 2-1** and **Figure 2-1** for projected future tailwater stages at RM 152.

Table 2-1 Future Projected Tailwater Stages for ABFS Flow Line Study

EC Sea Level Change Scenario	2059 Stage at RM 152 (ft NGVD)	2059 Stage at RM 152 (ft NAVD88)
Low	5.4	5.2
Intermediate	5.8	5.6
High	7.0	6.8

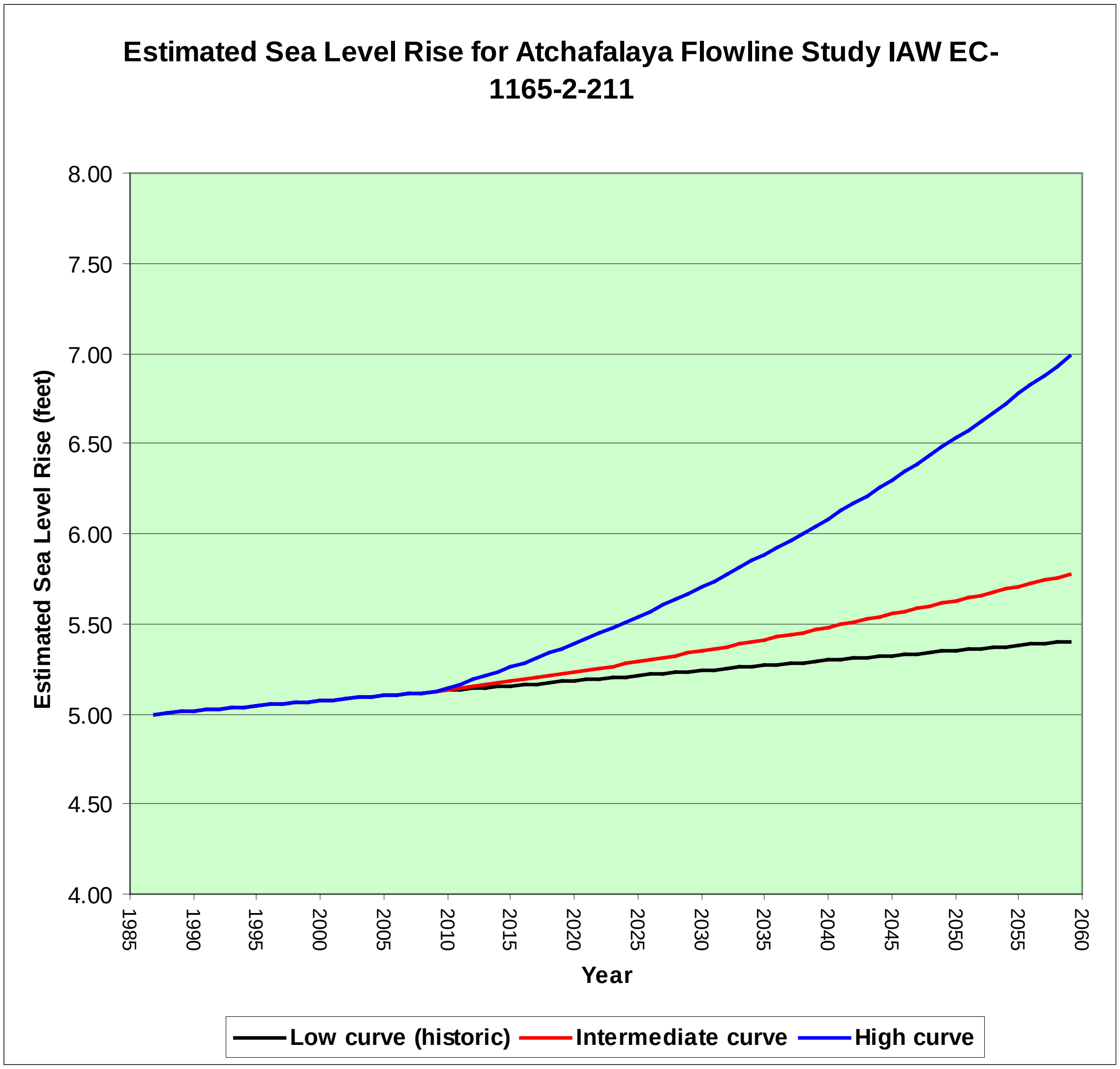


Figure 2-1 Future Projected Tailwater Stages for ABFS Flow Line Study

2.3 HURRICANE STORM DAMAGE RISK REDUCTION SYSTEM

In the design report for the HSDRRS, the future scenario was considered to reflect conditions that were likely to exist in the year 2057. Historical subsidence, projections of RSLC, and previous studies were used to estimate future changes in surge elevations. A RSLC of 1.0 ft over 50 years was used in the design analysis to represent future conditions in the entire area.

The 2011 HSDRRS Design Elevation Report references several ADCIRC and STWAVE model runs that were performed to investigate the effect of the increasing RSLC on surge levels and wave characteristics. These original LACPR ADCIRC runs are discussed in Section 2.8 and Chapter 4 on this report. The 2011 HSDRRS Design Elevation Report summarizes the findings as follows:

- Surge levels increase more than proportional to increasing RSLC (factor 1.5 to 2.0)
- The wave heights increase due to RSLC. The relative effect on the wave heights is about 0.3 to 0.6, which means that 1.0 ft surge level results in 0.3 to 0.6 ft increment of wave height.
- The effects are not uniform in the entire area and depend on the local water depth and geometry of the area of interest.

The future conditions for the HSDRRS were based on interpretation of the ADCIRC and STWAVE model results. The future conditions were used as boundary conditions in the design of floodwalls, levees, and other structures. **Table 2-2** summarizes the future conditions used for the design of the HSDRRS.

Table 2-2 Summary of HSDRRS Future Conditions for Surge Level and Wave Characteristics

Future Conditions	**Surge Level (hsurge)**		**Significant Wave Height (Hs)**		**Peak Period (Tp)**
	Δhsurge / Δhsealevel (-)	**Δhsurge (ft)**	**ΔHs / Δhsurge (-)**	**ΔHs (ft)**	**ΔTp (sec)**
Lake Pontchartrain, New Orleans East, IHNC, and GIWW, St. Bernard	1.5	+ 1.5 ft	0.5	+ 0.75 ft	Increase by unchanged wave steepness (H/T^2)
Caernarvon and West Bank	2.0	+ 2.0 ft	0.5	+ 1.0 ft	Increase by unchanged wave steepness (H/T^2)

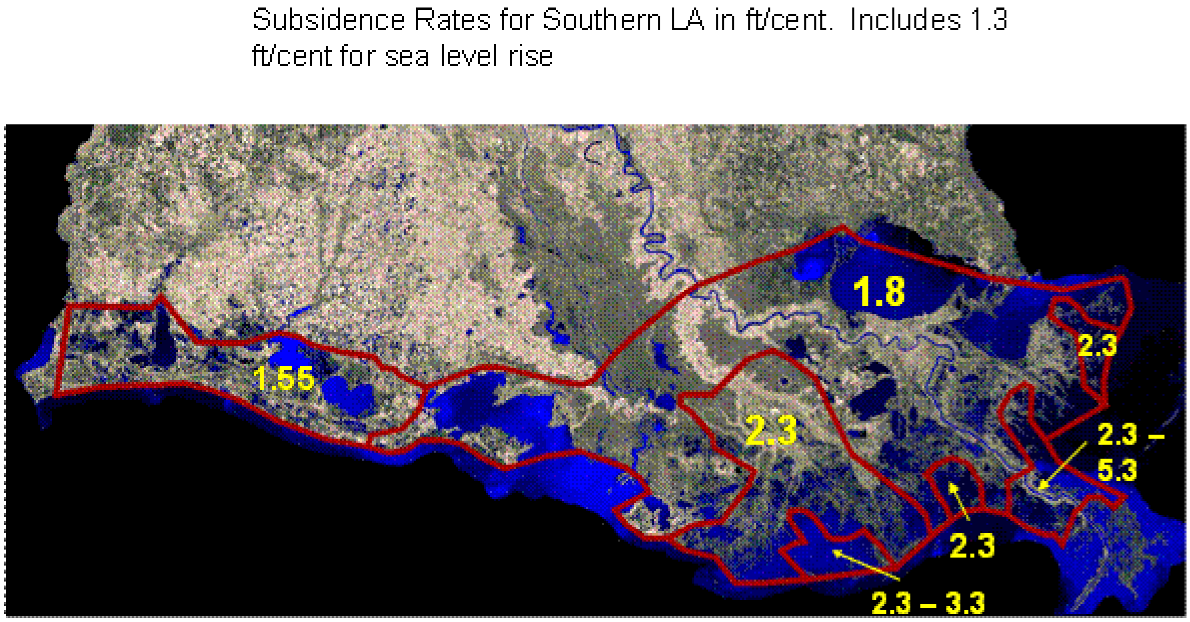


Figure 2-2 Estimated Relative Sea Level Rise During 100 Year (Subsidence + Eustatic Sea Level Rise)

2.4 LOUISIANA COASTAL PROTECTION RESTORATION

The LACPR report included an evaluation of hydraulic performance for the 2060 future conditions and accounted for RSLC, subsidence, and changes to the foreshore (marshes). These factors are visualized schematically in **Figure 2-2**.

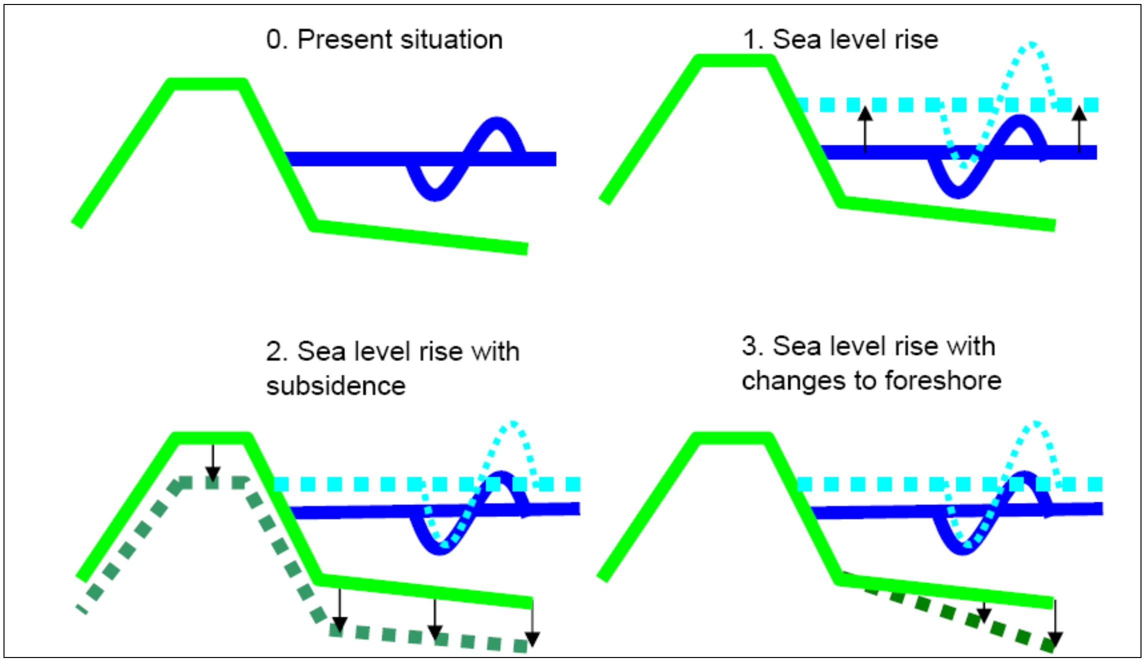


Figure 2-3 Future Factors in the Framework of LACPR

These factors potentially result in higher surges and wave heights, and also affect the levee heights that are required to provide a specific level of risk reduction. In the framework of LACPR, RSLC and subsidence has been allowed for in a combined value added to the surge levels. Three scenarios have been evaluated for the RSLC (sea level change + subsidence); 1) no RSLC, 2) a mid range RSLC value, and 3) a high RSLC value **(Table 2-3)**. Future rates of RSLR were determined by considering both the 1987 NRC*what is this global mean sea level change projections and the 2007 IPCC global mean sea level change projections, along with estimates for local and regional subsidence rates across coastal Louisiana.

Table 2-3 LACPR Relative Sea Level Change Values

Planning Unit	**No RSLC (ft)**	**Mid Range RSLC (ft)**	**High Range RSLC (ft)**
1 – Pontchartrain Basin	+ 0	+ 1.3	+ 2.6
2 – Barataria Basin	+ 0	+ 1.9	+ 3.2
3a – Terrebonne	+ 0	+ 1.9	+ 3.2
3b – Teche/Vermilion	+ 0	+ 1.9	+ 3.2
4 – Mermentau	+ 0	+ 1.3	+ 2.6

For LACPR, two future developments of the foreshore conditions were evaluated. The first is the “maintain coast” condition which was represented by the existing (2010) bathymetry assuming that the coastline will be maintained. The surge levels and waves do not change in this foreshore condition. The second is the “degraded coastal features” condition which has been represented by the bathymetry computed by the CLEAR model. Based on these runs, the effect of a degraded foreshore on the surge levels and waves were quantified. Given three values for RSLC and two future coastlines, LACPR evaluated six possible alternatives for the future scenarios with a range of results. **Table 2–4** summarizes how the future scenarios were incorporated in the levee heights and exterior stages.

Table 2-4 LACPR – Summary of Relative Sea Level Change Alternatives

Future Coastline Scenario	Future RSLC Scenarios	Levee Heights	Exterior Stages
Maintain Coastline	RSLC 0	Present Situation + effect of RSLC scenario	Present Situation + effect of RSLC scenario
	RSLC Mid		
	RSLC High		
Degraded Coastline	RSLC 0	Present Situation + effect of RSLC scenario + effect of degraded coastline	Present Situation + effect of RSLC scenario + effect of degraded coastline
	RSLC Mid		
	RSLC High		

In all future scenarios, the values for RSLC were added linearly to the levee heights and exterior stages.

LACPR also performed a sensitivity analysis on the effects of RSLC. The RSLC analysis consisted of 27 ADCIRC storm simulations. Nine storms were selected from the 2010 simulations and each was run with 1, 2, and 3 ft increase in water level. The purpose of the analysis was to estimate the impact of RSLC on surge and waves for the design of the flood defenses. In summary, these model runs found a non-linear response to surge amplification. More discussion of these runs and a new interpretation of the results are provided in **Chapter 4.0** of this report.

3.0 DATA AND DATA ANALYSIS

3.1 SUMMARY OF EC 1165-2-211 IMPLEMENTATION AT MVN

EC1165-2-211, Water Resource Policies and Authorities Incorporating Sea-Level Change Considerations in Civil Works Programs, effective 1 July 2009 and expiring on 1 July 2011, requires project performance to be assessed using three RSLC scenarios; 1) a low estimate, 2) an intermediate estimate, and 3) a high estimate. The low estimate is based on a linear projection of the historical rate for the study area. The intermediate estimate is based on the modified NRC Curve I and the local historical vertical shift rate. The high estimate is based on the modified NRC Curve III and the local historical vertical shift rate.

An estimate of a representative vertical land movement rate may be derived from the gage data by subtracting an estimate of the eustatic sea level change rate from the relative sea level rate for the gage. IPCC concludes that global mean sea level rose at an average rate of about 1.7 +/- 0.5, or 1.2 – 2.2 mm/year during the twentieth century.

The EC also provides guidance on estimating the eustatic rate by observation of a geographic region that is thought to be vertically stable. **Figure 3-1** is the example shown in the EC for the Gulf Coast. The average of the five stations shown in the rectangle is 2.28 mm/year. This estimate of the regional sea level trend may be subtracted from the historical local sea level trend to obtain an estimate of the local vertical land movement. However, this rate is even higher than the upper range of the global eustatic rate estimated by the IPCC, 1.7 + 0.5, or 2.2 mm/year and no explanation is given in the EC as to why the Gulf Coast would experience a somewhat higher rate of sea level change than the global average. As the low estimate is based on a linear projection of the local sea level trend only, the method selected for estimating the vertical land movement only effects the intermediate and high estimates of RSLC. The ultimate difference at the end of a 50-year projection is minimal, in the order of 1 to 2 tenths of a foot, so selection of either method is not critical to the overall projection of sea level rates. Using 1.7 mm/year provides a slightly higher estimate of vertical movement and therefore a more conservative estimate of future sea levee rise.

Those projections made by MVN personnel have been based on the global eustatic rate of 1.7 mm/year. However, the sea level trend estimated by MVS personnel for the LCA Atchafalaya River diversion to Terrebonne Marshes used the 2.28 mm/year rate to estimate vertical land movement. Further analysis may be warranted to determine the most appropriate rate for use in sea level trend projections for MVN projects. Consideration should be given to the numerical inconsistency introduced by using a rate of 1.7 mm/year to project future eustatic trends for the intermediate and high estimates and the use of 2.28 mm/year to estimate the local vertical movement rate. **Figure 3-1** is Figure C-2 from the EC showing an example of a region that may exhibit a regional rate of mean sea level change that is different than the global eustatic rate.

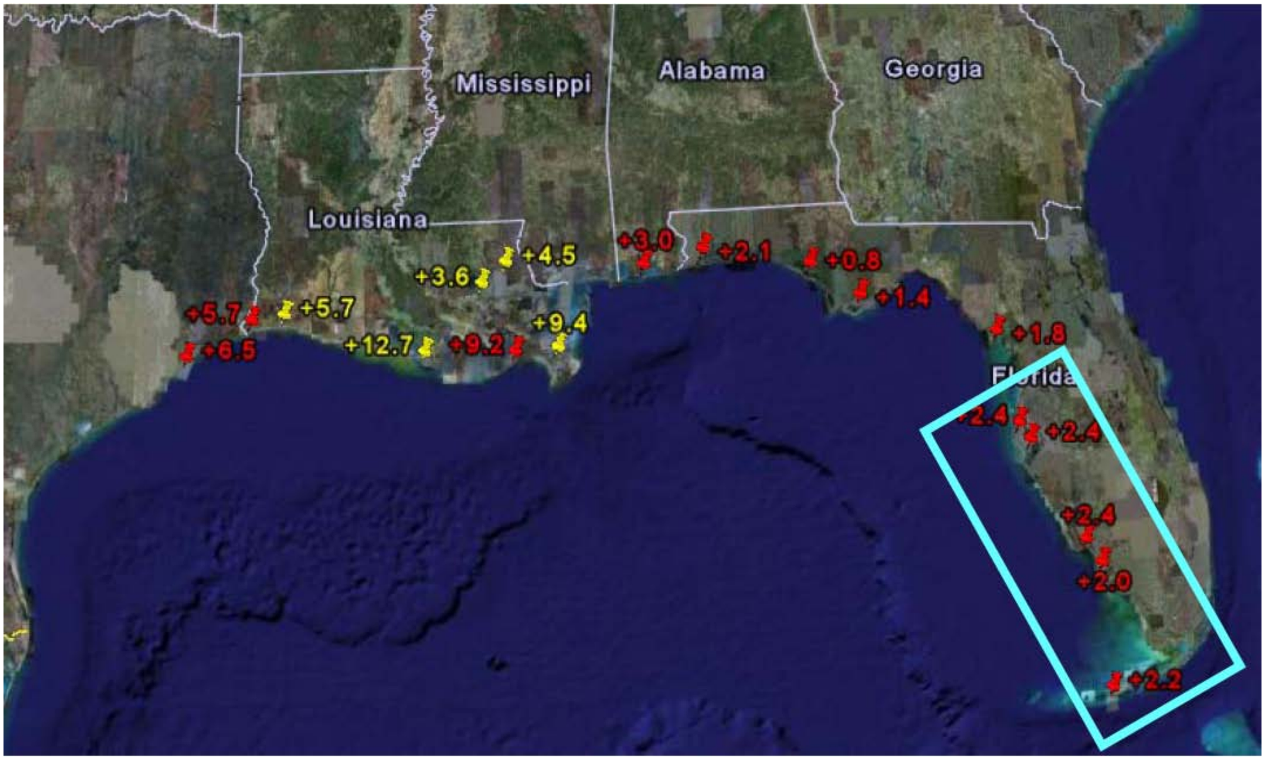


Figure 3-1 Figure C-2 from the EC

3.2 SUMMARY OF CO-OPS TIDAL GAGE SEA LEVEL TRENDS THAT MAY BE OF USE TO MVN INTERESTS

The Center for Operational Oceanographic Products and Services (CO-OPS), National Ocean Service (NOS), National Oceanic and Atmospheric Administration (NOAA) is recommended by the EC as a source of information on tide gage local historical trends. However, only one gage is currently available with up-to-date information regarding sea level trend data in Louisiana, namely the NOAA tide gage at Grand Isle, LA:

http://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?stnid=8761724 Grand Isle, LA

As of September, 2009, the rate shown on the site is 9.24 mm/year with a 95% confidence interval of +/- 0.59 mm/year based on monthly mean sea level data from 1947 to 2006 which is equivalent to a change of 3.03 ft in 100 years.

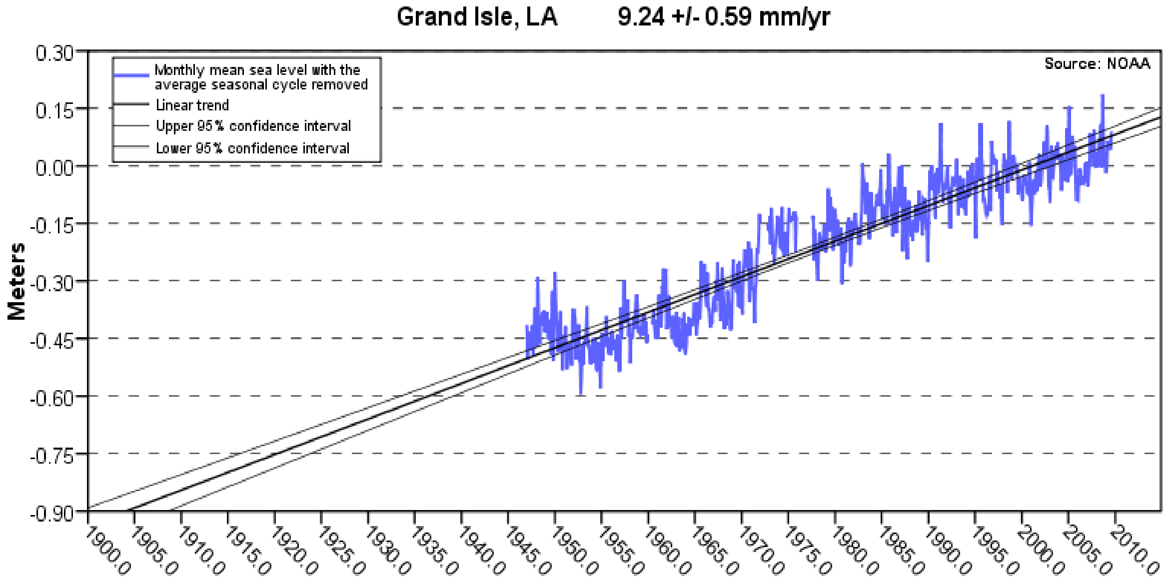


Figure 3-2 CO-OPS Grand Isle Sea Level Trend

Sea level trend data for Eugene Island is available on the CO-OPS internet site. However, the record length is insufficient to meet the EC requirement of a 40-year record length. The rate for Eugene Island is very close to that given for Grand Isle. Therefore, Grand Isle may be considered a viable alternate data source for trend information for projects closer in proximity to the Eugene Island geographic area.

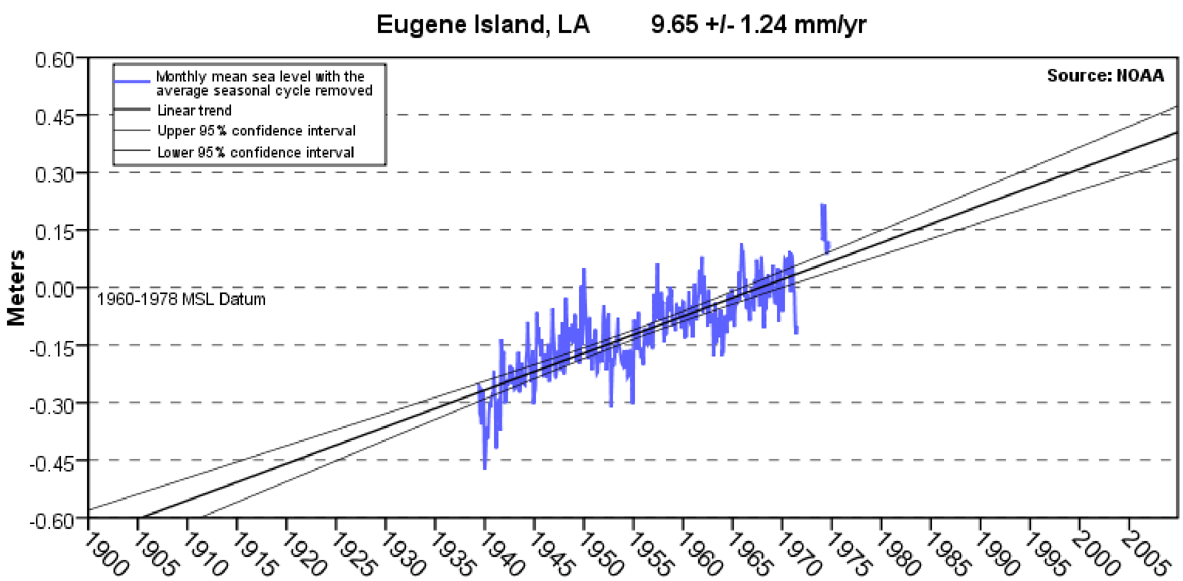


Figure 3-3 CO-OPS Eugene Island Sea Level Trend

Another possible source of sea level trend data that may be of use in MVN projects is also located on the CO-OPS internet site, namely the Sabine Pass gage located near the Texas-Louisiana border. This data source may be of use in projects located on the Chenier Plains of Western Louisiana.

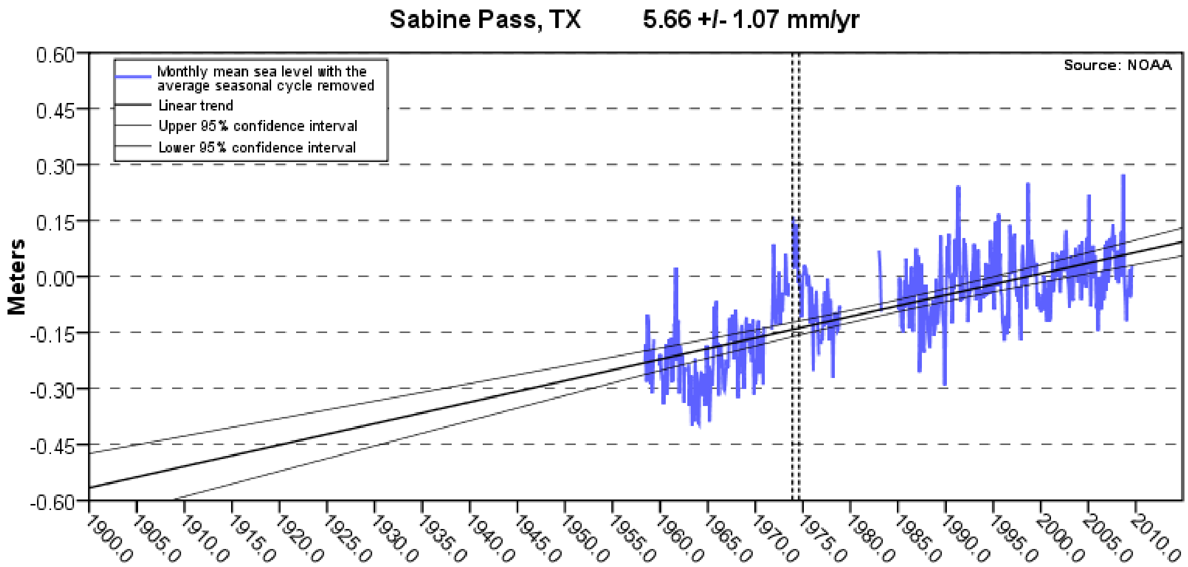


Figure 3-4 CO-OPS Sabine Pass, TX Sea Level Trend

The closest CO-OPS tide station with sea level trend data to the East of Louisiana is the gage located at Dauphin Island, AL. Although the vertical land movement rates derived from this trend should not be considered representative of conditions in Louisiana, the gage may be useful in providing vertical movement adjustment data for numerical model grids and meshes that are used to assess water level trends for MVN projects.

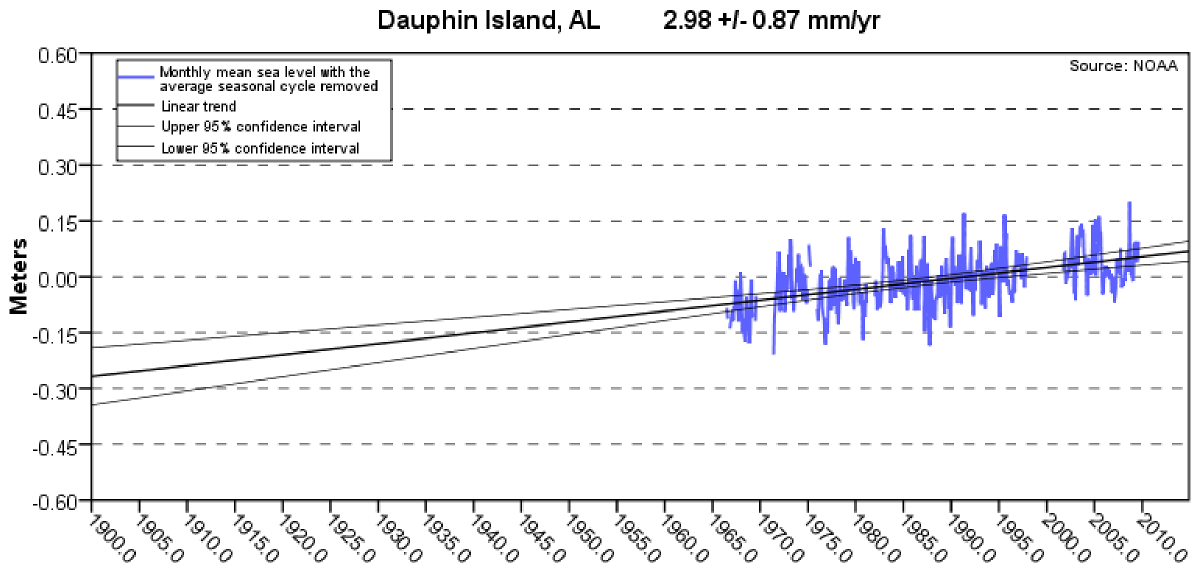


Figure 3-5 CO-OPS Dauphin Island, AL Sea Level Trend

3.3 EXAMPLE EC METHODOLOGY USING CO-OPS DATA

As an example, using the methods outlined in the EC and the Grand Isle trend data as given by CO-OPS, sea level trends may be estimated as follows:

Using the rate of 9.24 mm/year, a starting year of 2006, and a 50-year project life, a RSLC of 1.5 ft is projected for the end of the project life in the year 2062. The rate of 9.24 mm/year is considered to include both the eustatic and local vertical movement contributions to the estimated RSLC.

In order to estimate the local vertical movement trend for the project area, the global eustatic rate (1.7 mm/year) is subtracted from the local sea level rate or:

Local vertical trend = 9.24 mm/year – 1.7 mm/year = 7.54 mm/year.

The estimate for the local vertical land movement is used in conjunction with estimates for the eustatic rates using NRC Curves I and III to determine the intermediate and high projections of RSLC for the project. The following formula is used to estimate the total rise in eustatic sea level for the project life for the intermediate and high rate scenarios of sea level change:

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^{\,2} - t_1^{\,2})$$

where:
b is the acceleration factor related to NRC Curves I and III or 2.36E-5 and 1.005E-4, respectively
t_1 is the time between the project's construction date and 1986 (in years)

t_2 is the time between a future date at which one wants an estimate for sea-level rise and 1986 (in years)

These eustatic estimates are added to the local vertical trend estimate to get the total RSLC for the intermediate and high rate scenarios.

A summary of the estimated total RSLC is summarized in **Table 3-1** in 5-year increments for each of the scenarios through the project life of 50 years.

Table 3-1 Summary of 5-year Relative Sea Level Change for Each Scenario

Project Year	Scenario 1 Low Rate (ft)	Scenario 2 Intermediate Rate (ft)	Scenario 3 High Rate (ft)
2006	0.0	0.0	0.0
2011	0.2	0.2	0.2
2016	0.3	0.3	0.5
2021	0.5	0.5	0.7
2026	0.6	0.7	1.0
2031	0.8	0.9	1.3
2036	0.9	1.1	1.6
2041	1.1	1.3	1.9
2046	1.2	1.5	2.3
2051	1.4	1.7	2.6
2056	1.5	1.9	3.0

3.4 USE OF CORPS TIDAL GAGES TO ASSESS SEA LEVEL TRENDS

Unfortunately, there are no data sources on the CO-OPS internet site that may be considered representative of sea level trends for MVN projects on the east side of the Mississippi River. Therefore, an alternate data source is needed to determine sea level trends on the east side of the River and further inland than those projects on the immediate coast of Louisiana.

The Corps of Engineers has historically maintained several tidal gages that may be a source of local sea level trend data for those geographic areas that are not represented by the CO-OPS data, especially the areas in and around Lake Pontchartrain, the Breton Sound area, and the interior areas of the Barataria and Terrebonne basins.

To date, data has been collected and analyzed for several gage records, including the following listed in **Table 3-2**.

Table 3-2 List of USACE Gages Evaluated for Sea Level Trends

Gage	ID No	Period of Record Analyzed
Lake Pontchartrain at West End, LA	85625	3/10/1949 - 8/4/2009
Bayou Barataria at Barataria, LA	82750	11/2/1951 – 11/12/1992
Mississippi River – Gulf Outlet at Shell Beach, LA	85800	6/30/1961 – 12/16/2002
Lake Pontchartrain at Mandeville, LA	85575	8/2/1957 – 7/26/2002
Bayou Terre Aux Boeufs at Delacroix, LA	85780	5/15/1975 – 8/28/2005
Bayou Bienvenue at Floodgate (East), LA	76025	12/23/1974 – 11/23/1992

Only those records at West End, Shell Beach, Mandeville, and Barataria met the requirement for the 40-year record length and had sufficient adjustment records to complete an analysis with a high level of confidence in the predicted trends. Detailed descriptions of the analyses of these gages follow.

Lake Pontchartrain at West End, LA

The following adjustments were received from MVN Survey Section for the West End gage:

-10.68 ft on 4/7/1975, adjusted to MSL (1975 adj.), gage book indicates this correction was applied for the entire year when published

-0.25 ft on 3/19/1986, correction, this correction was applied for the entire year

-0.32 ft on 5/15/1987, epoch adjustment, this adjustment was applied for the entire year

-0.81 ft on 12/19/2006, datum change to NAVD, applies to data starting on 19 December 2006 according to www.rivergages.com site

These adjustments were added back into the data in a cumulative manner to get the results shown on the figure below.

+ 10.68 ft from 1/1/1975 to 12/31/1985
+ 10.93 ft from 1/1/1986 to 12/31/1986
+ 11.25 ft from 1/1/1987 to 12/18/2006
+ 12.06 ft from 12/19/2006 to latest

A linear best fit gives a slope of 0.0283 ft/year (8.63 mm/year). The standard error for the linear model data set is 1.16 ft (354 mm).

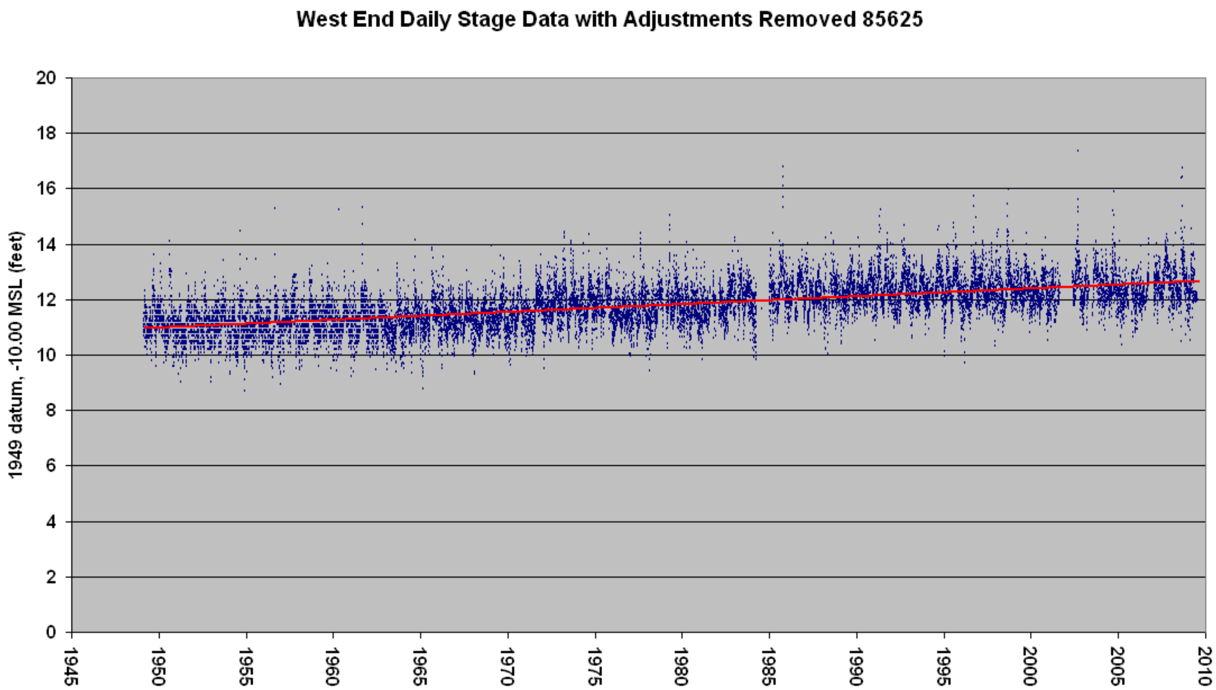


Figure 3-6 USACE West End Gage Daily Stage Data with Trend Line

Bayou Barataria at Barataria, LA

One adjustment was provided by ED-SS, an adjustment of -1.26 ft was removed from the data starting at 1 January 1981.

The linear relative sea level trend for this gage is 0.0219 ft/year (6.68 mm/year) with a standard model error of 0.4503 ft (137 mm).

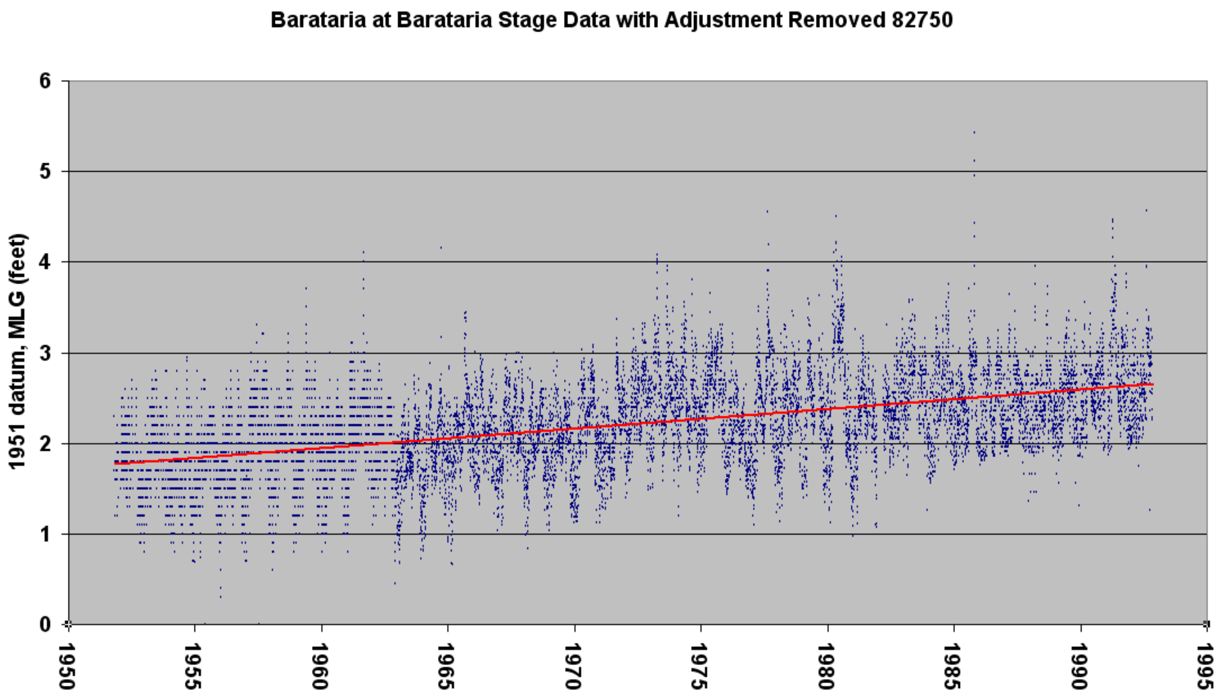


Figure 3-7 USACE Barataria at Barataria Daily Stage Data with Trend Line

Mississippi River Gulf Outlet at Shell Beach, LA

No adjustments were found in the records which only go back to the 1980's, however the gage books show a datum change between MSL in 1977 and NGVD in 1978. As the records in surveys do not go back this far, the adjustment was estimated mathematically.

It is not clear exactly when the adjustment to the data occurred, but there is a gap in the data from 16 June 1978 to 6 August 1978. Assuming the adjustment occurred during this data gap, linear best fit relations can be developed for the two records from 30 June 1961 to 15 June 1978 and 7August 1978 to 16 December 2002. Using these two linear equations and 1978.5 as the independent variable, a difference of 1.0 ft is determined as the adjustment.

This adjustment was added back into the data to get the results shown on the figure below.

+1.0 ft from 17 August 1978 to latest

A linear best fit gives a slope of 0.0336 ft/year (10.24 mm/year). The standard model error for this data set is 0.7747 ft (236 mm).

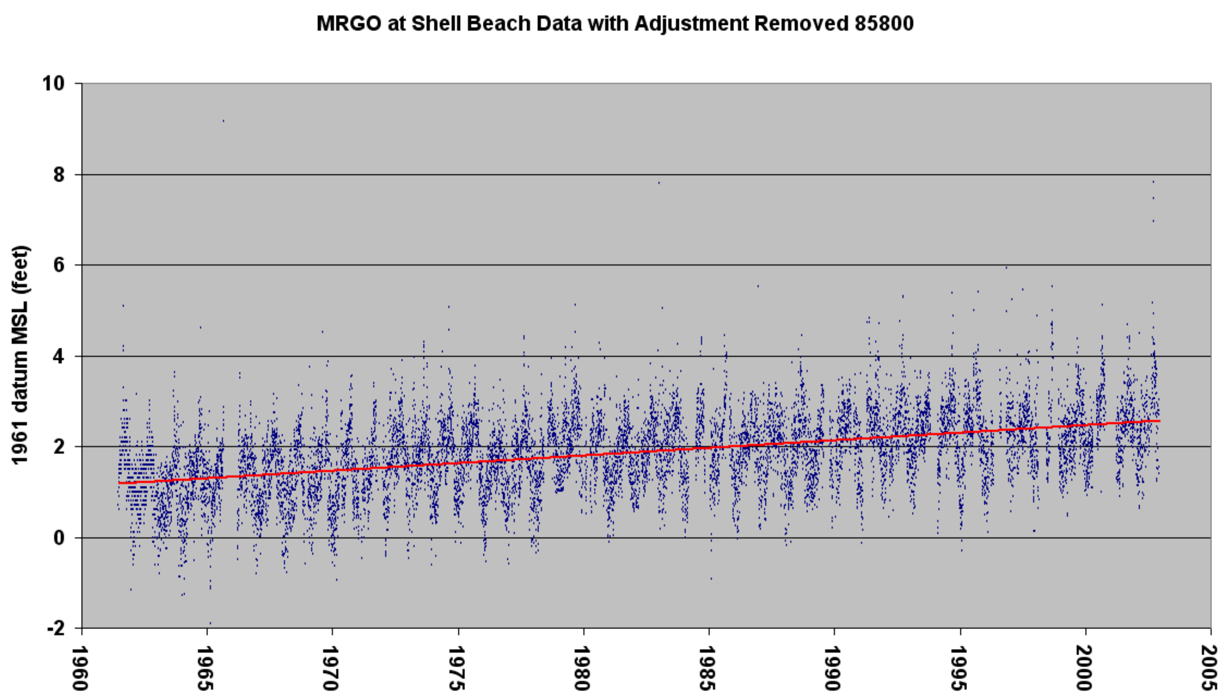


Figure 3-8 USACE MRGO at Shell Beach Daily Stage Data with Trend Line

Lake Pontchartrain at Mandeville

Adjustment records were not available from ED-SS for this site, but it was known that at least one adjustment was made as a result of a change in the reference bench mark. Therefore, a difference analysis was performed with the West End gage to determine the adjustments made to this gage. This analysis revealed that at least two adjustments were made to the Mandeville gage assuming the West End data could be used as an accurate reference.

The adjustments were estimated, as follows, from the difference analysis and the gage data was shifted accordingly.

A datum adjustment of +10.00 starting on 1 January 1975
A cumulative adjustment of +10.36 ft starting on 23 July 1985
A cumulative adjustment of +11.19 ft starting on 4 November 1989

The linear slope of the trend is 0.0218 ft/year (6.64 mm/year). The standard model error for this data set is 0.6512 ft (198 mm).

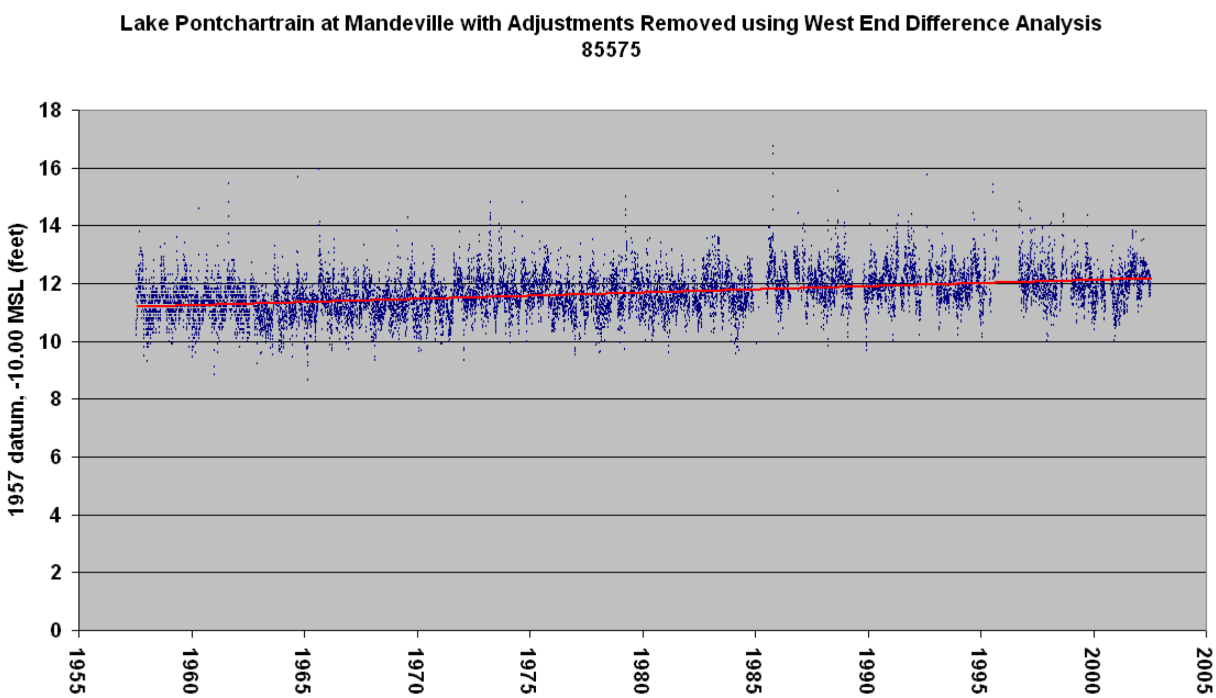


Figure 3-9 USACE Lake Pontchartrain at Mandeville Daily Stage Data with Trend Line

Bayou Terre Aux Boeufs at Delacroix, LA

No adjustments were made to this gage throughout its history.

The relative sea level rate predicted by this gage using a linear trend is 0.0197 ft/year (6.00 mm/year) with a standard model error of 0.5847 ft (178 mm).

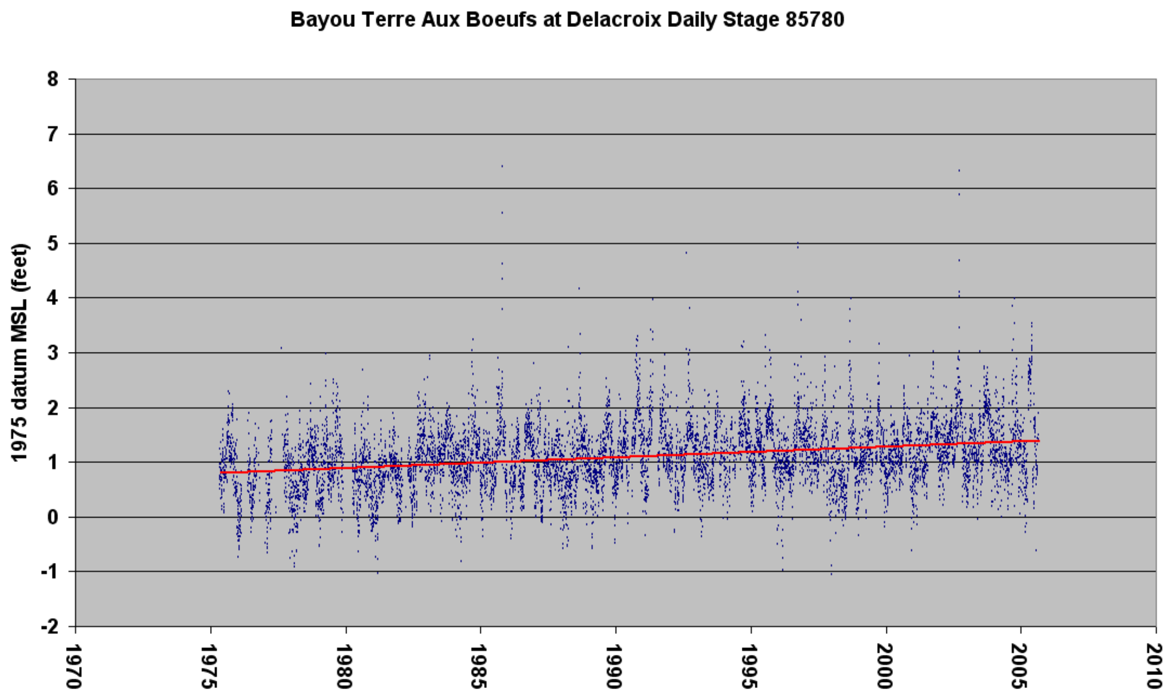


Figure 3-10 USACE Bayou Terre Aux Boeufs Daily Stage Data with Trend Line

Bayou Bienvenue at Floodgate (East), LA

An adjustment of 0.5 ft was added back to the data from 1 January 1983 – 23 November 1992.

The relative sea level rate predicted by this gage using a linear trend is 0.0471 ft/year (14.36 mm/year) with a standard model error of 0.7424 ft (226 mm).

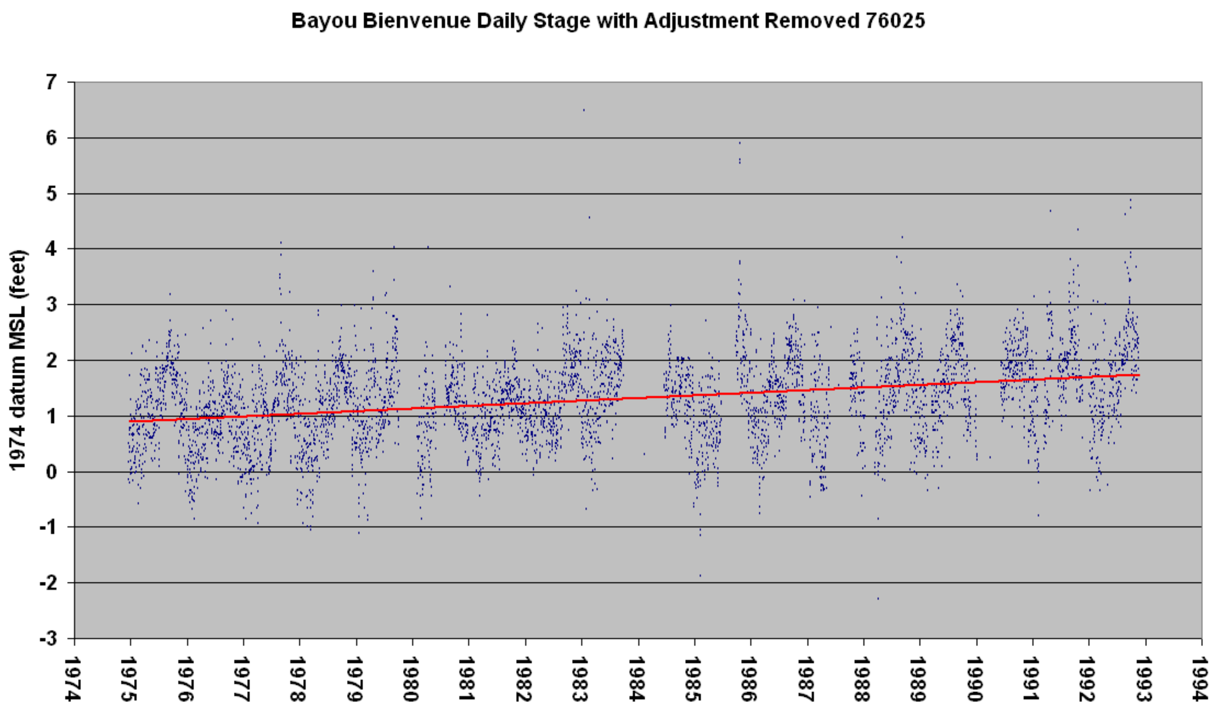


Figure 3-11 USACE Bayou Bienvenue Daily Stage Data with Trend Line

A comparison between the Bayou Bienvenue and Shell Beach stage data for the Bayou Bienvenue period of record reveals the disparate RSLC rates for two gages in close proximity to each other. See **Figure 3-12** for trend information.

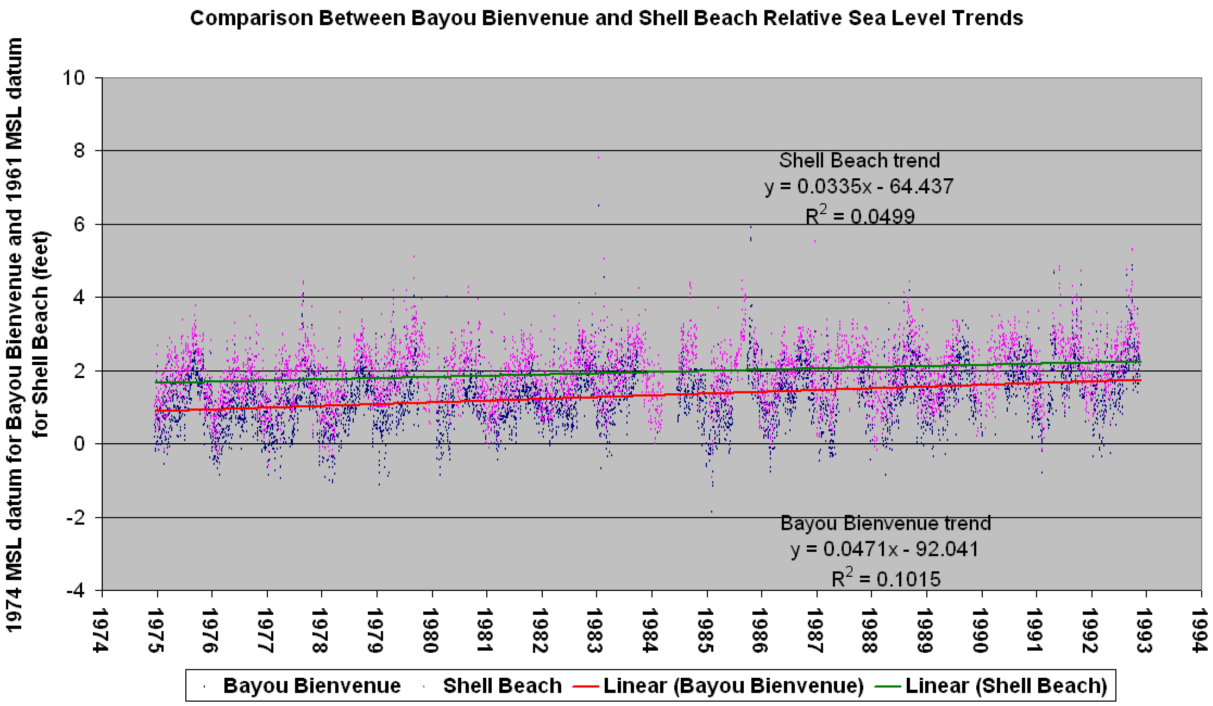


Figure 3-12 Trend Comparison Between Shell Beach and Bayou Bienvenue for the Same Period of Record

3.5 SUMMARY AND CONCLUSIONS

Table 3 summarizes the relative sea level trend for the CO-OPS and USACE gage sites. The CO-OPS data was fitted with a linear regression with autoregressive residuals to monthly averages. The Corps gages were fitted with a simple linear regression to daily data.

Table 3-3 Summary of Relative Sea Level Trends for CO-OPS and USACE Gages

Gage Location	Period of Record	Mean Relative Sea Level Trend (mm/year)	Standard Error of Model (mm)
Grand Isle (CO-OPS)	1947 - 2006	+9.24	51*
Eugene Island (CO-OPS)	1939 - 1974	+9.65	53*
Sabine Pass (CO-OPS)	1958 - 2006	+5.66	73*
Dauphine Island (CO-OPS)	1966 - 2006	+2.98	53*
West End (USACE 85625)	1949 - 2009	+8.63	354
Mandeville (USACE 85575)	1959 - 2002	+6.64	198
Barataria at Barataria (USACE 82750)	1951- 1992	+6.68	137
Shell Beach (USACE 85800)	1961 - 2002	+10.24	236
Bayou Terre Aux Boeufs (USACE 85780)	1975 - 2005	+6.00	178

Gage Location	Period of Record	Mean Relative Sea Level Trend (mm/year)	Standard Error of Model (mm)
Bayou Bienvenue (USACE 76025)	1974 - 1992	+14.36	226

* Source: NOAA Technical Report NOS CO-OPS 36 for data through 2000 except

Given the need for relative sea level trend information in the New Orleans and surrounding area, at this time it is recommended that the trends determined for the USACE gages at West End, Barataria, and Shell Beach represent the relative sea level trend for the Lake Pontchartrain, West Bank, and Lake Borgne areas respectively. See **Table 3-4** for relative sea level projections using trend information from the three gage analyses. The analyses were performed following (EC) 1165-2-211 methodology with a starting year of 2011 and a global eustatic rate of 1.7 mm/year to estimate local vertical movement.

Table 3-4 Geographically Representative Relative Sea Level Projection Estimates for the Year 2057

Representative Location	Low Estimate for Year 2057 (ft higher than year 2011 zero)	Intermediate Estimate for Year 2057 (ft higher than year 2011 zero)	High Estimate for Year 2057 (ft higher than year 2011 zero)
Lake Pontchartrain (based on West End gage analysis)	1.3	1.6	2.8
West Bank (based on Barataria gage analysis)	1.0	1.4	2.5
Lake Borgne (based on Shell Beach gage analysis)	1.5	1.9	3.0

4.0 ADCIRC MODEL RESULTS ANALYSIS

4.1 INTRODUCTION

The effects of RSLC on storm surge have been evaluated using ADCIRC, a numerical storm surge model, for a number of USACE projects in Southern Louisiana. **Table 4-1** provides a summary of the storm sets and RSLC scenarios evaluated.

Table 4-1 Summary of ADCIRC Storms with Relative Sea Level Change

Grid	ERDC Modeling Directory	Number of Storms	Relative Sea Level Change (ft)
2010	Simulations.sl15_2010_SLR1.46_57	9	1.00
2010	Simulations.sl15_2010_SLR2.46_57	9	2.00
2010	Simulations.sl15_2010_SLR3.46_57	9	3.00
2007	Simulations.sl15v6f_2007_MTG_sea_level_rise _0.35052.46_58	115	1.15
2007	Simulations.sl15v6f_2007_MTG_sea_level_rise _0.97536.46_58	11	3.20

The latest modeling study which evaluated the effects of RSLC was the Morganza to Gulf Hurricane Protection Project. For this project, 115 synthetic storms were run for the without RSLC and RSLC = 1.15 ft scenarios. 11 storms were also completed on the MTG* what is this grid with RSLC = 3.2 ft. In a separate study, a set of nine storms were ran on the 2010 base condition mesh for three RSLC scenarios: 1.0, 2.0, and 3.0 ft. These 27 ADCIRC runs were part of the LACPR investigation into the effects of RSLC. The results of both sets of runs provided much insight into the amplification of storm surge due to RSLC.

Historically, RSLC estimates were linearly added to design surge levels, but this approach is too simplistic. Recent efforts to model the effects of RSLC have shown a non-linear effect. The impacts of increasing relative sea level are two-fold, the surge wave propagates faster, and the depth-limited wave height and setup increases. RSLC increases peak surge more than linearly, meaning that a hypothetical sea level change of 1.0 ft will result in more than 1.0 ft increase in surge level. The complex, shallow geometry and bathymetry of Southeast Louisiana and the relative phasing of the storm and the surge propagation contribute to the amplification. To determine the non-linear effect on storm surge, a regression analysis was performed on the peak surge results of the 115 Morganza to Gulf ADCIRC storms and the original nine storms for all RSLC scenarios.

A regression was made by plotting surge without RSLC versus surge with RSLC and determining a regression line through the data. From the regression analysis, an amplification factor was determined for different areas around the HSDRRS.

4.2 REGIONAL SEA LEVEL CHANGE SURGE REGRESSION ANALYSIS

The existing condition 1% surge levels and standard deviations for the HSDRRS of greater New Orleans were derived from surge results from 304 ADCIRC runs, 152 for Southeastern Louisiana, and 152 for Southwestern Louisiana. The modified probabilistic JPM-OS method was used to determine stage-frequency curves. To use this statistical method a large number of synthetic storms were needed. 304 storms for with RSLC conditions were not run for the HSDRRS, so future 1% surge levels could not be determined using the modified probabilistic JPM-OS method. Since a limited number of runs with RSLC exist, a different approach was developed to determine 1% future surge levels.

For the Morganza to Gulf project, a partial set of the original 304 storms was used to determine the 1% existing and future condition surge levels. This partial set of storms was evaluated for the future condition by changing the initial water surface elevation to account for a RSLC of 1.15 ft and changing the roughness to account for wetland loss. The Morganza to Gulf runs consisted of 115 storms out of the original 304, with 41 from the East set and 74 from the West set. The rest of the storms were thrown out to save computation time as they do not cause significant surge in the project area, and thus are not significant to the statistical process used to determine the 1% surge levels. The Morganza to Gulf runs provided accurate 1% future surge conditions for the project area, but no statistical calculations for areas outside of the project area. However, the results of the partial set of storms provided insight into the effects of RSLC on surge for other areas.

Peak storm surge for a set of save points, known as the Q-set, were extracted from the ADCIRC output files for all 115 Morganza to Gulf storms and the nine original RSLC storms. **Figure 4-1** shows the distribution of the Q-set save points over Southern Louisiana.

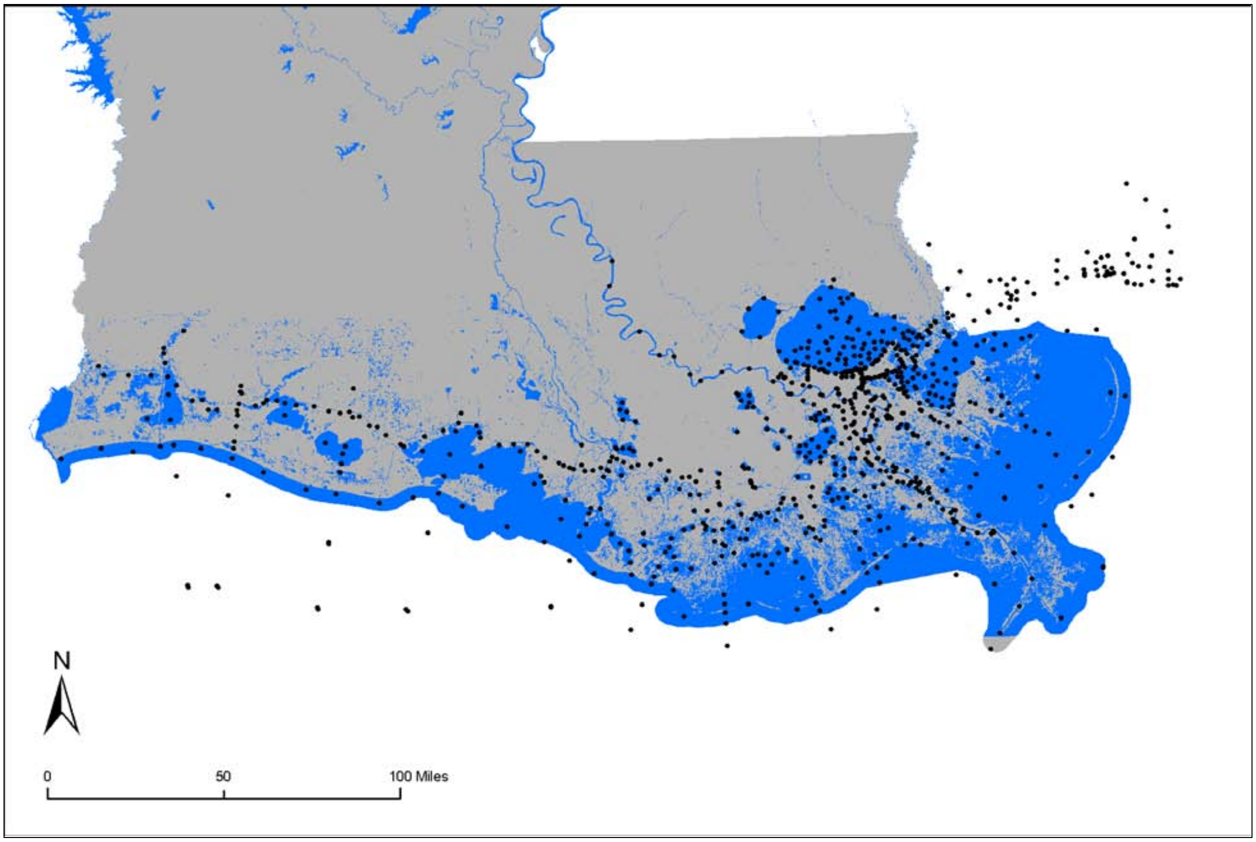


Figure 4-1 ADCIRC Save Points – Q-Set Distribution Over Southern Louisiana

A regression analysis was performed for each Q point by plotting storm surge without RSLC versus storm surge with RSLC for all storms. A fairly good correlation between surge without RSLC and surge with RSLC was observed for many points in Southern Louisiana, although some points had better correlations than others. A second order polynomial trend line was computed for each regression at each Q point. The R^2 value and standard error values were determined to assess the goodness of the fit. A second order polynomial was shown to statistically give a better fit than a linear trend line. The future condition 1% surge was computed by plugging in the existing 1% surge level into the 2nd order polynomial trend line equation. **Table 4-2** provides the 1% surge without RSLC, 2nd order polynomial trend line, the R^2 value, the standard deviation and the computed 1% surge with RSLC, the increase in 1% surge due to RSLC, and the surge amplification factor for several points around the HSDRRS of greater New Orleans. The amplification factor is equal to the increase in surge divided by the increase in regional sea level. **Table 4-3** shows the same tables for the original RSCL set of runs with RSLC 1.0 ft. **Figure 4-2** shows the location of the selected Q-set points. **Figures 4-3** through **4-5** show the regression plots for the selected Q-pts for the Morganza to Gulf runs with RSLC 1.15. **Figures 4-6** through **4-8** show the regression plots for the selected Q-pts for the Original RSLC runs with RSLC 1.00 ft. **Figures 4-9** through **4-11** show the regression plots for the selected Q-pts for the original RSLC runs with RSLC 3.00 ft.

Table 4-2 Regression Analysis – Morganza to Gulf with Relative Sea Level Change 1.15

Without SLR vs. With Relative Sea Level Change 115 Storms - 2nd Order Polynomial Regression Analysis Morganza to Gulf Runs with Relative Sea Level Change 1.15 ft								
Point ID	**Location**	**1% Surge Without RSLC (ft NAVD88 2004.65)**	**2nd Order Polynomial Trend Line**	**R^2 Value**	**Standard Deviation**	**1% Surge With RSLC (ft NAVD88 2004.65)**	**Increase in SWL Due to RSLC (ft)**	**Surge Amplification Factor**
305	Lake Borgne	17.5	$y = -0.000x^2 + 0.977x + 1.202$	0.999	0.189	18.31	0.81	0.70
301	Lake Borgne	15.4	$y = -0.002x^2 + 1.022x + 1.043$	0.999	0.183	16.27	0.87	0.76
286	Lake Pontchartrain	8.8	$y = -0.026x^2 + 1.353x + 0.583$	0.981	0.335	10.44	1.64	1.43
288	Lake Pontchartrain	8.7	$y = -0.022x^2 + 1.330x + 0.620$	0.988	0.282	10.50	1.80	1.57
334	West Bank	6.0	$y = -0.029x^2 + 1.404x + 0.498$	0.970	0.425	7.86	1.86	1.62
467	West Bank	7.0	$y = -0.019x^2 + 1.291x + 0.570$	0.976	0.458	8.67	1.67	1.45

Table 4-3 Regression Analysis – HSDRRS Original Sea Level Change Runs with Relative Sea Level Change 1.0 ft

Without Relative Sea Level Rise vs. With Relative Sea Level Change 9 Storms - 2nd Order Polynomial Regression Analysis Original Relative Sea Level Change Runs With Relative Sea Level Change 1.00 ft								
Point ID	**Location**	**1% Surge Without RSLC (ft NAVD88 2004.65)**	**2nd Order Polynomial Trend Line**	**R^2 Value**	**Standard Deviation**	**1% Surge With RSLC (ft NAVD88 2004.65)**	**Increase in SWL Due to RSLC (ft)**	**Surge Amplification Factor**
305	Lake Borgne	17.5	$y = -0.00x^2 + 1.183x + 0.151$	0.998	0.126	18.33	0.83	0.83
301	Lake Borgne	15.4	$y = -0.00x^2 + 1.143x + 0.499$	0.999	0.095	16.45	1.05	1.05
286	Lake Pontchartrain	8.8	$y = -0.00x^2 + 1.035x + 1.108$	0.993	0.120	9.88	1.08	1.08
288	Lake Pontchartrain	8.7	$y = 0.007x^2 + 0.907x + 1.398$	0.993	0.118	9.90	1.20	1.20
334	West Bank	6.0	$y = -0.00x^2 + 1.168x + 0.491$	0.986	0.242	7.20	1.20	1.20
467	West Bank	7.0	$y = -0.00x^2 + 1.140x + 0.619$	0.991	0.256	8.27	1.27	1.27

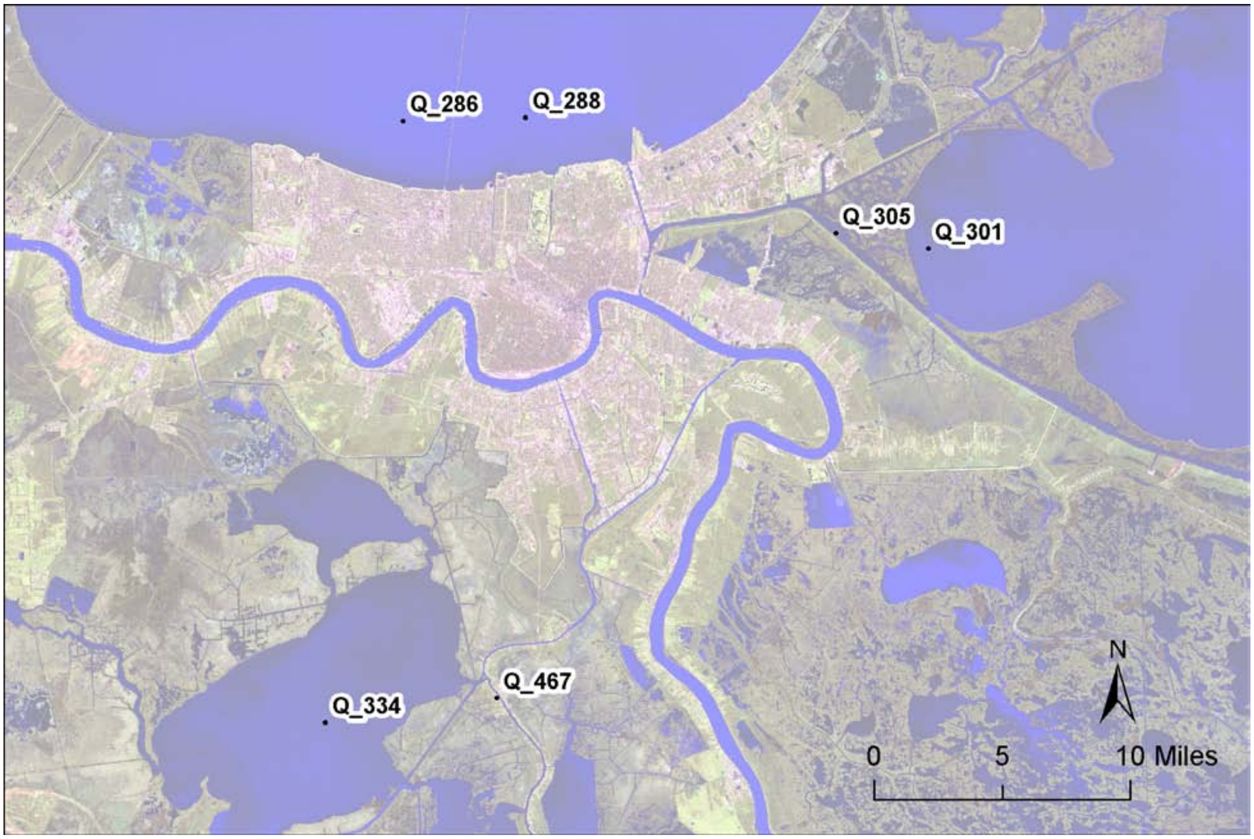


Figure 4-2 Location of Selected Q-set Points

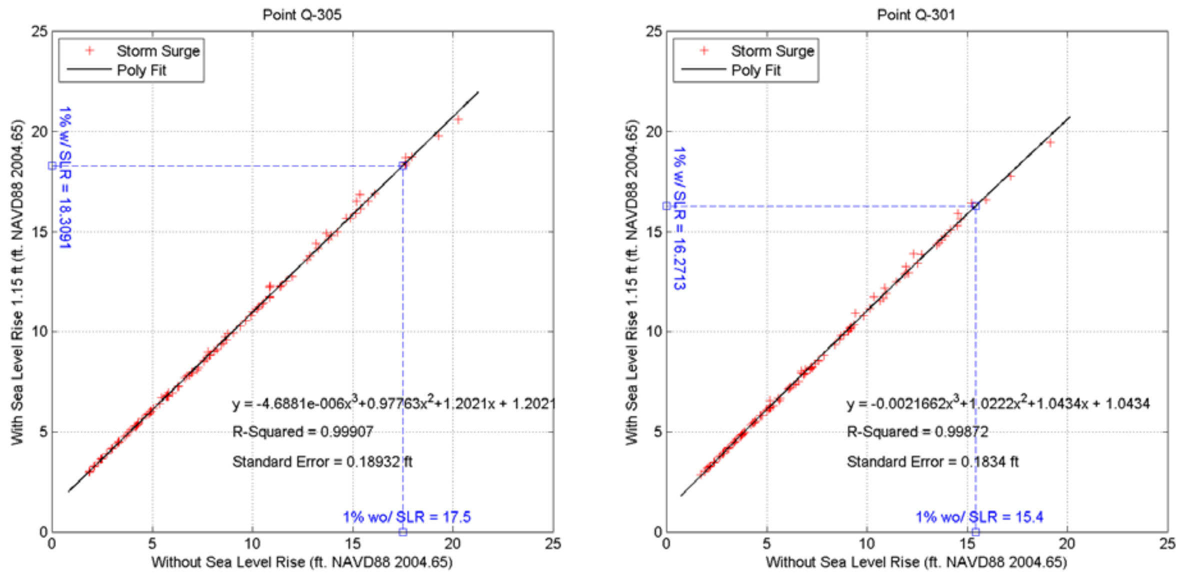


Figure 4-3 Regression Plots for Lake Borgne Area for MTG Runs With Relative Sea Level Change 1.15

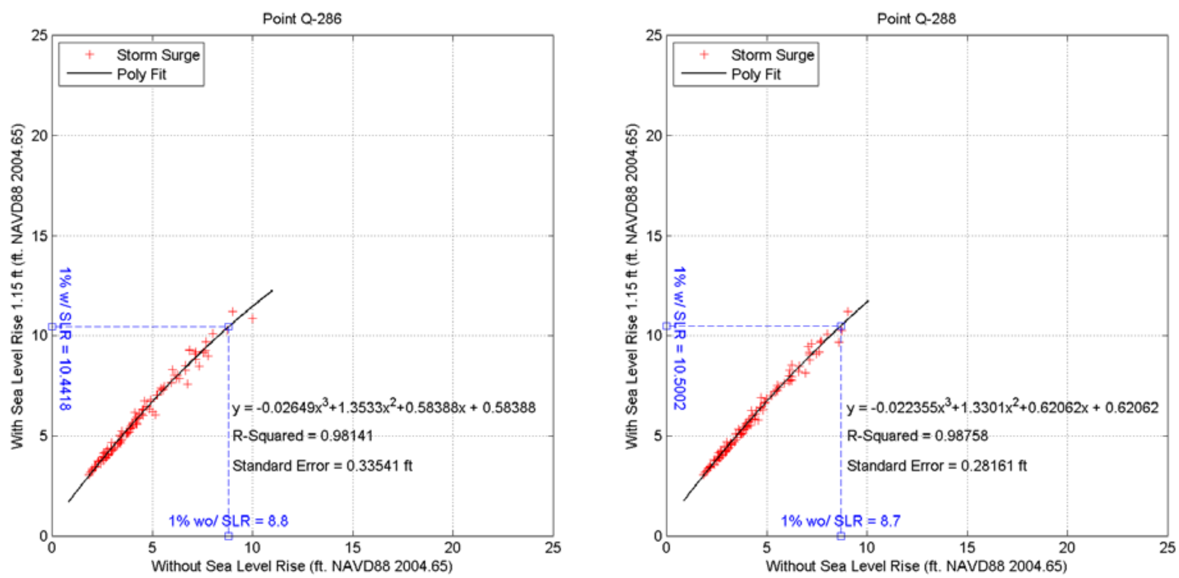


Figure 4-4 Regression Plots for Lake Pontchartrain Area for MTG Runs With Relative Sea Level Change 1.15

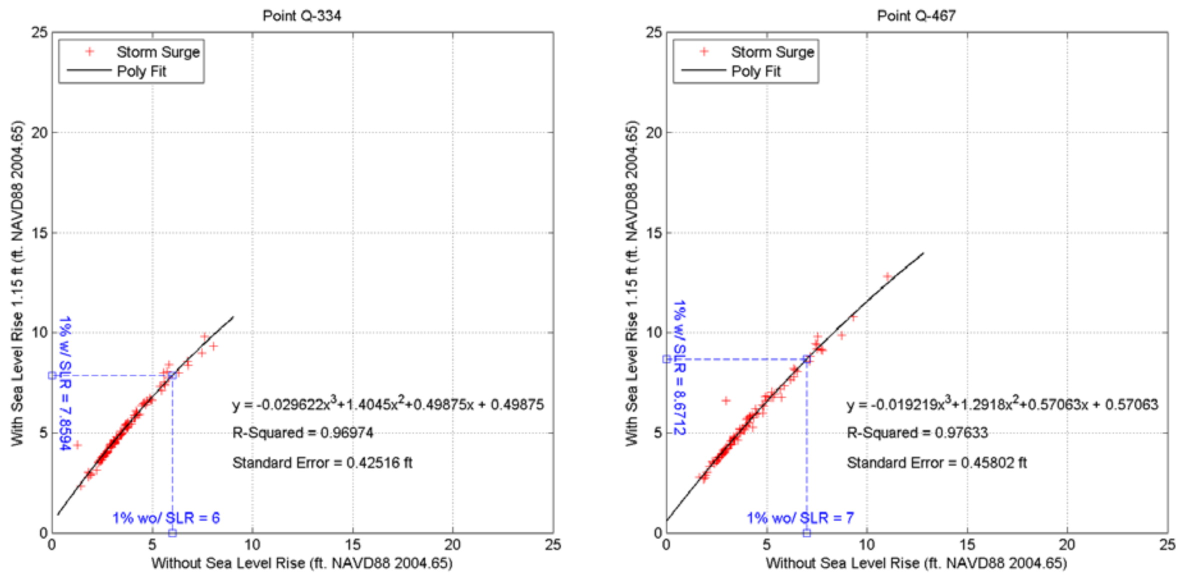


Figure 4-5 Regression Plots for West Bank Area for MTG Runs With Relative Sea Level Change 1.15

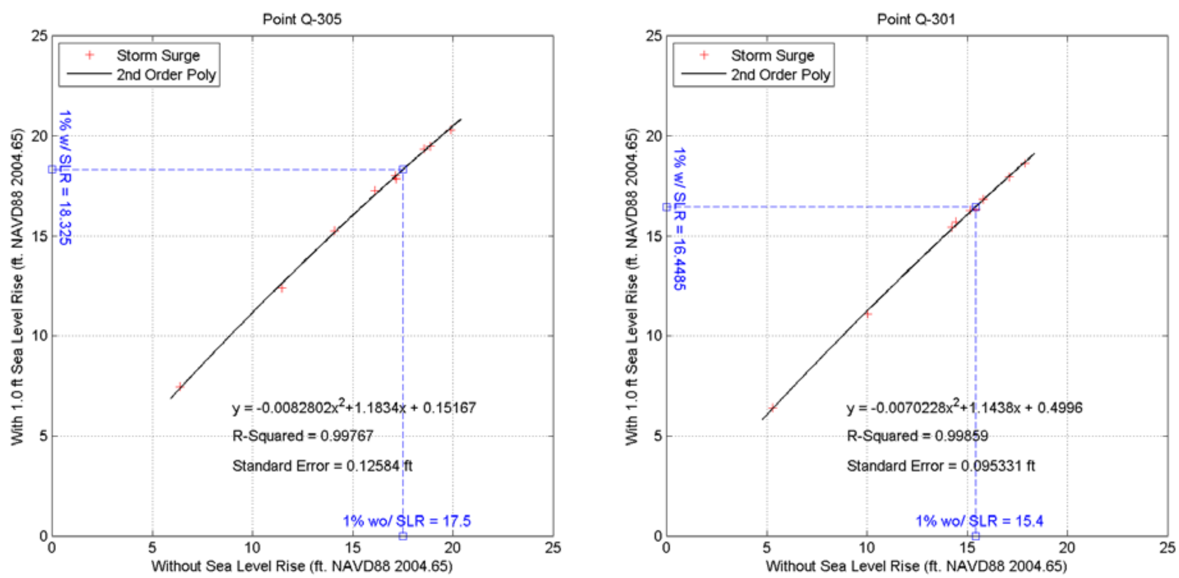


Figure 4-6 Regression Plots for Lake Borgne Area for Original Relative Sea Level Rise Runs With Relative Sea Level Change 1.0 ft

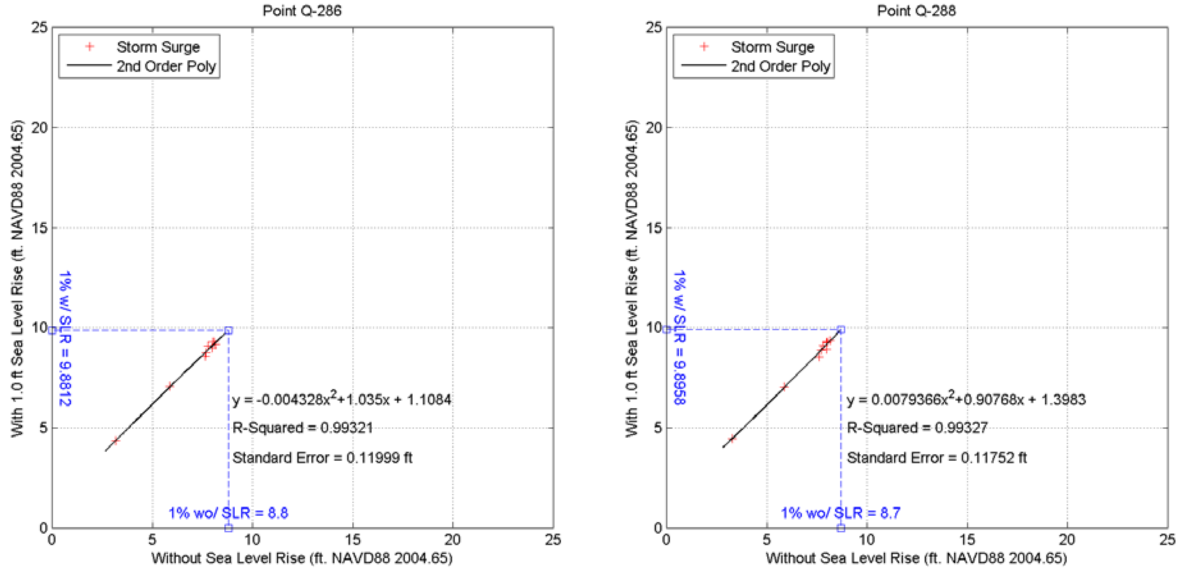


Figure 4-7 Regression Plots for Lake Pontchartrain Area for Original Relative Sea Level Change Runs With Relative Sea Level Change 1.0 ft

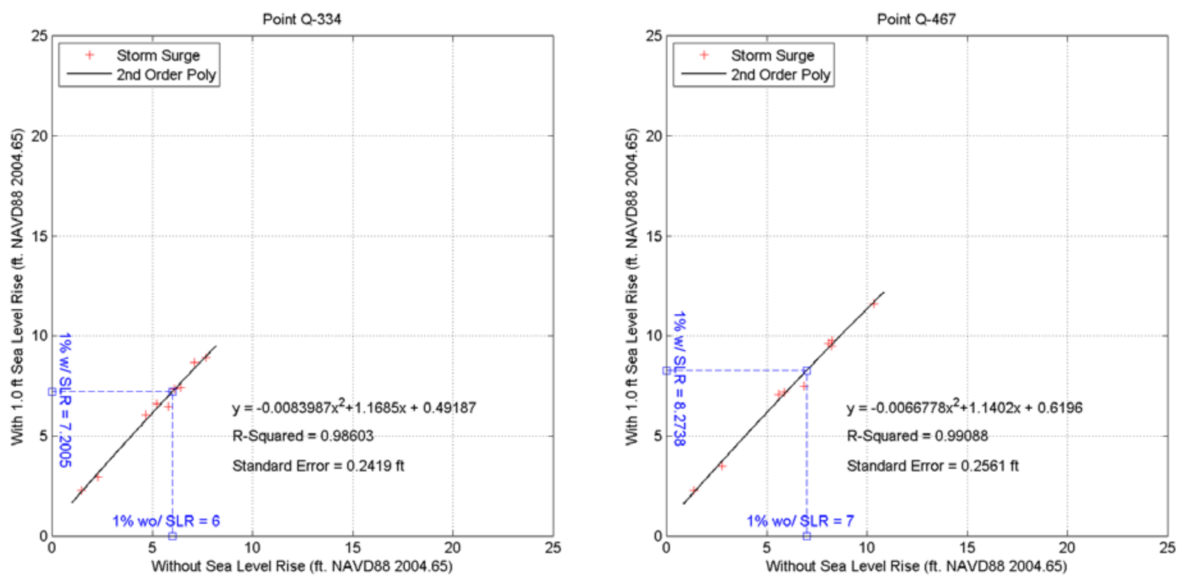


Figure 4-8 Regression Plots for West Bank Area for Original Relative Sea Level Rise Change With Relative Sea Level Change 1.0 ft

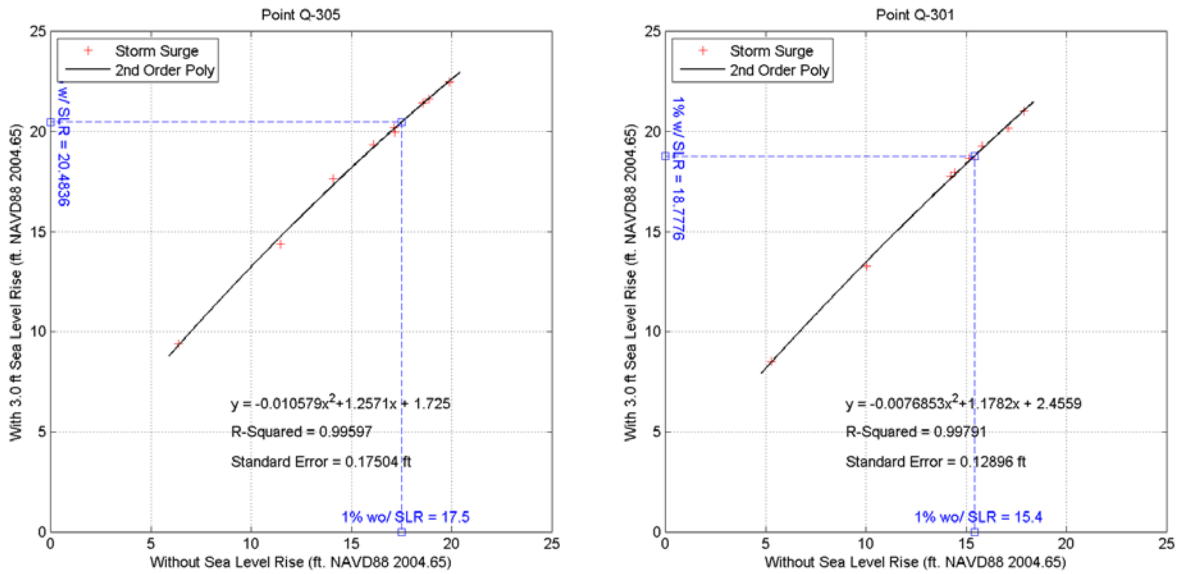


Figure 4-9 Regression Plots for Lake Borgne Area for Original Relative Sea Level Change Runs With Relative Sea Level Change 3.0 ft

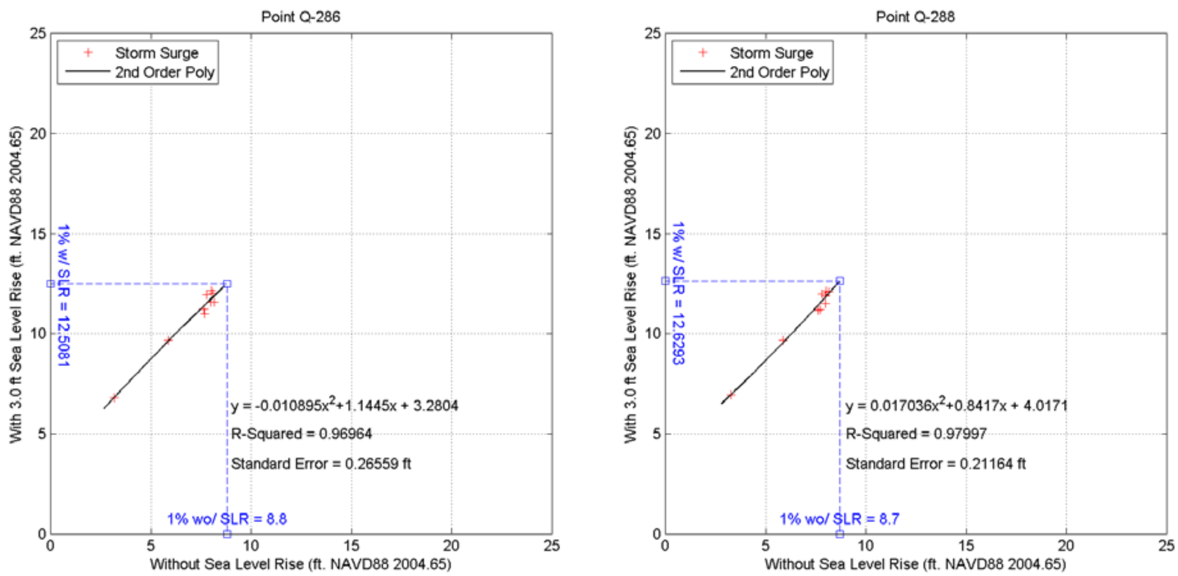


Figure 4-10 Regression Plots for Lake Pontchartrain Area for Original Relative Sea Level Rise Runs With Relative Sea Level Change 3.0 ft

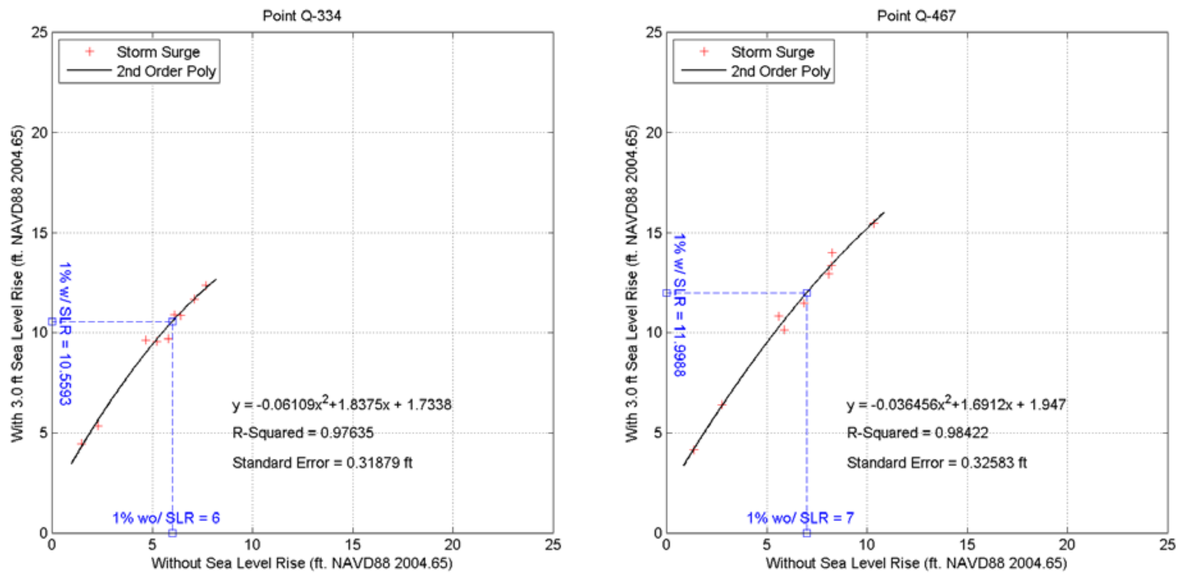


Figure 4-11 Regression Plots for West Bank Area for Original Relative Sea Level Change Runs With Relative Sea Level Change 3.0 ft

The regression analysis of the sea level rise storms provided a surge amplification factor for the entire Q-set for each RSCL scenario. From this set, a number of points were selected for the Lake Pontchartrain area, the Lake Borgne area, and the West Bank area. The following map shows the location of the selected points. **Table 4-4** summarizes the surge amplification factors from all of the RSLC runs evaluated in this analysis.

Table 4-4 Summary of Surge Amplification Factors for All Relative Sea Level Change Scenarios

Surge Amplification Factors for All Relative Sea Level Change Runs						
	RSLC Run Set:	**Original RSLC**	**MTG**	**Original RSLC**	**Original RSLC**	**MTG**
	RSLC Value (ft)	**1.00**	**1.15**	**2.00**	**3.00**	**3.20**
	Point Id					
Lake Pontchartrain	676	0.69	1.29	0.81	0.87	
	562	0.94	1.05	1.00	1.04	0.77
	260	0.97	1.32	1.01	1.06	0.87
	560	0.98	1.31	1.01	1.04	0.83
	286	1.08	1.43	1.19	1.24	0.96
	157	1.20	1.70	1.30	1.32	1.17
	288	1.20	1.57	1.28	1.31	1.01
	556	1.15	1.48	1.15	1.20	0.91
	557	1.14	1.46	1.14	1.19	0.91
	551	1.23	1.70	1.25	1.30	0.93

Surge Amplification Factors for All Relative Sea Level Change Runs						
	RSLC Run Set:	**Original RSLC**	**MTG**	**Original RSLC**	**Original RSLC**	**MTG**
	RSLC Value (ft)	**1.00**	**1.15**	**2.00**	**3.00**	**3.20**
	Point Id					
	550	1.31	1.69	1.35	1.38	0.92
	141	1.24	1.67	1.35	1.35	0.98
	644	1.26	1.63	1.32	1.35	0.83
	645	1.20	1.62	1.24	1.28	0.77
	298	1.29	2.46	1.25	1.25	
Lake Borgne	651	1.04	0.65	1.04	1.10	1.21
	305	0.83	0.70	0.91	0.99	1.14
	304	0.93	0.76	0.98	1.06	1.17
	300	1.07	0.72	1.07	1.13	1.18
	132	1.17	0.76	1.13	1.18	1.16
	139	1.05	0.83	1.07	1.13	1.15
	442	1.26	1.08	1.23	1.23	1.12
	443	1.10	1.64	1.13	1.04	1.13
	353	1.47	1.82	1.29	1.24	1.25
	444	1.37	1.90	1.32	1.11	1.21
	445	1.64	2.25	1.52	1.30	1.36
	447	1.83	2.57	1.48	1.36	1.49
West Bank	719	1.54	1.17	1.84	1.91	1.60
	717	1.84	1.57	2.21	2.20	1.00
	345	1.12	0.96	1.42	1.51	1.04
	467	1.27	1.45	1.61	1.67	1.39
	468	1.07	0.85	1.33	1.42	1.02
	755	1.45	2.02	1.72	1.71	1.55
	754	1.38	1.88	1.67	1.65	1.36
	753	1.27	2.20	1.61	1.66	1.08
	752	1.39	1.99	1.72	1.69	1.02
	745	1.26	2.08	1.62	1.69	1.32
	80	1.30	1.96	1.63	1.68	1.53
	79	1.28	1.81	1.57	1.62	1.56
	334	1.20	1.62	1.47	1.52	1.44
	744	1.17	2.16	1.59	1.69	1.89
	743	1.20	2.32	1.55	1.64	1.95

An average surge amplification factor was determined for each region by averaging the amplification factors factor from all sets of runs for all RSLC scenarios. **Table 4-5** summarizes the average surge amplification factor for each region.

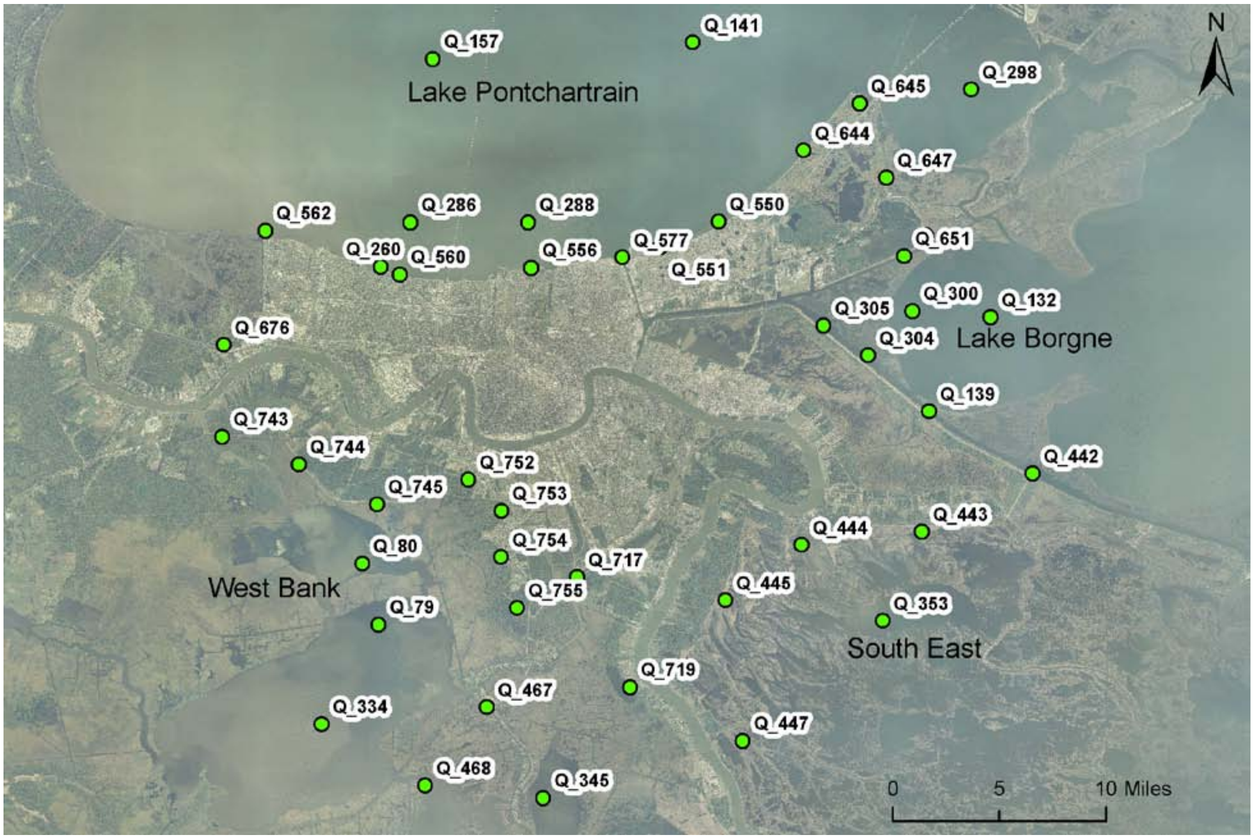


Figure 4-12 Location of Q-set Points used for Average Surge Amplification Factor Calculation

Table 4-5 Average Surge Amplification Factor Computed for Each Region

Area	Points Used in Average	Average Surge Amplification Factor
Lake Pontchartrain	676,562,260,560,286,157,288,556,557,551 550,141,644,645,298	1.20
Lake Borgne	651,305,304,300,132,139,442,443,353,444 445,447	1.22
West Bank	719,717,345,467,468,755,754,753,752,745 80,79,334,744,743	1.55

Table 4-5 shows the low, intermediate and high estimates for RSLC from **Section 3.5** of this report. These values represent the best estimate of RSLC using the guidelines in the EC. These estimates for RSLC are multiplied by the surge amplification factor to determine the increase in surge for each estimate. **Chapter 5.0** discusses how surge amplification due to RSLC impacts the HSDRRS.

Table 4-6 Surge Increase for Three Estimated Relative Sea Level Change Scenarios

Representative Location	2057 RSLC Scenario from Section 3.5 (ft Higher Than Year 2011 Zero)			Surge Increase Due to RSLC (ft)		
	Low	**Intermediate**	**High**	**Low**	**Intermediate**	**High**
Lake Pontchartrain	1.3	1.6	2.8	1.6	1.9	3.4
Lake Borgne	1.5	1.9	3	1.8	2.3	3.7
West Bank	1	1.4	2.5	1.6	2.2	3.9

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5.0 HSDRRS IMPACT ASSESSMENT

5.1 HSDRRS DESIGN

This chapter presents an impact assessment of the different RSLC scenarios on the current HSDRRS design. The HSDRRS design has been conducted after Katrina using new guidelines and prior to the new EC for RSLC. At this moment, most of the designs are complete, and many projects are already under construction. **Table 5-1** summarizes the 1% design elevations for 2060 and the 1% hydraulic boundary conditions (2010) for the three distinct hydraulic sub-basins of the HSDRRS. Note that these numbers are "generalized" numbers. For specific projects reaches, the actual design numbers may be slightly different. For the purpose of this report, however, these numbers have sufficient accuracy to show the impact of RSLC.

Table 5-1 Typical 1% Design Numbers from HSDRRS

Area	2060 Design Elevation (ft)	2010 Conditions		
		Surge Level (ft)	Wave Height Hs (ft)	Wave Period Tm-10 (s)
West Bank	14	7.5	1.5	4.5
Lake Borgne	32	18.5	7.5	7.0
Lake Pontchartrain	16	9.0	7.0	7.0

The 2060 design elevation of the HSDRRS design listed in **Table 5-1** accounts for RSLC. This effect was incorporated as follows (refer to **Chapter 2.0** more details): The RSLC scenario was deducted from the LCA study of 2004. A +1.0 ft RSLC was considered to be adequate for these three areas over a period of 50 years. The 2060 1% surge level was increased with +2.0 ft (West Bank) and +1.5 ft (Lake Pontchartrain and Lake Borgne) because of the amplification effect. The 1% wave height was increased with 50% of the surge level increments for both areas. The wave period was raised assuming constant wave steepness in 2010 and 2060. Based on these 2060 hydraulic boundary conditions, overtopping computations were performed and the 2060 elevation was established to meet the overtopping design criterion.

This chapter builds on the information gathered in the previous chapters in order to assess the impact of the new EC on the HSDRRS designs. **Chapter 3.0** presented the new three different RSLC scenarios for West Bank, Lake Pontchartrain, and Lake Borgne. These RSLC scenarios are repeated in **Table 5-2**. Next, **Chapter 4.0** presented the impact of these RSLC scenarios on the 1% design surge levels because this impact is not necessarily linear. The derived surge amplification factors are also listed in **Table 5-2**. A 1% surge amplification of 1.2 means the 1% surge level increases with 1.2 x the RSLC, and so on. For example, the 1% surge level increase for Lake Pontchartrain in RSLC scenario NRC – 1 equals 1.2 x 1.8 ft = +2.2 ft.

Table 5-2 Relative Sea Level Change and Surge Amplification Based on Chapter 3.0 and 4.0

Area	1% Surge Amplification (Refer to Chapter 4.0)	Relative Sea Level Change 2057 (ft) Refer to Chapter 3.0		
		Historical	NRC - 1	NRC – 2
West Bank	1.6	1.1	1.5	2.7
Lake Borgne	1.1	1.7	2.1	3.3
Lake Pontchartrain	1.2	1.4	1.8	3.0

Using the information from **Chapter 3.0** and **4.0**, this chapter will discuss, for each area in **Section 5.2**, what the (hydraulic) consequences are for the current HSDRRS design. This will be done in two different ways:

- The 1% overtopping rate for the three different RSLC scenarios will be presented in time with the existing HSDRRS design in place. This will indicate how long the current HSDRRS design will be sufficient to meet the 1% criteria under the different RSLC scenarios. It will indicate **when** additional measures will be needed to upgraded the system in order to meet the 1% criteria.

- The 1% surge level will be shown in relation to the current design elevation. This will show at what point in the future the 1% surge will be higher than the design elevation. This information is critical for discussing adaptive measures since **raising** the elevation of the HSDRRS is the ***only*** option at that point. As long as the 1% surge level is below the design elevation, other measures to reduce the wave overtopping in front of the flood defense is **also** an option.

Section 5.3 will discuss possible measures to expand the hydraulic life time.

Note that the analysis in the coming sections assumes that 2060 design height is in place or will be achieved. The floodwalls in the HSDRRS will be built to 2060 elevation, but this is not the case for levees. In between 2010 and 2060, the levees will be raised through various lifts since these cannot be built to 2060 elevations directly.

5.2 HSDRRS DESIGN ASSESSMENT

5.2.1 West Bank

Figure 5-1 presents the 1% overtopping rate for the three different RSLC scenarios in the period 2010 – 2060. This figure is created using the 2060 design height at the West Bank (14 ft) and using the various RSLC scenarios following the EC. The wave characteristics have been adapted following the same procedure as in the original HSDRRS design. Note that the design elevation is assumed to be fixed through time.

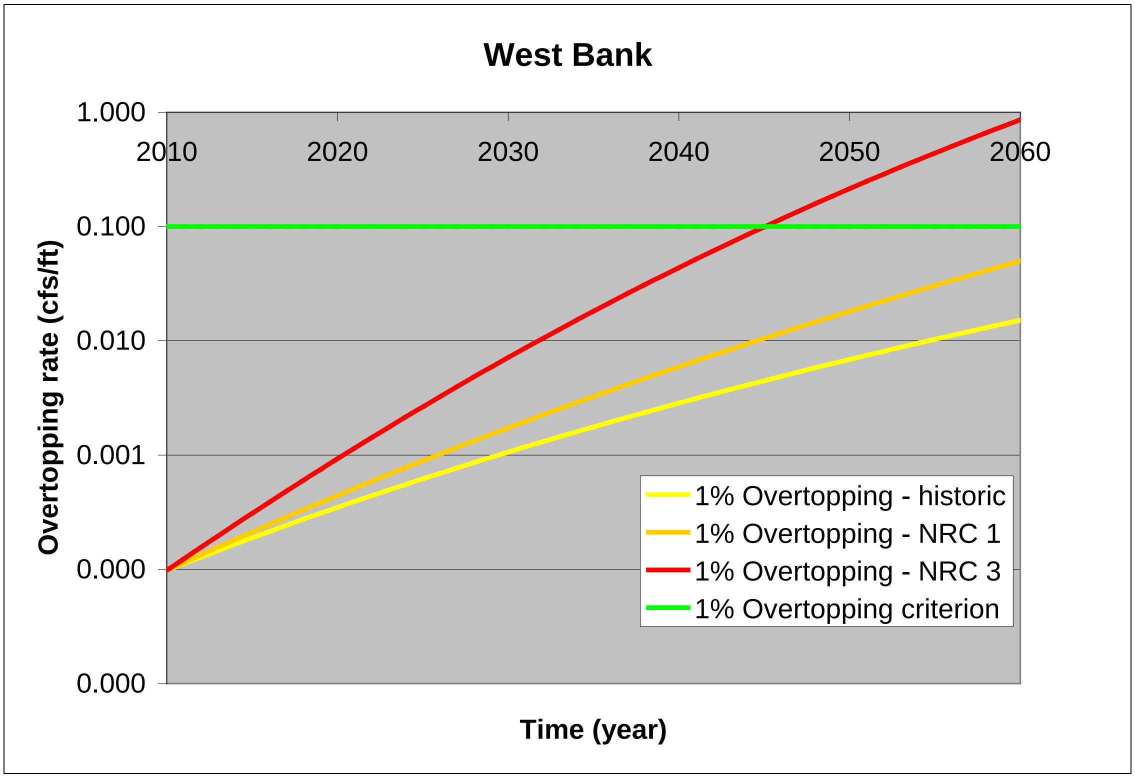

Figure 5-1 1% overtopping rates for West Bank 2010 – 2060 with current HSDRRS design

Figure 5-1 shows that for the current 2060 HSDRRS design height will be sufficient both the historical and the NRC-1 scenario under the current overtopping design criterion of 0.1 cfs/ft. The NRC-3 scenario, however, will result in an exceedence of the overtopping criterion in 2045. It is important to note here that the RSLC scenario applied in the HSDRRS was comparable with the NRC-1 scenario. This explains why both the historical and the NRC-1 scenario are below the overtopping threshold in 2060.

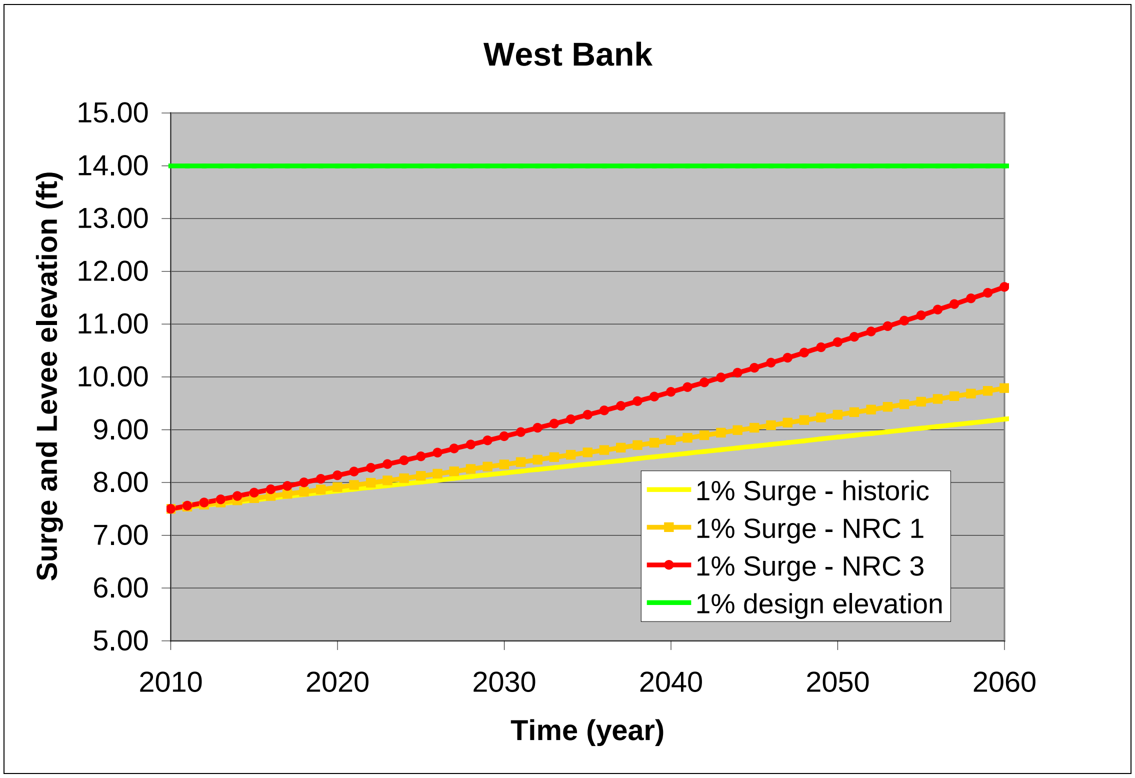

Figure 5-2 1% surge levels and HSDRRS design elevation for West Bank 2010 – 2060

Figure 5-2 shows the temporal development of the 1% surge between 2010 – 2060. The graph shows that the 1% surge will be lower than the design elevation in 2060 for all scenarios. For the worst-case scenario, there is still a freeboard of about 2 ft left in 2060 for this design event.

5.2.2 Lake Pontchartrain

Figure 5-3 presents the 1% overtopping rate for the three different RSLC scenarios in the period 2010 – 2060. This figure is created using the 2060 design height at the Lake Pontchartrain (16 ft) and using the various RSLC scenarios following the EC. The wave characteristics have been adapted following the same procedure as in the original HSDRRS design. Note that the design elevation is assumed to be fixed through time.

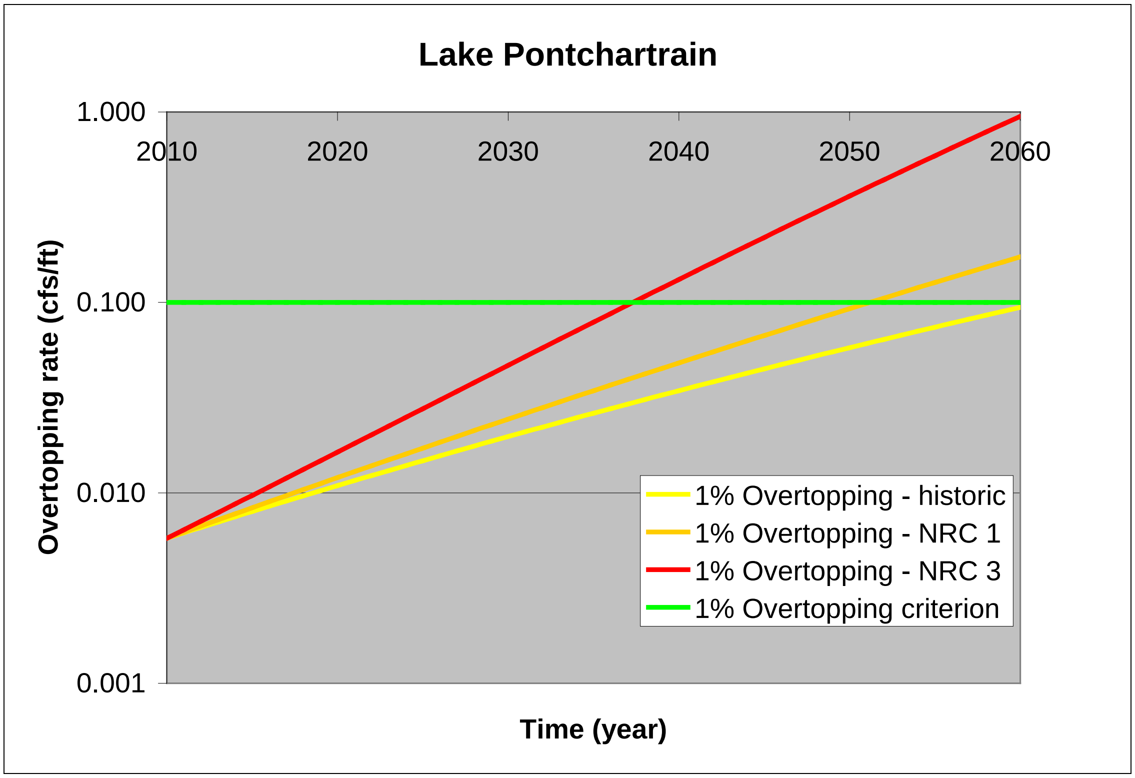

Figure 5-3 1% overtopping rates for Lake Pontchartrain 2010 – 2060 with current HSDRRS design

Figure 5-3 shows that the current 2060 HSDRRS design height will be sufficient for the historical scenarios under the current overtopping design criterion of 0.1 cfs/ft. The NRC-1 and NRC-3 scenario, however, will result in an exceedence of the overtopping criterion in 2050 and 2035, respectively. It is important to note here that the RSLC scenario applied in the HSDRRS was comparable with the historical scenario.

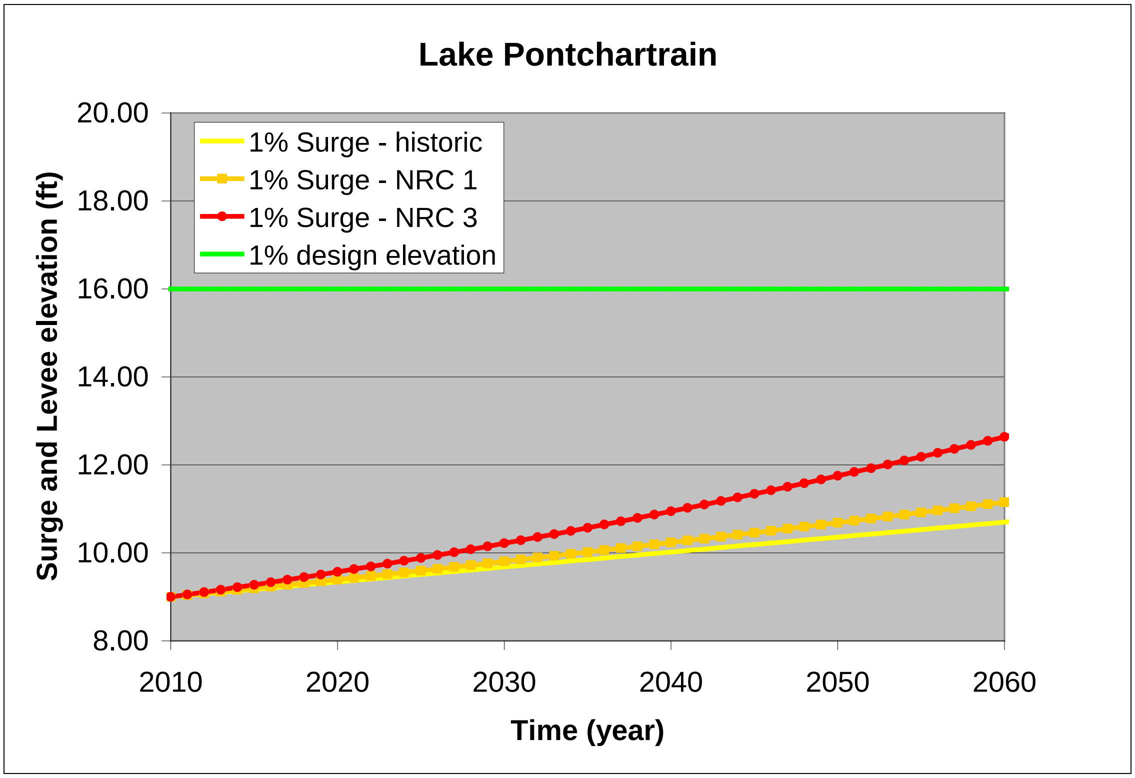

Figure 5-4 1% surge levels and HSDRRS design elevation for Lake Pontchartrain 2010 – 2060

Figure 5-4 shows the temporal development of the 1% surge between 2010 and 2060. The graph shows that the 1% surge will be lower than the design elevation in 2060 for all scenarios. For the worst-case RSLC scenario, there is still a freeboard of about 3.5 ft left in 2060 for this design event.

5.2.3 Lake Borgne

Figure 5-5 presents the 1% overtopping rate for the three different RSLC scenarios in the period 2010 – 2060. This figure is created using the 2060 design height at the West Bank (14 ft) and using the various RSLC scenarios following the EC. The wave characteristics have been adapted following the same procedure as in the original HSDRRS design. Note that the design elevation is assumed to be fixed through time.

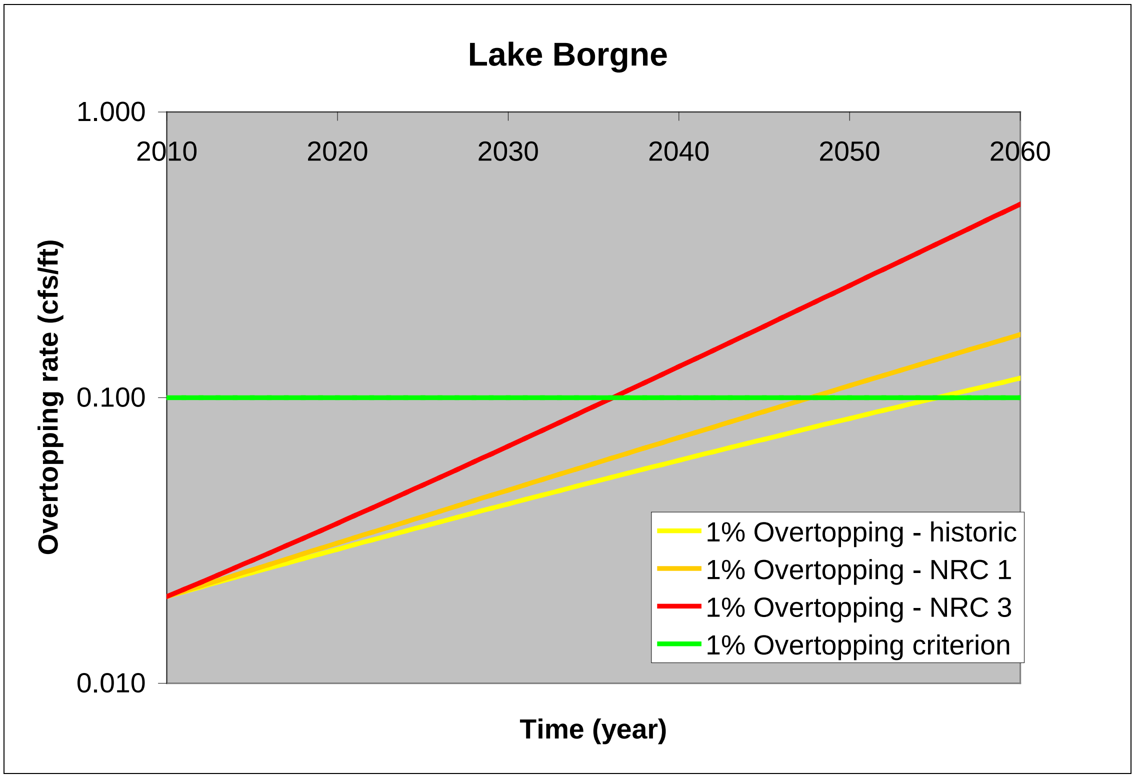

Figure 5-5 1% overtopping rates for Lake Borgne 2010 – 2060 with current HSDRRS design

Figure 5-5 shows that for the current 2060 HSDRRS design height will be sufficient the historical scenario under the current overtopping design criterion of 0.1 cfs/ft. The NRC-1 and NRC-3 scenario, however, will result in an exceedence of the overtopping criterion in 2050 and 2040, respectively. It is important to note here that the RSLC scenario applied in the HSDRRS was comparable with the historical scenario. This explains why the historical scenario matches with the 0.1 cfs/ft line in 2060.

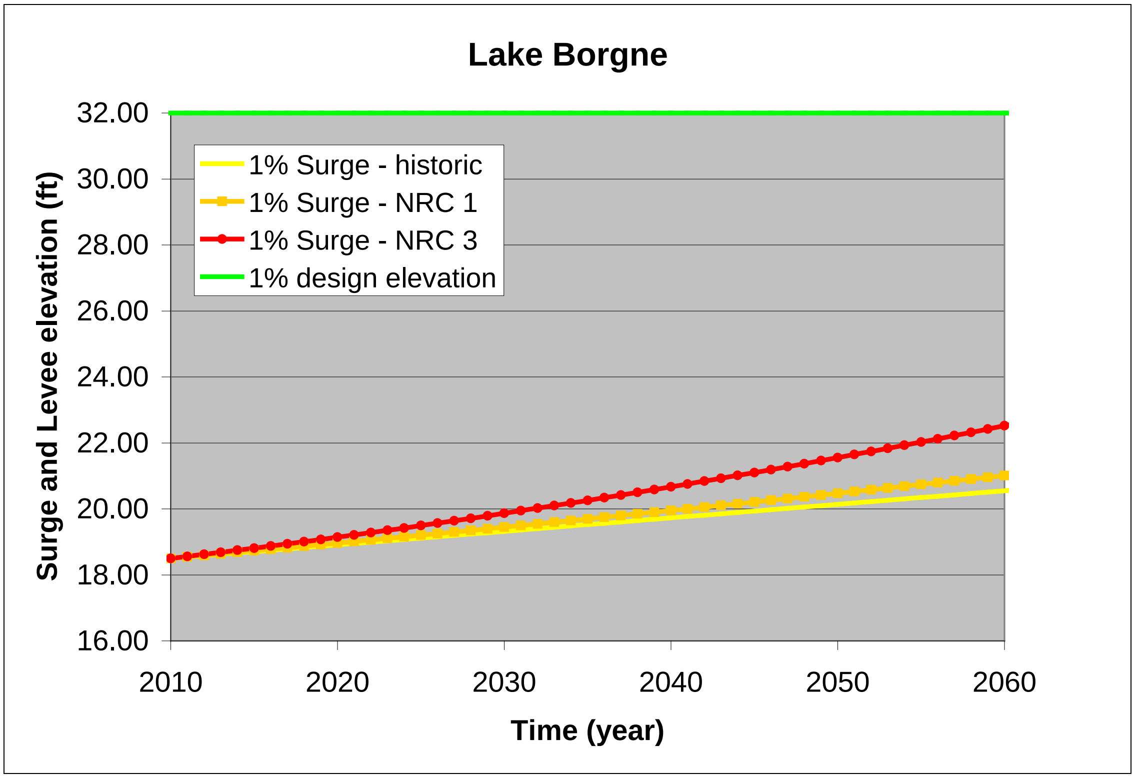

Figure 5-6 1% surge levels and HSDRRS design elevation for Lake Borgne 2010 – 2060

Figure 5-6 shows the temporal development of the 1% surge between 2010 and 2060. The graph shows that the 1% surge will be significantly lower than the design elevation in 2060 for all scenarios. For the worst-case scenario, there is still a freeboard of about 10 ft left in 2060 for the 1% design event. The reason of this large available freeboard is that the waves are high in the Lake Borgne area. Therefore, the difference between the 1% surge level and the design elevation needs to be large to limit the overtopping rates.

5.3 SUMMARY

Table 5-3 summarizes the results presented in the previous sections. It can be concluded from this table that the hydraulic life time is 50 years or higher for the historical sea level change scenario in the entire HSDRRS. If NRC-3 scenario would become true, the lifetime of the HSDRRS system is still at least 30 – 35 years from now (instead of the expected design life time of 50 years). The table also shows that for all RSCL scenarios the current HSDRRS design elevations are high enough to prevent free flow over the levees and floodwalls within 50 years from now.

Table 5-3 Summary of Relative Sea Level Change Impacts

Area	Parameter	Relative Sea Level Change Scenario		
		Historical	NRC – 1	NRC – 3
West Bank	Hydraulic lifetime current design (years)	> 50	> 50	35
	1% freeboard in 2060 (ft)	4.5	4	2
Lake Borgne	Hydraulic lifetime current design (years)	50	40	30
	1% freeboard in 2060 (ft)	11.5	11	10
Lake Pontchartrain	Hydraulic lifetime current design (years)	50	40	35
	1% freeboard in 2060 (ft)	5.5	5	3.5

Definitions:

- Hydraulic lifetime is the year at which the overtopping criterion of 0.1 cfs/ft is just met. Obviously, this year is closer for a more severe RSLC scenario.
- 1% freeboard is the difference between the 1% still water level and the current design elevation in 2060.

5.4 POSSIBLE STRUCTURAL MEASURES

This section discusses several measures which could be taken to expand the hydraulic life time of the construction if a more severe RSLC scenario would become true. The table below discusses various optional measures for this situation including its working principle and some comments about pros and cons of the specific solution.

Table 5-4 Summary of Possible Structural Measures

Measure	Working Principle	Comments
Raise flood defense	Reduce overtopping rate	May be impossible for floodwall without complete re-design; for levees very dependent on local situation
Add artificial armoring at backside	Allow more overtopping	Check on interior flooding is necessary; reduces flexibility for adaptation later on because of hard structural element in flood defense
Add armoring at the front side	Reduce overtopping rate	Reduces flexibility for adaptation later on because of hard structural element in flood defense
Increase wave berm/foreshore	Reduce overtopping rate	Geotechnical stability may be an issue; relatively flexible
Add (or strengthen existing) breakwater	Reduce overtopping rate	---

6.0 CONCLUSIONS AND PATH FORWARD

The objective of this document is to perform an assessment of the hydraulic impact of RSLC on the flood risk reduction projects in the New Orleans area following the new guidance provided in EC1165-2-211. Questions which have been addressed are:

- **Chapter 2.0:** How was the effect of RSLC treated in the various projects prior to this EC?

- **Chapter 3.0:** What are the RSLC scenarios for the New Orleans area based on the new EC?

- **Chapter 4.0:** What is the impact of these new EC scenarios on the design surge levels?

- **Chapter 5.0:** How sensitive are the plans for different RSLC scenarios and what alternatives are available?

This report limits itself to the HSDRRS project in describing the sensitivity for the different RSLC and the available alternatives (**Chapter 5.0**). The conclusions will therefore focus on the HSDRRS program.

The following conclusions can be drawn from this study:

- **Chapter 2.0:** The effect of RSLC was treated differently in the various projects, see for details **Chapter 2.0**. With respect to the HSDRRS program, RSLC has been based on the LCA study (2004). A +1.0 ft RSLC has been applied in the entire HSDRRS project area to determine the 2060 conditions (50-year life span). Because of the non-linear response of the surge level, this has been translated to a +2.0 ft increase of the 1% surge level at the West Bank and a +1.5 ft increase for the Lake Pontchartrain and Lake Borgne area for the 2060 conditions, (USACE, 2007).

- **Chapter 3.0:** Following the EC an analysis of several water level gages in Louisiana is presented to determine the RSLC trend. Based on a trend analysis of a data record, the mean sea level trends are +8.6 mm/year (West End), 6.6 mm/year (Barataria), and 10.2 mm/year (Shell Beach). These rates have been used to generate the three different RSCL scenarios for the three areas in the HSDRRS; Lake Pontchartrain, West Bank, and Lake Borgne. The resulting RSCL at the end project life time of the HSDRRS program (2057) has been summarized in **Table 3.4**.

- **Chapter 4.0:** The effect of RSLC on the 1% design surge level has been assessed using the existing information from ADCIRC runs. Based on these runs, good (and almost linear) correlation has been found between the maximum surge level without and with RSCL. A so-called amplification factor is defined as the 1% surge level increase/sRSCL increase. The following amplification factors have been found: West Bank (1.6), Lake Pontchartrain (1.2), and Lake Borgne (1.1). Using these factors, the 1% design surge level changes have been determined based on the RSLC scenarios. The resulting effect on the design surge levels for 2057 has been summarized in **Table 4.5**.

- **Chapter 5.0:** The impact of the RSLC scenarios has been assessed by estimating the hydraulic lifetime of the current HSDRRS design. In addition to that, the freeboard is determined at the end of the lifetime **(Table 5.3)**. The assessment has been performed using one representative levee section for West Bank, Lake Pontchartrain and Lake Borgne. It is concluded that the hydraulic life time is 50 years or higher for the historical RSLC scenario in the entire HSDRRS. If NRC-3 scenario would become true, the lifetime of the HSDRRS system is still at least 30 – 35 years from now. The table also shows that the current HSDRRS design elevations are high enough to prevent free flow over the levees and floodwalls within 50 years from now for all RSLC scenarios. Although raising the system is an option, a wide variety of other options (e.g. adding roughening, add/increase a breakwater, strengthen inner slope, etc.) are available to meet the hydraulic requirements in more severe RSLC scenarios.

The following path forward is recommended:

- **Data analysis :** TBD

- **ADCIRC analysis:** A limited number of ADCIRC modeling runs which include RSLC were made for the HSDRRS. The effects of RSLC on storm surge and waves can be explored in greater detail with more modeling runs. A complete set of 304 storms that include the best estimate for RSLC would provide more confidence in the values for future conditions. Recently, a set of runs is being completed to evaluate the effects of RSLC on storm surge in the Mississippi River. A set of 70 storms with varying RSLC values and discharges in the Mississippi will be completed to establish future surge and wave values in the river. These storms will also provide valuable information for other parts of the HSDRRS.

- **HSDRRS analysis:** The assessment in this report has been done using one representative section for the West Bank, Lake Pontchartrain, and Lake Borgne. A more detailed analysis is recommended to refine this analysis, and do specific recommendations per reach what alternatives are available to cope with more severe RSLC scenarios. This analysis could also include the issue of levee lifts which will be necessary during the lifetime of the design to keep up with RSLC. In addition to the analysis for the levee/floodwall system, a more detailed analysis of the various barriers and other structures is also recommended.

7.0 REFERENCES

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Appendix P

The I-10 & I-310 Floodwall Overtopping Analysis

(10 Apr 2009)

I-10 & I-310 Floodwall Overtopping Analysis

Study Location

Residents of St. Charles and Jefferson Parish rely heavily upon the ability of the St. Charles Parish levee and West Return Wall system for protection against rising waters of Lake Pontchartrain during hurricane events (figure 1). This protection system consists of a combination of floodwalls and earthen levees running south along the St. Charles and Jefferson Parish line following a path delineating the western portion of Louis Armstrong International Airport as well as the eastern bounds of St. Charles Parish wetlands. Looking on a broader scale, this protection ultimately ties into a much more comprehensive system. West of Interstate 310 the levee system ties into the Bonnet Carre Spillway east guide levee while north of Interstate 10 it ties to the Lake Pontchartrain floodwall system.

Levee Design Height Concerns

As shown in figure (1) the levee system runs beneath a series of ramps and loops for both I-10 and I-310. The 1% chance exceedence design standards call for an increase in protection height throughout most of surrounding levee systems. However, floodwall clearance has become an issue since overhead structures limit the amount of increase in construction height. The following are main concerns at each location (Figures 2 and 3).

- I-10 Bridge
    - East and West Bound Lanes

- I-310 Bridge
    - North and South Bound Lanes

Table (1) summarizes the 1% chance exceedence and proposed future condition design heights at each study location. Future conditions were established as 1.5 ft. higher than existing condition elevations as mandated in guidelines. All elevations in this report reference the NAVD 88 (2004.65) datum.

Methodology

An overtopping analysis was performed at each location to quantify overtopping volumes and magnitude resulting from floodwall height limitations. Overtopping calculations were determined based on a given surge response from 1% and 0.2% chance exceedence future condition (2057) events. Points initially had to be chosen from the ADCIRC grid to specify the location for extracting surge hydrograph data. ADCIRC point 242 near I-310 and point 220 near I-10 were the points from the D1479 dataset closest to each study location. If for some reason the hydrograph for a particular point was not continuous throughout the storm the hydrograph was extracted from the next closest D1479 point. The analysis was run with a few storms selected out of the entire 152 synthetic east storm dataset. Storms 53, 61, and 77 were chosen since each had characteristics approximately equaling a 1% chance exceedence surge event. In other terms, the single storm SWL and the statistical 100 year SWL were approximately equal for the specific storm and point. Using multiple storms allowed for a range of storm surge durations to be analyzed.

To artificially generate a surge response for 1% and 0.2% chance exceedence future condition still water elevation (SWE) a normalization technique was implemented. To detail the process, storm 53 and ADCIRC point 220 near the I-10 study area will be used as an example. At the start, the surge hydrograph for this storm and point was extracted from ADCIRC output. At the I-10 location, the hydrographs were extracted from point 220. However, I-310 hydrographs were extracted from point 240 instead of point 242 for all storms (Table 2). The replacement is valid since for all storms the difference in surge between points 242 and 240 is less than 0.3 ft. at any point on the hydrograph. Moreover, the average surge difference over all storms is less than 0.1 ft. The peak surge produced by storm 53 was used to normalize 1% or 0.2% chance exceedence future condition SWE. The basic formula used to normalize the 1% chance exceedence future condition SWE is shown below.

$$factor = {SWE_{1\%}} \Big/ {SWE_{peak\ surge\ (individual\ \ storm)}}$$

After the factor was calculated, it was multiplied to each surge value in the storm surge hydrograph. The result was a future condition surge hydrograph with a 1% chance exceedence SWE peak and similar surge duration as that produced in storm 53. A plot of both hydrographs corresponding to this example is shown in figure (4). Each of the storms in our analysis was used to develop 1% and 0.2% chance exceedence future condition hydrographs at both study locations. In all, a total of twelve synthetic hydrographs were developed through this technique.

If the still water elevation (SWE) rises above the floodwall free flow will occur. A general equation was used to calculate free flow. The broad crested weir equation is defined as follows:

$$Q_{free\ flow} = C_d \times L \times H^{3/2}$$

, where Q is free flow in cfs, C_d is the weir overtopping coefficient in $ft^{0.5}$/sec., L is the length of the weir crest perpendicular to flow in ft., and H is the head of water above the weir crest in ft.

In addition to free flow, wave overtopping is critical. A wave overtopping formula was also applied in addition to the weir flow equation to calculate flow rate solely due to wave splash over. The overtopping formula was developed by Franco and Franco (1999) and is defined as follows:

$$Q_{wave} = 0.082 \times \sqrt{gH_s^{\ 3}} \times \exp\left(-(\mu_b - \sigma_b) \times \left(R_c \big/ H_s \right) \times \left(1 \big/ \gamma_b \gamma_s \right) \right)$$

, where Q is the wave overtopping in cfs/ft, g is acceleration due to gravity in ft/sec^2, H_s is the significant wave height in ft., mean and standard deviation of dimensionless coefficient *b*, R_c is the freeboard in ft., γ_b is a wave obliquity reduction factor, and γ_s is a geometry correction factor.

Most of the variables in the weir flow and overtopping formulas were kept constant for both I-310 and I-10 analyses. Below is a list of those constant variables.

- Weir coefficient (C_d): 3.1 ft$^{0.5}$/sec.
- Gravity (g): 32.19 ft/sec^2
- Significant wave height (H_s): 3.0 ft.
- Coefficent *b* mean and standard deviation (μ, σ): 3 and 0.26
- Wave obliquity reduction factor (γ_b): 0.83
- Geometry correction factor (γ_s): 1.0

The wave overtopping mean and standard deviation were based on Franco and Franco (1999) empirical laboratory results and are applied to the equation in a conservative manner. In addition, a wave overtopping rate of 2 cfs/ft. was set as a ceiling threshold for the maximum amount of wave overtopping possible per linear foot. This wave overtopping rate is set equivalent to the overtopping rate when the water level is equal to the top of the floodwall. In calculation of weir flow, the head of water above the floodwall constantly varied. Also, freeboard was continuously adjusted in the wave overtopping equation based on increase or decrease in SWE.

Another critical variable used in calculating overtopping volume was the length of floodwall vulnerable. The length had to be as precise as possible to ensure accurate calculation of overtopping volume. Effective flow length was also examined in the I-10 study since girders supporting the interstate spans obstructed flow over the floodwall. If multiple girders were found to obstruct flow, 1 ft. per girder was subtracted from the effective flow length.

Overtopping Results

I-310 Analysis

The profile drawing in figure (5) was provided by structures branch. Overtopping analyses were done using the values in table (3) along with the aforementioned constant values. In the table scenarios 1 and 2 correspond to the proposed floodwall design heights with 1% and 0.2% chance exceedence future condition SWE, respectively. Scenario 3 analysis is based on the 1% chance exceedence design floodwall height with the 0.2% chance exceedence future condition SWE. A summary of the final I-310 overtopping results for all scenarios are provided in tables (5-7). Total overtopping volumes for all scenarios were based on 46.5 hours of storm surge even though starting and ending times may be different between storms. Refer back to table (2) for starting and ending times of each storm event.

I-10 Analysis

Based on the I-10 profile drawing in figure (6), the values in table (4) were applied in the analysis. Just as in the I-310 setup, scenarios 1 and 2 correspond to analysis done using proposed design heights along with 1% and 0.2% chance exceedence future condition SWE. However, gaps between the floodwall and girders presented an issue at the I-10 location. Due to this, the analysis was separated into overtopping above floodwall and flow through gaps between floodwall and girders. Both flows were calculated and summed together in the results table. Scenario 3 provides an analysis based on a 1% design height without gaps in the floodwall and the 0.2% chance exceedence future condition SWE. Part 2 of scenarios 1 and 2 was simplified by adding the gap areas together instead of calculating flow through each gap separately. A summary of the final I-10 overtopping results for all scenarios are provided in tables (5-7). As with the I-310 study, overtopping was based on 46.5 hours of storm surge even though times may be shifted between storm events.

Summary

The scope of the I-310/I-10 study was to determine magnitude of overtopping resulting from floodwall elevations built to elevations less than 1% chance exceedence design standards due to clearance constraints. Multiple storms (53, 61, and 77) were used in the analysis to examine the effect surge duration has on overtopping volume. Based on selected storms, 1% and 0.2% chance exceedence future condition (2057) hydrographs were generated. Free flow and overtopping were calculated based on the broad crested weir equation as well as the wave overtopping equation by Franco and Franco (1999).

Overtopping results indicate limited differences between total volumes in each scenario. However, running other storms having approximate 1% chance exceedence characteristics might provide larger differences in total volume. Given that total overtopping volumes are in the millions of cubic feet with maximum overtopping reaching over 8 cfs/ft in the I-310 area for one of the 0.2% chance exceedence scenarios, further study should be done to determine whether or not the system can adequately handle such a large volume of water. In addition, armoring should be re-examined at both study locations to ensure size of armoring material is sufficiently large enough to withstand the amount of increase in overtopping.

References

Franco L., and Franco C., 1999. Overtopping Formulas for Caisson Breakwaters with nonbreaking 3D Waves. Journal of Waterway, Port, Coastal, and Ocean Engineering, Vol. 125, No. 2, ASCE (1999), pp. 98-108.

	Future Conditions (2057)			
	1% design height (ft)	Proposed design height (ft)	1% surge (ft)	0.2% surge (ft)
I-310	15.5	13.5	12	15.1
I-10	16	16*	11.8	14.7

Table 1: 1% chance exceedence and proposed design heights at I-310 and I-10 locations. 1% and 0.2% chance exceedence SWE are also shown. *Proposed design height includes gaps in floodwall at each girder.

Storm 53			Storm 61			Storm 77		
Time (hrs)	I-310 SWL (ft)	I-10 SWL (ft)	Time (hrs)	I-310 SWL (ft)	I-10 SWL (ft)	Time (hrs)	I-310 SWL (ft)	I-10 SWL (ft)
78.5	1.682	1.947	71.5	1.415	1.655	121.5	1.66	1.651
79	1.707	1.97	72	1.426	1.681	122	1.665	1.661
79.5	1.733	1.998	72.5	1.446	1.709	122.5	1.67	1.673
80	1.761	2.037	73	1.464	1.733	123	1.675	1.679
80.5	1.79	2.077	73.5	1.487	1.76	123.5	1.68	1.683
81	1.82	2.126	74	1.51	1.788	124	1.686	1.689
81.5	1.852	2.179	74.5	1.541	1.816	124.5	1.692	1.694
82	1.887	2.236	75	1.564	1.843	125	1.698	1.694
82.5	1.924	2.292	75.5	1.587	1.871	125.5	1.704	1.695
83	1.963	2.342	76	1.611	1.899	126	1.71	1.698
83.5	2.006	2.379	76.5	1.636	1.927	126.5	1.716	1.702
84	2.052	2.426	77	1.661	1.959	127	1.722	1.705
84.5	2.101	2.496	77.5	1.688	1.999	127.5	1.728	1.708
...	...	...	...	...	...	...	...	...
97.5	7.299	7.397	92	6.6	7.505	141	2.946	6.454
98	8.054	7.919	92.5	7.49	7.941	141.5	3.52	7.03
98.5	8.807	8.512	93	8.294	8.535	142	4.753	7.423
99	9.612	9.158	93.5	9.058	9.197	142.5	6.785	8.009
99.5	10.404	9.853	94	9.768	9.837	143	8.95	9.055
100	11.257	10.563	94.5	10.396	10.413	143.5	10.483	10.239
100.5	11.909	11.027	95	10.994	10.972	144	11.531	11.125
101	12.269	11.222	95.5	11.498	11.473	144.5	12.126	11.565
101.5	12.299	11.14	96	11.933	11.89	145	12.174	11.574
102	12.027	10.792	96.5	12.195	12.099	145.5	11.843	11.265
102.5	11.553	10.505	97	12.099	11.922	146	11.365	10.865
103	10.825	9.805	97.5	11.495	11.295	146.5	10.85	10.427
103.5	9.967	8.993	98	10.462	10.388	147	10.361	10
104	9.108	8.173	98.5	9.343	9.278	147.5	9.831	9.56
104.5	8.311	7.418	99	8.495	8.293	148	9.33	9.105
105	7.599	6.736	99.5	7.791	7.525	148.5	8.844	8.657
105.5	7.023	6.047	100	7.235	6.892	149	8.404	8.243
106	6.606	5.356	100.5	6.801	6.26	149.5	7.995	7.844
106.5	6.3	4.752	101	6.467	5.65	150	7.614	7.47
107	6.03	4.339	101.5	6.167	5.153	150.5	7.194	7.044
...	...	...	...	...	...	...	...	...
119	7.944	7.963	112	6.198	6.38	162	4.741	4.743
119.5	7.956	7.949	112.5	6.287	6.383	162.5	4.719	4.715
120	7.958	7.938	113	6.339	6.388	163	4.698	4.691
120.5	7.943	7.92	113.5	6.367	6.401	163.5	4.678	4.665
121	7.911	7.881	114	6.379	6.416	164	4.653	4.627
121.5	7.858	7.825	114.5	6.392	6.391	164.5	4.623	4.592
122	7.795	7.764	115	6.389	6.352	165	4.596	4.575
122.5	7.735	7.708	115.5	6.363	6.303	165.5	4.575	4.565
123	7.685	7.659	116	6.325	6.263	166	4.557	4.547
123.5	7.647	7.625	116.5	6.288	6.239	166.5	4.538	4.509
124	7.619	7.593	117	6.256	6.219	167	4.513	4.47
124.5	7.586	7.555	117.5	6.227	6.187	167.5	4.484	4.435
125	7.539	7.506	118	6.199	6.148	168	4.456	4.407

Table 2: Storm surge hydrographs for specified points near I-310 and I-10. Storms used in the analysis are 53, 61, and 77. Blank lines (with dots) indicate gaps in hydrograph. Grey values indicate peaks in surge.

I-310 Study Area	Scenario 1 (1% SWE)	Scenario 2 (0.2% SWE)	Scenario 3 (0.2% SWE)
Foodwall Height (ft)	13.5	13.5	15.5
Length of Floodwall (ft)	100	100	100
Number of Girder Obstructing Flow	10	10	10
Effective Length of Floodwall (ft)	90	90	90
Still Water Elevation (ft)	12.0	15.1	15.1

Table 3: Summary of scenarios implemented within the analysis for I-310 area.

I-10 Study Area	Scenario 1 (1% SWE)		Scenario 2 (0.2% SWE)		Scenario 3 (0.2% SWE)
	Part 1	Part 2	Part 1	Part 2	
Foodwall Height (ft)	16.0	13.5	16.0	13.5	16.0
Length of Floodwall (ft)	195	4	195	4	195
Number of Girder Obstructing Flow	0	0	0	0	0
Effective Length of Floodwall (ft)	195	4	195	4	195
Still Water Elevation (ft)	11.8	11.8	14.7	14.7	14.7

Table 4: Summary of scenarios implemented within the analysis for I-10 area.

1% Chance Exceedence Event Future Conditions (2057)

		Top of Floodwall (ft)	Storm	SWE (ft)	Total Overtopping Volume (ft^3)	Max Overtopping Volume (cfs/ft)
I-310	1%	13.5	53	12.0	3.7E+05	0.46
			61	12.0	3.3E+05	0.46
			77	12.0	3.3E+05	0.46
I-10		16	53	11.8	5.9E+04	0.40
			61	11.8	5.0E+04	0.40
			77	11.8	5.3E+04	0.40

Table 5: 1% chance exceedence results for storms 53, 61, 77 at I-310 and I-10 locations with the proposed wall heights below the bridges.

0.2% Chance Exceedence Event Future Conditions (2057)

		Top of Floodwall (ft)	Storm	SWE (ft)	Total Overtopping Volume (ft^3)	Max Overtopping Volume (cfs/ft)
I-310	0.2%	13.5	53	15.1	6.5E+06	8.27
			61	15.1	5.9E+06	8.27
			77	15.1	5.8E+06	8.27
I-10		16	53	14.7	1.1E+06	6.65
			61	14.7	9.8E+05	6.65
			77	14.7	1.1E+06	6.65

Table 6: 0.2% chance exceedence results for storms 53, 61, 77 at I-310 and I-10 locations with the proposed wall heights below the bridges.

0.2% Chance Exceedence Event Future Conditions (2057)

		Top of Floodwall (ft)	Storm	SWE (ft)	Total Overtopping Volume (ft^3)	Max Overtopping Volume (cfs/ft)
I-310	0.2%	15.5	53	15.1	1.1E+06	1.56
			61	15.1	9.6E+05	1.56
			77	15.1	9.5E+05	1.56
I-10		16	53	14.7	9.1E+05	0.58
			61	14.7	7.9E+05	0.58
			77	14.7	8.5E+05	0.58

Table 7: 0.2% chance exceedence results for storms 53, 61, 77 at I-310 and I-10 locations. Floodwall elevations increased for both study areas to the adjacent 1% elevations.

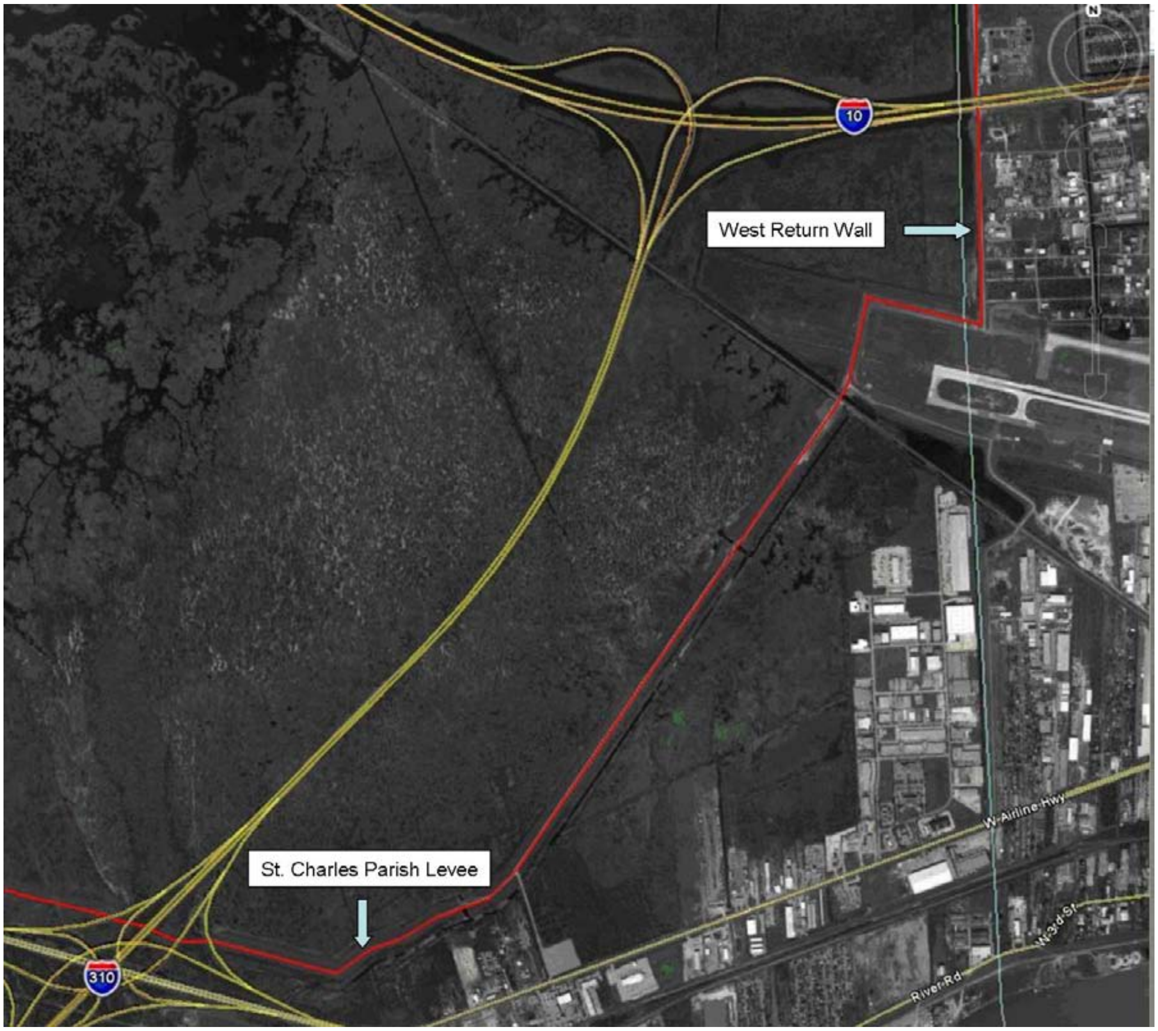

Figure 1: (Red line) St. Charles Parish and West Return Wall Levee System (Google Earth imagery)

Figure 2: (Blue circle) I-310 area of concern (Google Earth imagery).

Figure 3: (Blue circle) I-10 area of concern (Google Earth imagery).

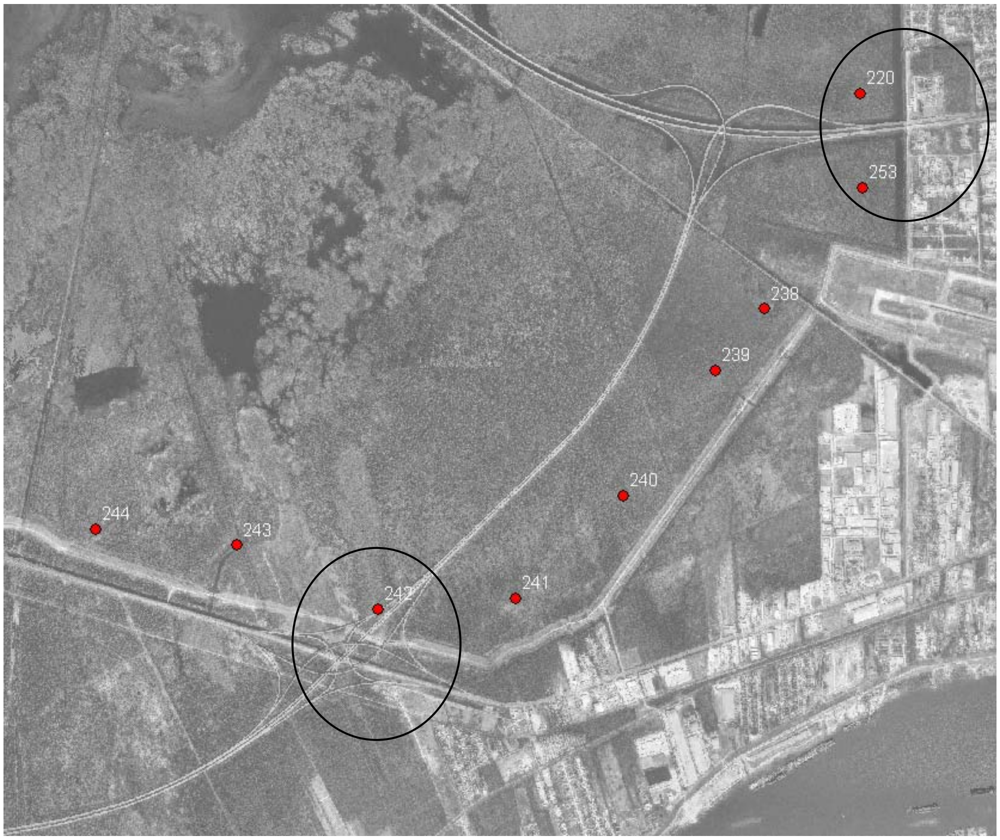

Figure 4: Locations of points with the D1479 point set. As stated in the report, point 241

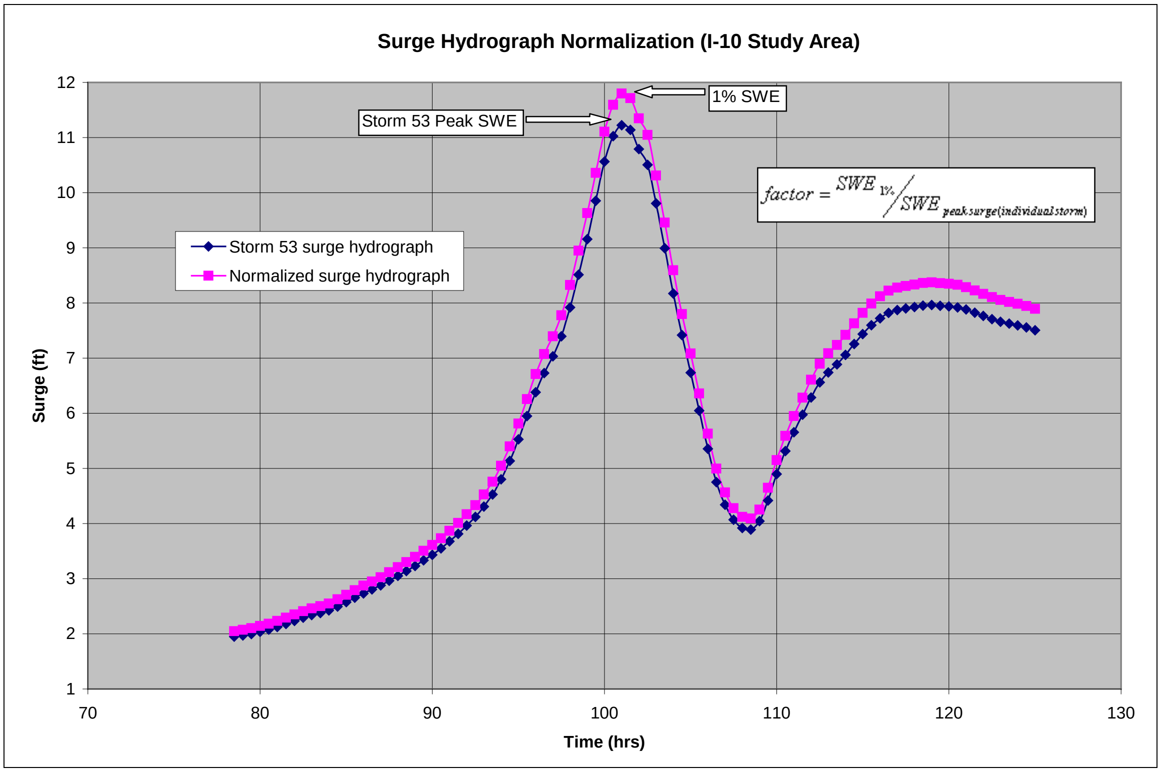

Figure 5: Example of the surge hydrograph normalization technique using storm 53 – ADCIRC pt. 220 near the I-10 study area.

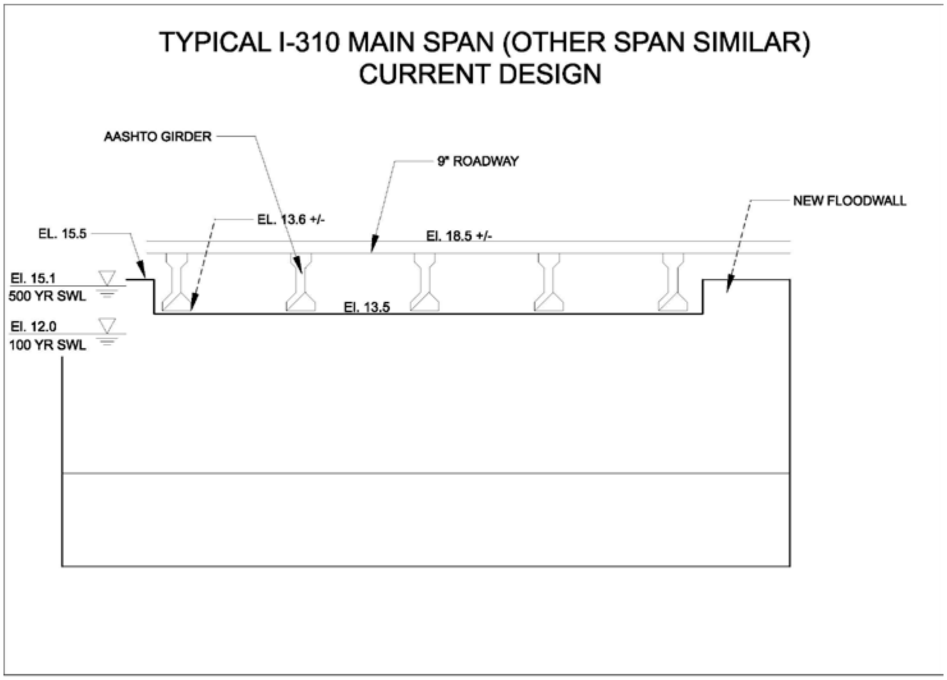

Figure 6: I-310 Span and Floodwall Design. 1% chance exceedence and proposed floodwall design heights are shown along with the 1% and 0.2% chance exceedence SWE.

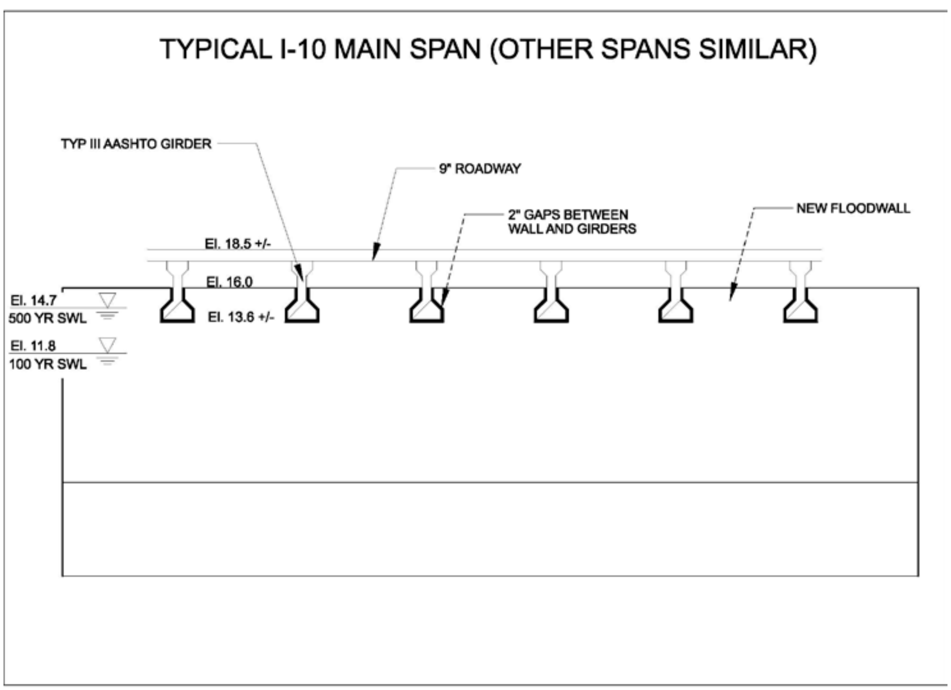

Figure 7: I-10 Span and Floodwall Design. 1% chance exceedence and proposed floodwall design heights are shown along with the 1% and 0.2% chance exceedence SWE.

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Appendix Q

Investigation of ADCIRC Surge Results in St. Charles Parish

(16 May 2008, Authors: H. Roberts, Arcadis; J. Atkinson, Ayres Associates; J. Westerink and H. Westerink. University of Notre Dame)

Introduction

Upon reviewing the FEMA study results in the region of interest, it was observed that the historical storms used in the validation process reported higher surge levels than available gauge data for the St Charles parish region. The model input and output were further evaluated in order to consider whether the results are reasonable and representative. This analysis is intended to summarize the flow features in the St Charles region, to explain how the regional characteristics are implemented in the ADCIRC model, and to describe how the computed peak surge elevations changed after modifying the ADCIRC model. It was found that making scientifically defendable enhancements to the model resulted in only modest reductions in peak storm surge values.

Region of Interest

In St. Charles Parish, Louisiana, a flood protection levee exists on the north side of Airport Highway which runs on an south-east to north-west alignment on the southern edge of a large marsh region. The marsh extends northward from Airport Highway to Lake Pontchartrain. The marsh is mostly natural with the exception of several canals and the crossing of U.S. Interstate 10 and the CSX Railroad, both running roughly parallel to the levee at Airport Highway. The interstate is elevated on piers for the entire crossing of the marsh and does not significantly affect surge in or across the marsh. In contrast, the railroad is a significant hydraulic barrier. Built as a solid gravel bed with the rails at approximately 7ft NAVD88, there are only five small openings through the railroad across the marsh. On the east, the marsh terminates at the Kenner flood protection levee. On the west, the marsh ends at the Bonnet Carre spillway. The marsh region south of Lake Pontchartrian and confined between the three levees on the east, south, and west is the primary region of interest for this study (see Figure 1). This marsh region is roughly 30 square miles and influences the propagation of surge from Lake Pontchartrain to the levee at the Airport Highway.

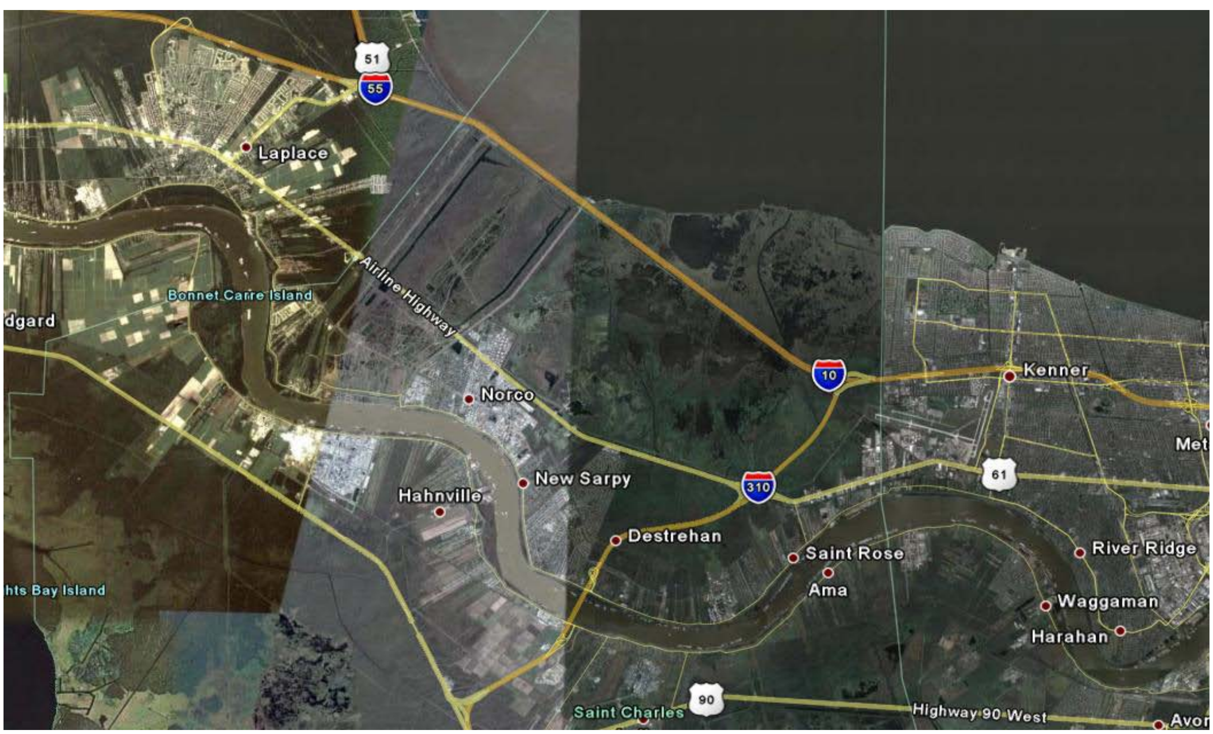


Figure 1 Area of interest

Performance of the Original FEMA Study ADCIRC Model

The available data in the region is limited to several High Water Marks (HWM) from Hurricane Katrina and a USGS Gauge at Bayou LaBranche. Comparisons of the ADCIRC hindcast to the Katrina HWM are shown in Figure 2 through Figure 6. At Bayou LaBranche on the western edge of the marsh, the ADCIRC Katrina surge is 1.64 ft higher than the HWM. At the New Orleans airport, the ADCIRC surge is 4.43 ft higher than the HWM. West of the Bonnet Carre spillway, the ADCIRC Katrina surge is nearly identical to the HWM and as you move to the east of the marsh toward the causeway, the ADCIRC Katrina results under-predict the HWM.

Several explanations are offered to describe the observed behavior. At Bayou Labrance, the HWM is derived from a USGS gauge that is behind the CSX railroad which suggests that the ADCIRC model is not properly accounting for flow through the railroad openings. In addition, there is a restoration area with a constructed levee and new dense vegetation just lakeside of the railroad near the gauge. While the ADCIRC model has the appropriate topography for the restoration area, the land cover data sets used to automate the application of frictional parameters designates the region as open water. Since the friction values are derived from the land cover data sets via an automated procedure, the ADCIRC friction parameter may be low, resulting in locally increased surge.

At the New Orleans airport, a large part of the discrepancy between the model and the HWM is due to a catastrophic blowout that occurred to the levee system during Hurricane Katrina. This blowout created a large opening through which water flowed south, locally reducing the maximum high water. However, the ADCIRC model employs static topography and levee heights, thus dynamic changes in topography were not included and the water levels were computed with an intact levee system.

While that may account for a significant portion of the difference between the computed and actual HWM adjacent to the airport, addition questions were raised regarding the depths in the open water regions of the marsh and the impact depth has on wind setup. During some time intervals of the Katrina hindcast, there is a very large fetch over which the wind drives water toward the airport. The impact of the wind stress on the momentum balance of the water column is inversely proportional to depth. In the case that the depths in the ADCIRC model are unrealistically shallow, the wind setup contribution to the peak surge is too high. Topographic heights for the original FEMA mesh were obtained from LSU's Atlas Lidar. However, lidar does not provide elevations below water. Hence area averaged elevations would be potentially lower when averages include bathymetric values, potentially rendering lower surge values. An additional topographic concern was the depiction of the CSX railroad. Surge in the region is greatly influenced by the overtopping height of the railroad. Thus it is critical to adequately represent the railroad's height and openings in the ADCIRC model.

Three areas of investigation were outlined for evaluating the representation of the original ADCIRC model. First, actual bathymetric survey would be obtained and compared to the existing ADCIRC model. Second, additional resolution would be added in the region near the CSX railroad to allow smaller scale details to be captured, particularly openings at channel crossings. In addition, the elevation of the railroad crest would be checked in the field and in the model. Finally, the assignment of Manning's n value would be explored to ensure that appropriate values were used in the model. It was desired to determine justifiable changes to the ADCIRC model and the effect those changes would have on the computed surge.

ADCIRC High Water Marks

<--Lower -3.0 -1.5 0.0 1.5 3.0 Higher-->

Katrina_sl15v3_uthr18_r8bb_EVM_Wave070621_AVGTAU0_070622

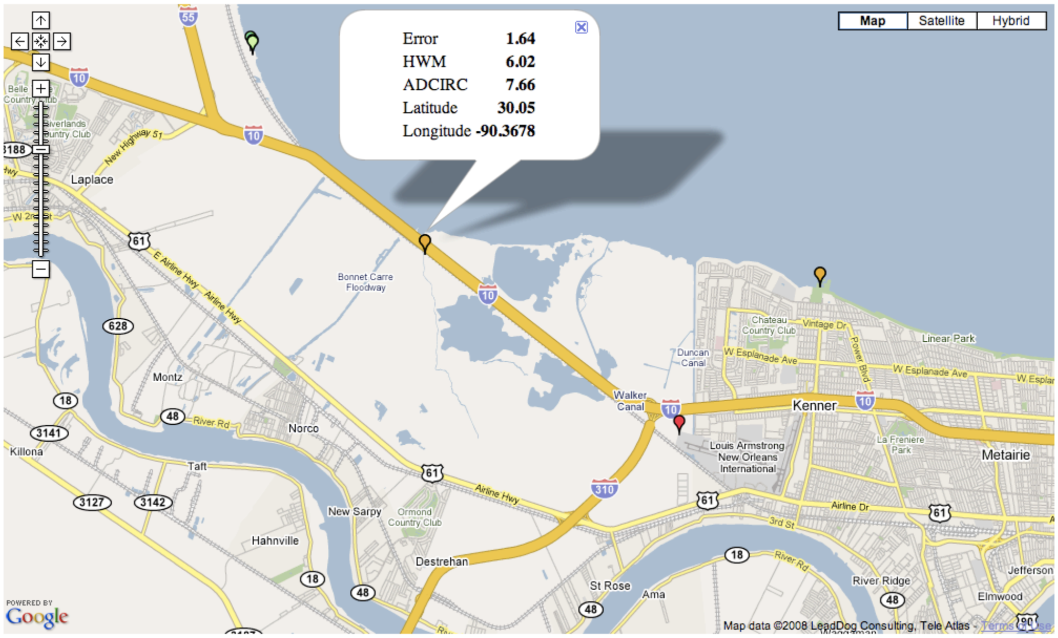


Figure 2 Comparison of ADCIRC Katrina surge and HWM

ADCIRC High Water Marks

<--Lower -3.0 -1.5 0.0 1.5 3.0 Higher-->

Katrina_sl15v3_uthr18_r8bb_EVM_Wave070621_AVGTAU0_070622

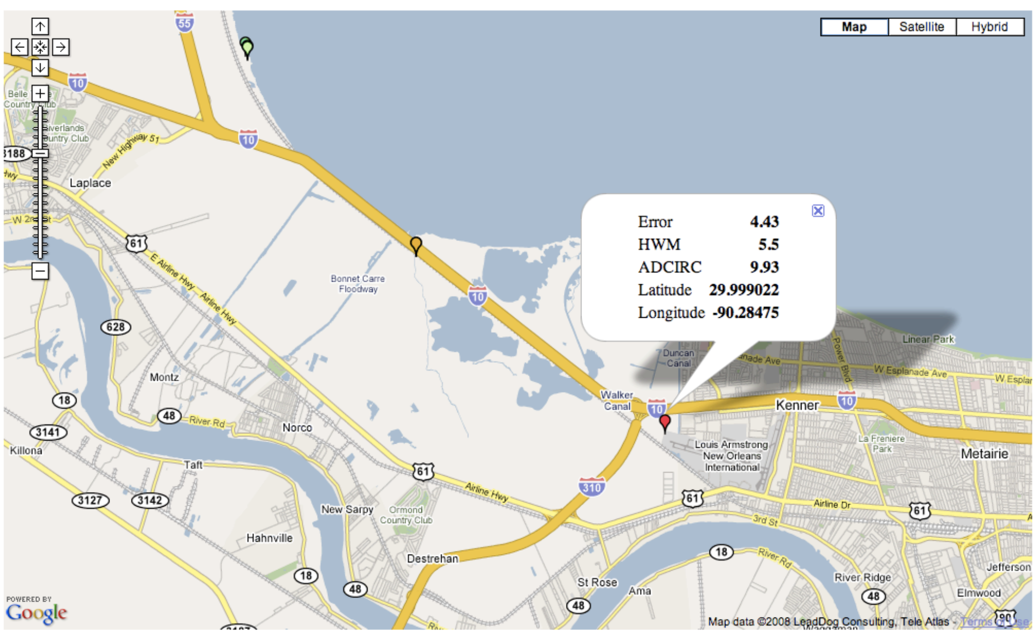


Figure 3 Comparison of ADCIRC Katrina surge and HWM

ADCIRC High Water Marks

<--Lower -3.0 -1.5 0.0 1.5 3.0 Higher-->

Katrina_sl15v3_uthr18_r8bb_EVM_Wave070621_AVGTAU0_070622

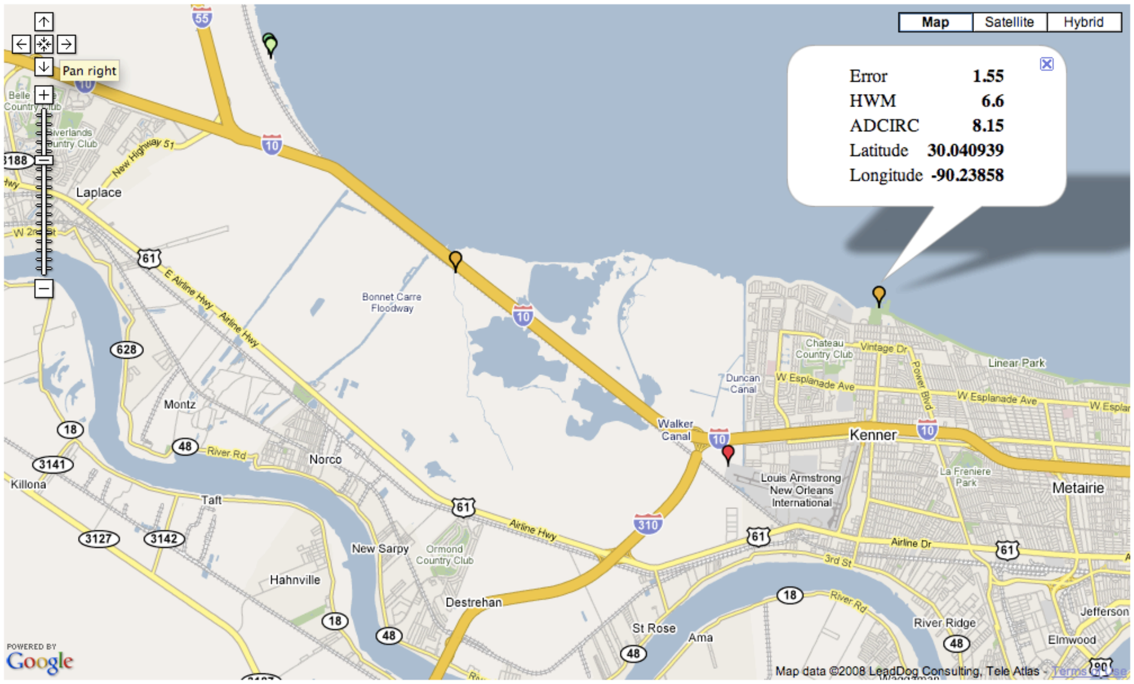


Figure 4 Comparison of ADCIRC Katrina surge and HWM

ADCIRC High Water Marks

<--Lower -3.0 -1.5 0.0 1.5 3.0 Higher-->

Katrina_sl15v3_uthr18_r8bb_EVM_Wave070621_AVGTAU0_070622

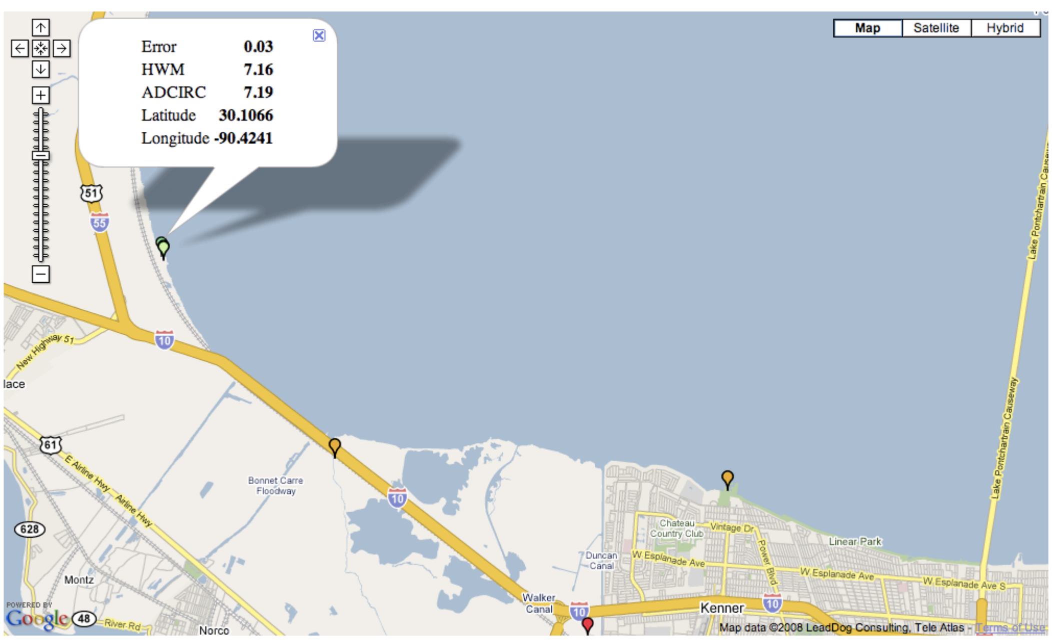


Figure 5 Comparison of ADCIRC Katrina surge and HWM

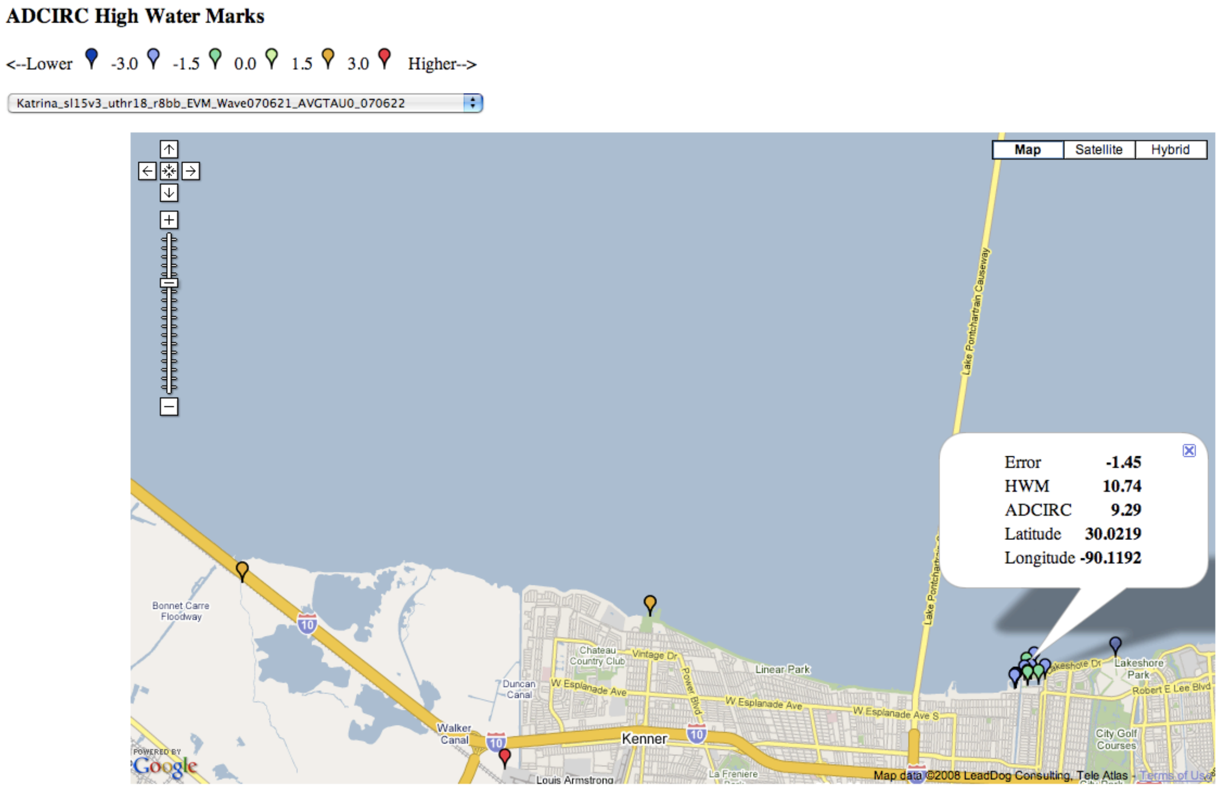


Figure 6 - Comparison of ADCIRC Katrina surge and HWM

Modifications to the ADCIRC Model

Resolution

One of the hypothesis regarding performance of the ADICRC model in the St Charles region was that there was insufficient resolution to capture the thin channels and the details around the CSX Railroad. The finite element mesh in the region was refined from approximately 225m down to 55m. This additional resolution allowed several channels to be represented and permitted a more accurate representation of the openings through the railroad. Figures 7 and 8 show the resolution of the original mesh and Figures 9 and 10 show the resolution of the refined mesh.

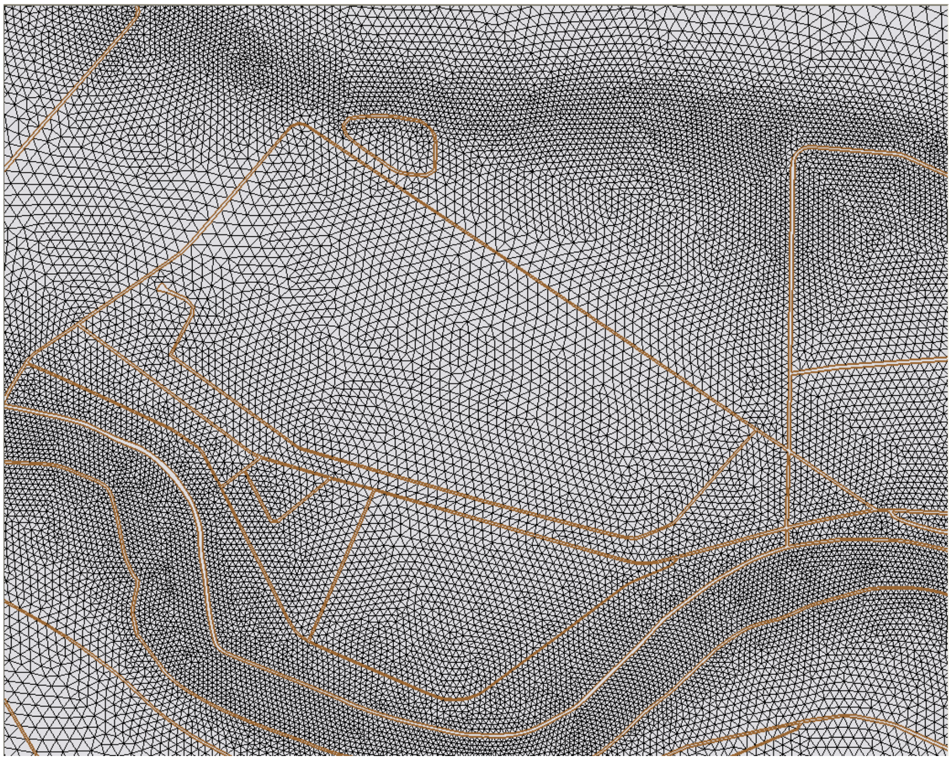


Figure 7 - Original finite element mesh in St Charles marsh.

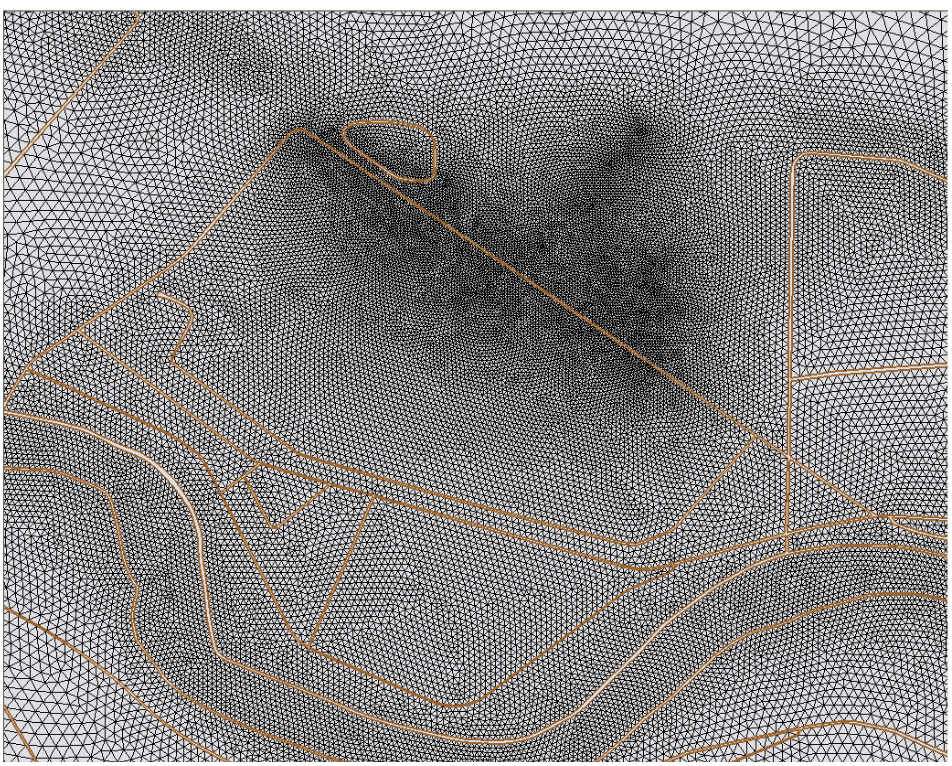


Figure 8 - Refined finite element mesh for St. Charles marsh.

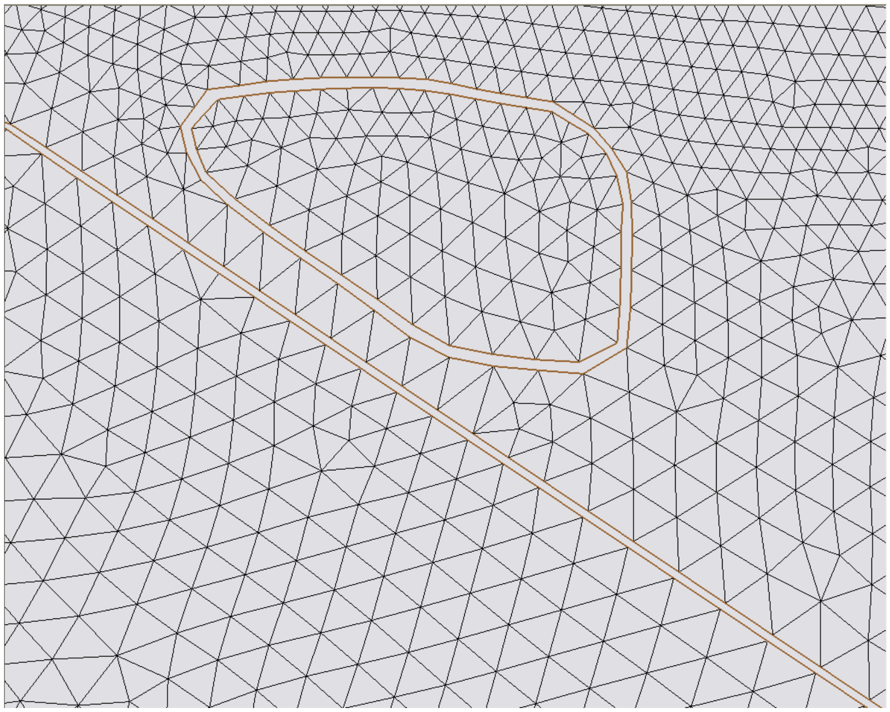


Figure 9 - Original finite element mesh near Bayou La Branche restoration area.

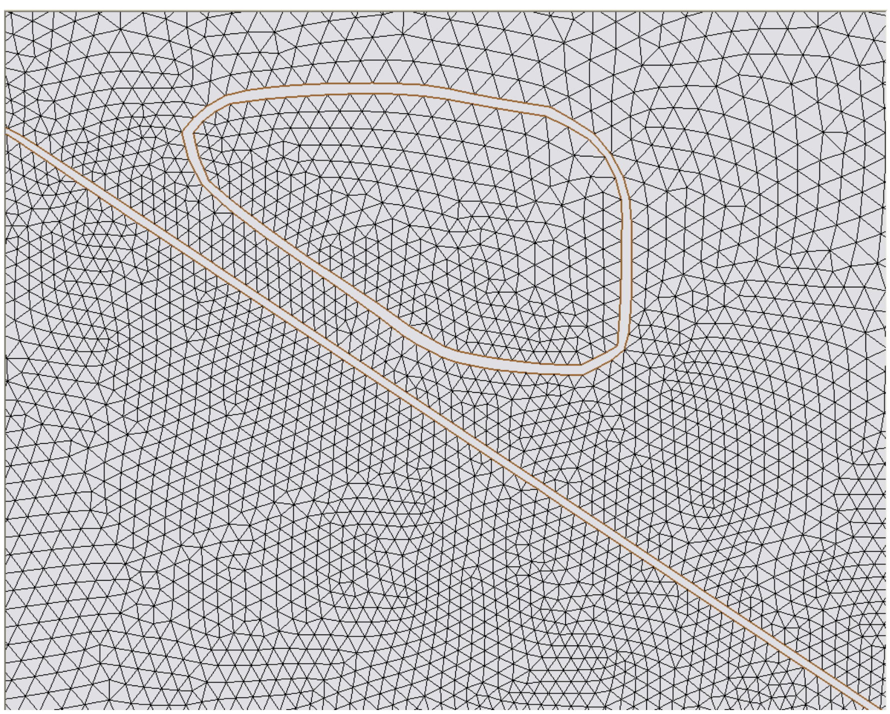


Figure 10 - Refined finite element marsh near Bayou La Branche restoration area.

Bathymetric Survey

The survey was conducted by small water craft and point-wise depths were measured manually. The data points were converted to meters and compared with the elevations in the existing ADCIRC mesh. Inspection revealed that most of the bathymetric values in the mesh were very similar to the survey values. However, there were three deeper channels that were not resolved in the ADCIRC mesh. These channels lead from Pontchartrain through the railroad and into the inner marsh area. Resolution was added to the mesh to capture these channels and elevations were implemented by manually re-setting the nodal elevations to create a channel in the model with similar conveyance characteristics as suggested by the survey. Typically, the survey captured only a single point within the channel, so estimates of channel width were made from aerial photography. Figures 11 and 12 show the original elevations in the mesh and Figures 13 and 14 show the revised elevations for this study.

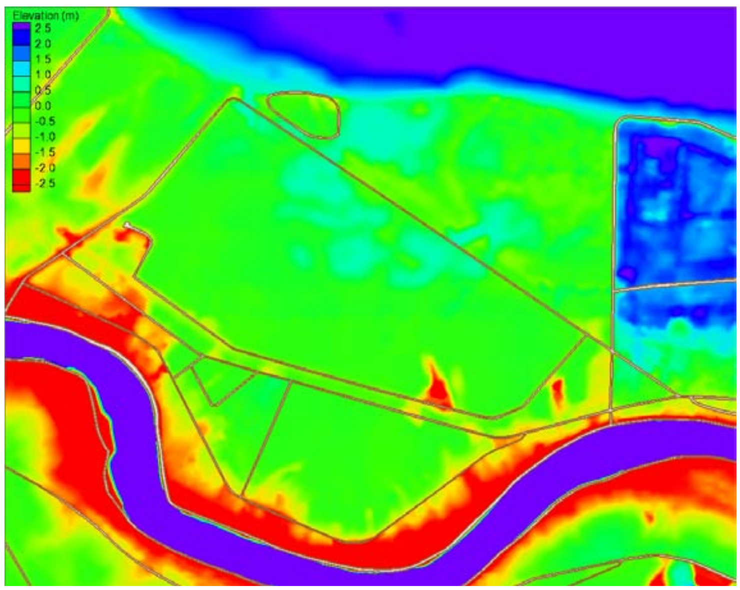


Figure 11 - Original elevations in the St. Charles marsh.

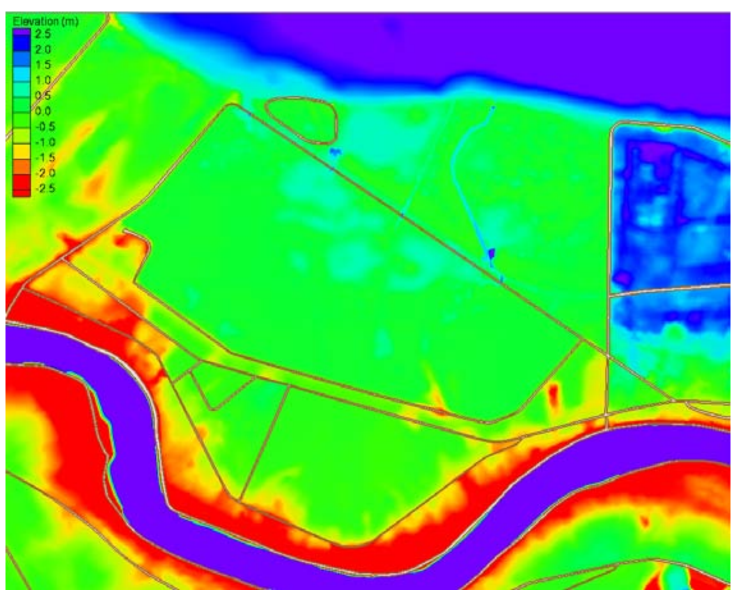


Figure 12 - Updated elevations in St. Charles marsh.

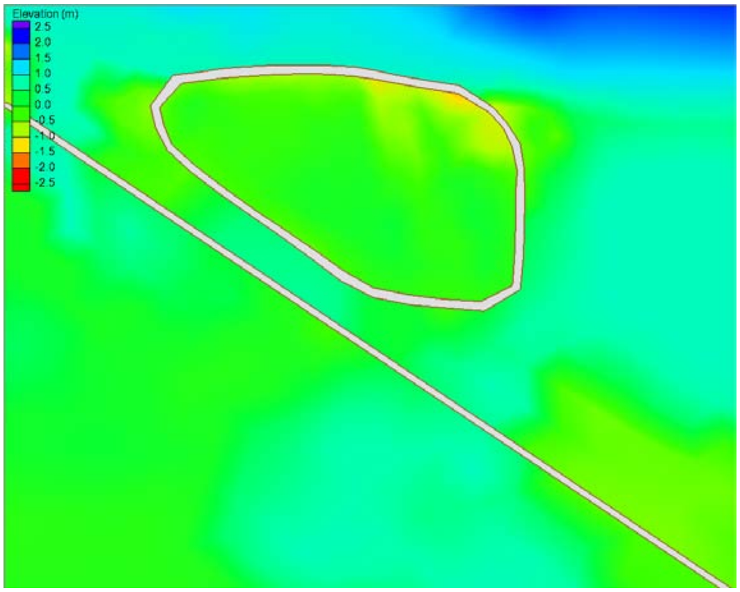


Figure 13 - Original elevations near Bayou La Branche.

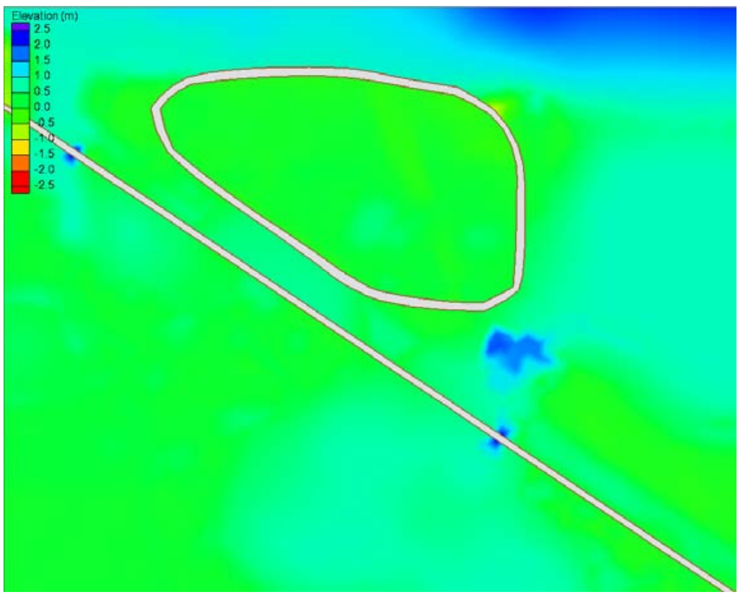


Figure 14 - Updated elevations near Bayou La Branche.

Manning's n Values

Manning-n values are derived from land cover data sets through an automated procedure. Thus, the friction parameter is dependent upon the land-cover data set, and it has been found that some regions in the data set do not reflect present conditions on the ground. Typically, this occurs as a result of new developments that have taken place since the data sets were created. Thus, aerial images and notes from a site visit in January 2008 were used to check the validity of the land cover data set for the marsh region in St Charles Parish. On average, the land cover data set values are reasonable and correlate to observed distribution of trees, marsh, and open water. The one exception is near Bayou La Branche where a restoration area has more vegetation coverage. Thus, the Manning's n values were manually reset in the restoration area according to information obtained during the site visit. Figures 15 and 16 show two frames of the Manning's n parameter distribution in the original grid and Figures 17 and 18 show the distribution in the updated grid.

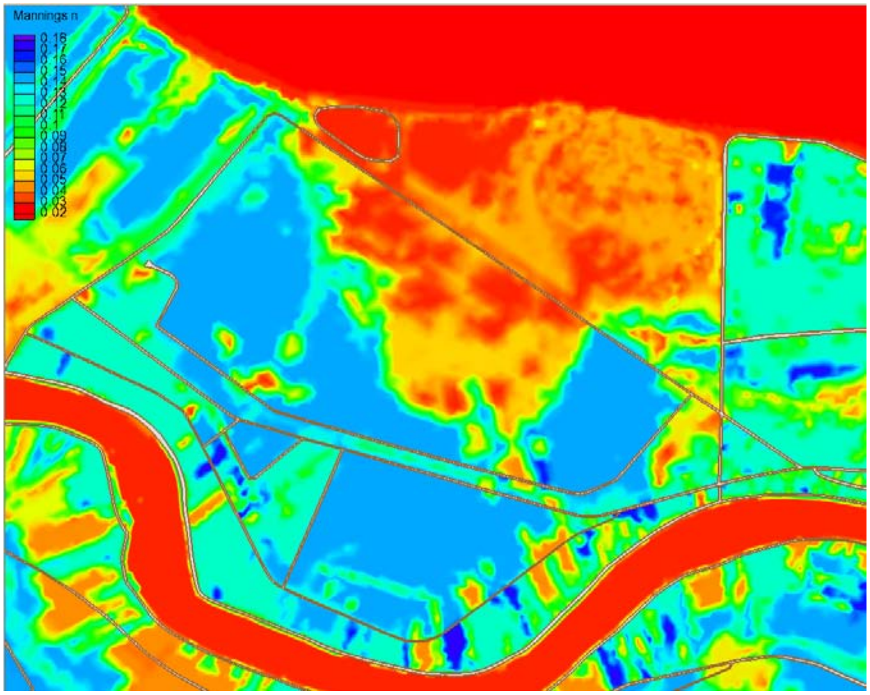


Figure 15. Original Manning's n parameter in St. Charles marsh.

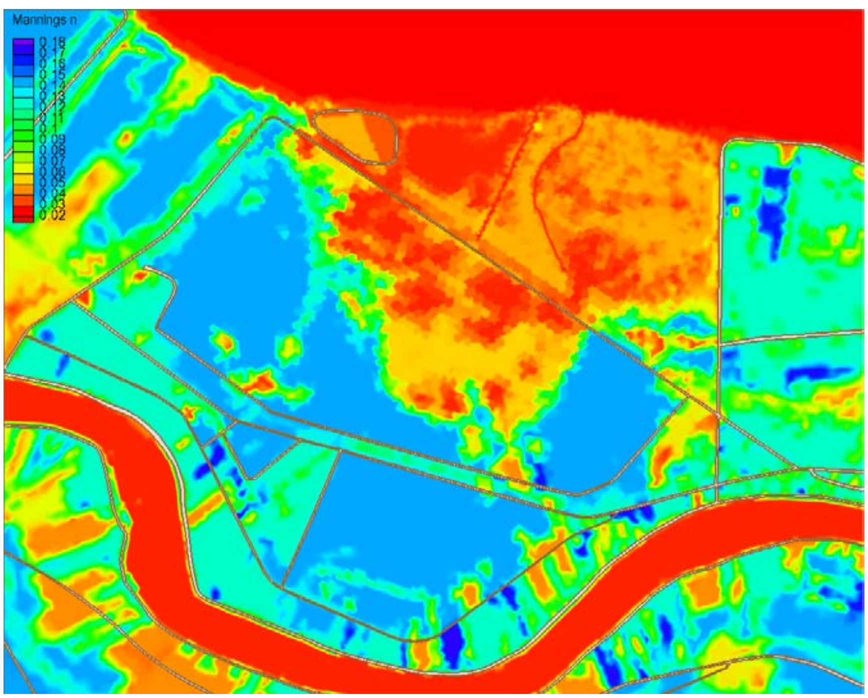


Figure 16. Updated Manning's n parameter in St. Charles marsh.

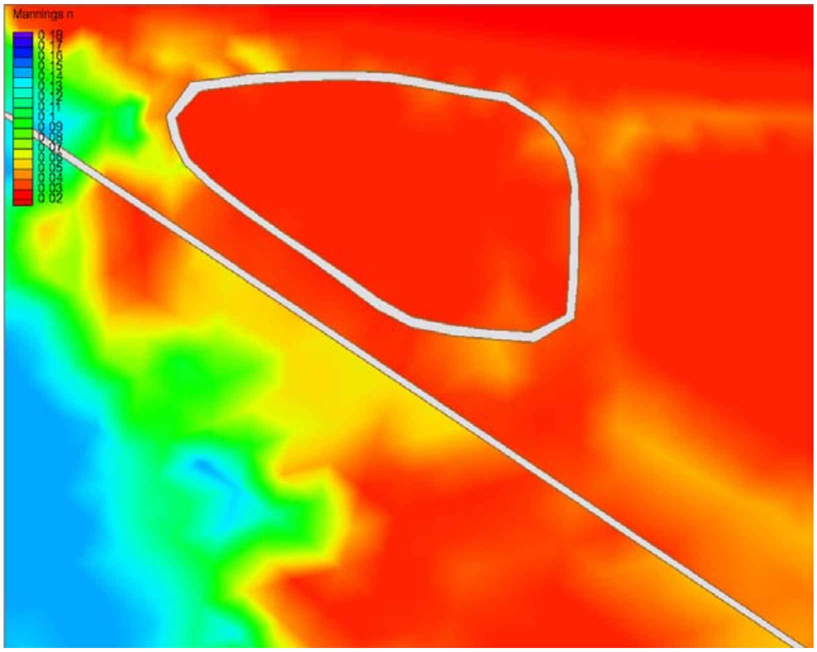


Figure 17. Original Manning's n parameter neat Bayou La Branche.

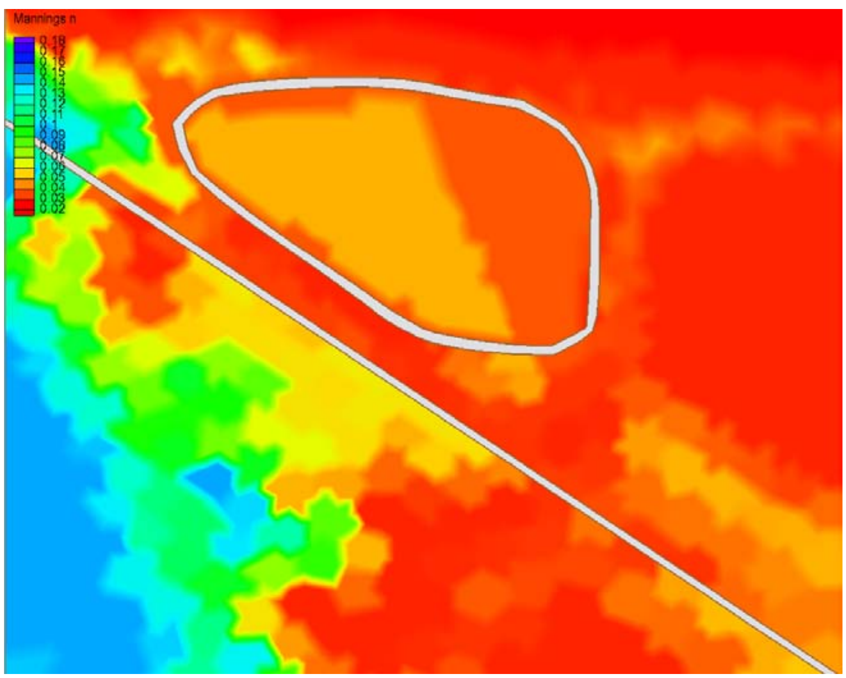


Figure 18. Updated Manning's n parameter near Bayou La Branche.

Performance of the Updated ADCIRC Model

Using the updated ADCIRC model, a hindcast of hurricane Katrina was computed to examine the changes in computed surge at the original HWMs. The new simulation had lower maximum surge levels throughout the region. At the three HWM most immediate to St. Charles marsh, the surge was lowered an average of 0.54 feet. Figures 19 through 21 show the new HWM comparisons. Refer to Figure 2 through 4 for the original HWM comparisons.

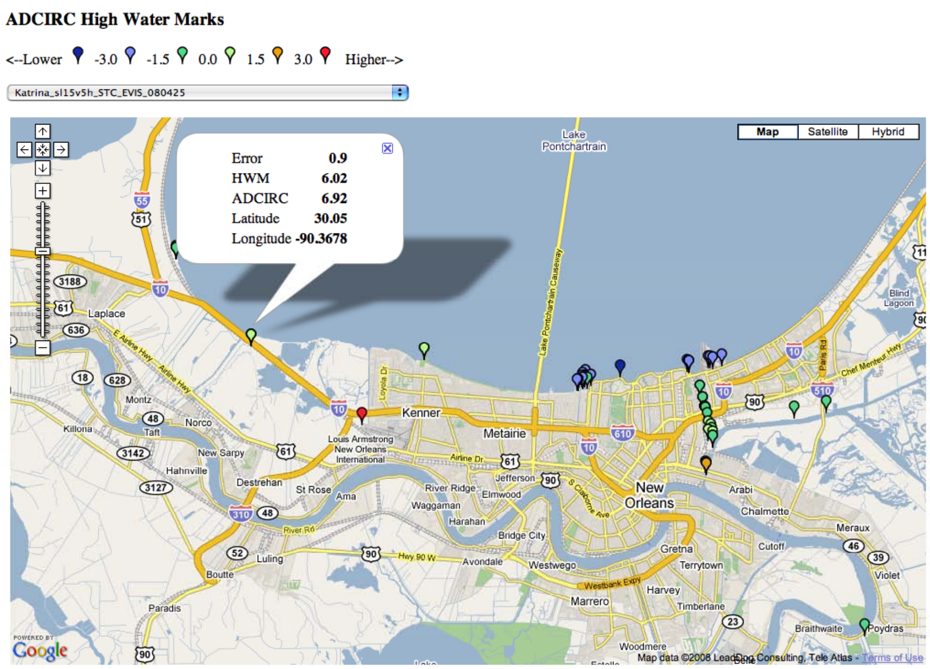


Figure 19. Katrina HWM Comparison at Bayou La Branche.

Figure 20. Katrina HWM Comparison east of St. Charles

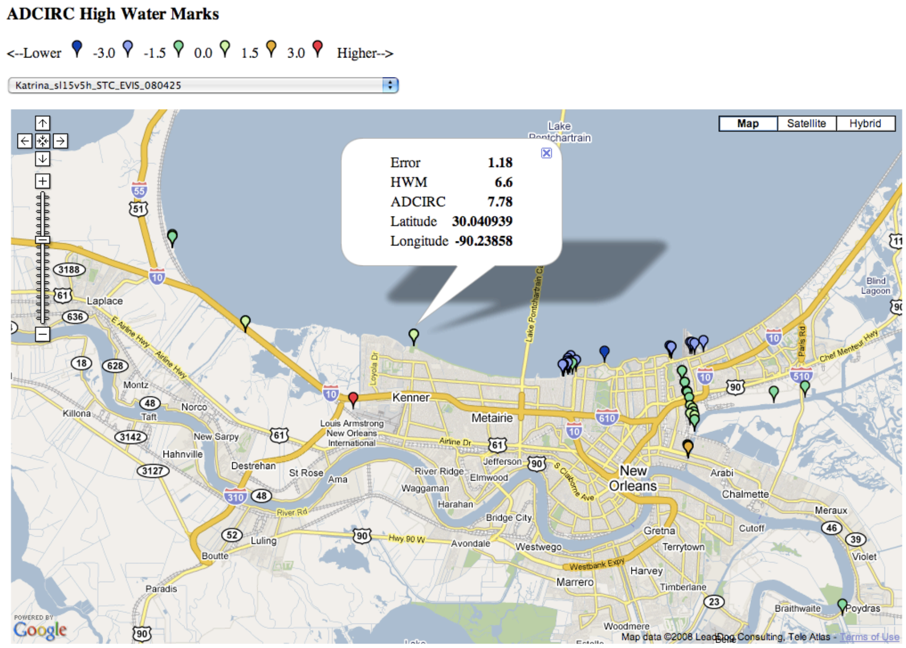


Figure 21. Katrina HWM near New Orleans Airport.

In addition, a selection of thirty-four storms from the original 152 Southeastern Louisiana FEMA storm suite was utilized to run with the updated model and evaluate the effect of the mesh adjustments. The storms were selected by USACE as reflective of the most significant storms affecting the St Charles region. An example can be seen in Figures 22 through 24. Differences in maximum surge elevation between the original 2007 FEMA runs and the St. Charles Parish analysis runs are presented in Appendix H-A. Only the ten most consequential storms are depicted in the appendix. An example plot can be seen in Figure 25.

Additional maximum surge data at specific point locations can be found in Appendix H-B for all 34 storms for both mesh configurations. Figure 26 displays the location of the analysis points. Although the differences varied throughout the marsh and varied by storm, the typical peak elevations reduced by approximately 0.40 to 0.80 feet along the Airport Highway levee at the southern edge of the St. Charles marsh. The average difference for all analysis points and all storms was a decrease in surge by 0.57 feet. No further analysis, including the effects on the 100-year return period was done.

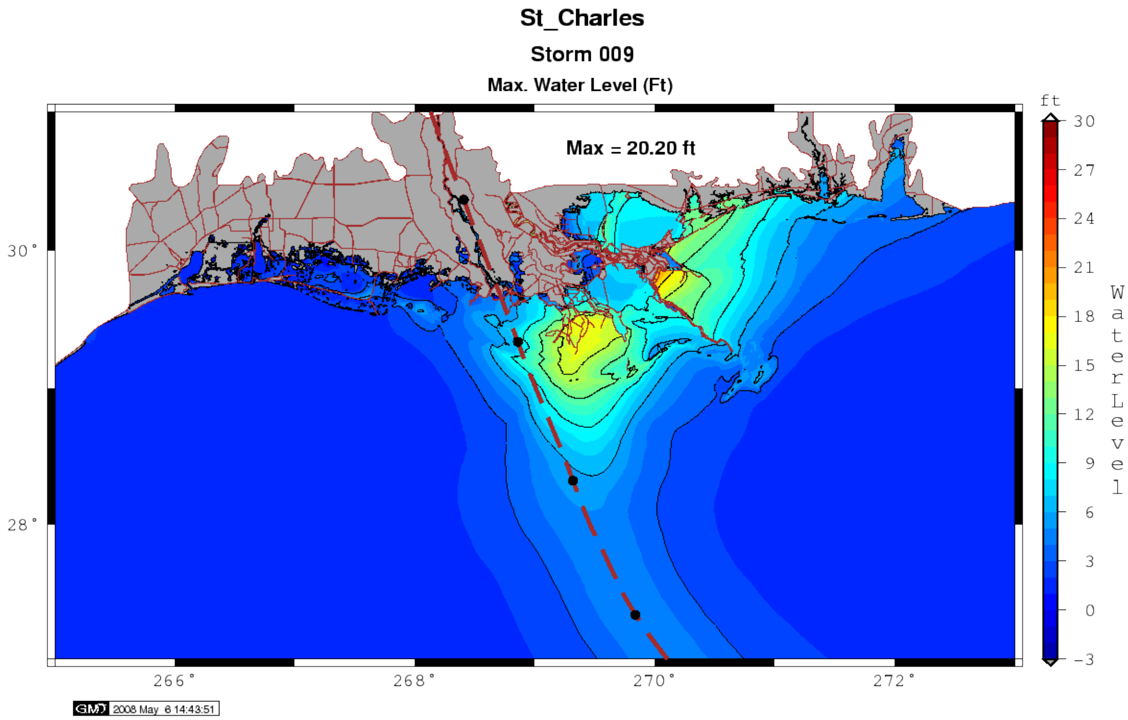


Figure 22. Maximum storm surge elevations (ft) for Storm 009 in Southern Louisiana.

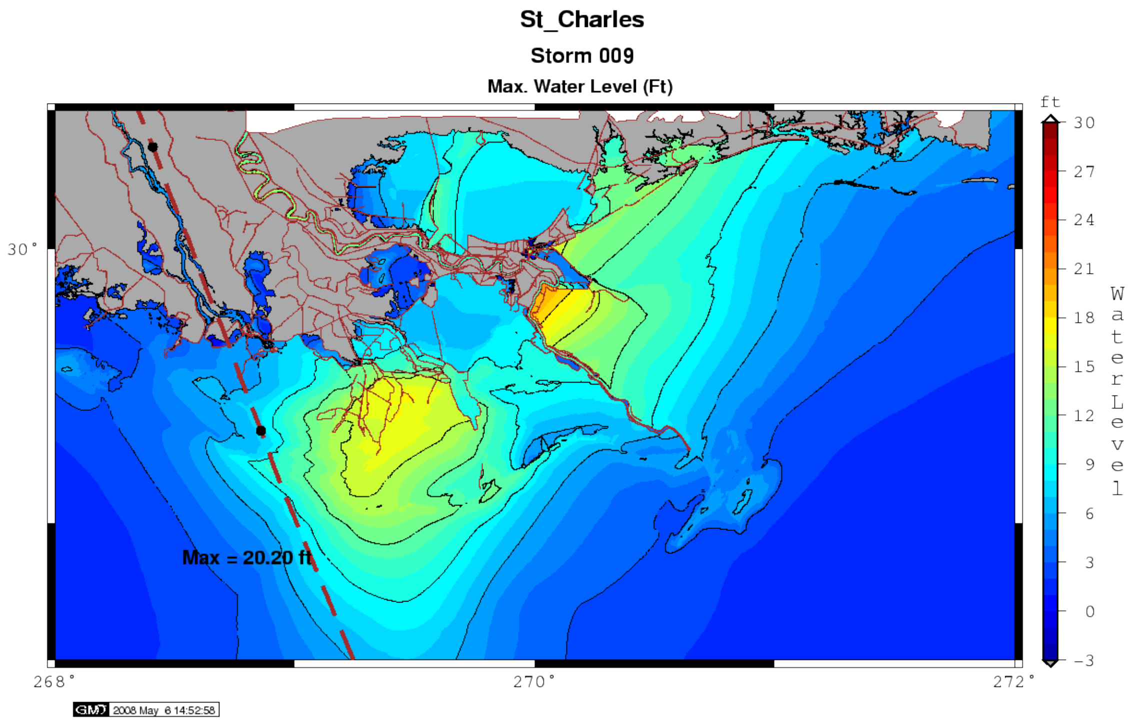


Figure 23. Maximum storm surge elevations (ft) for Storm 009 in Southeast Louisiana.

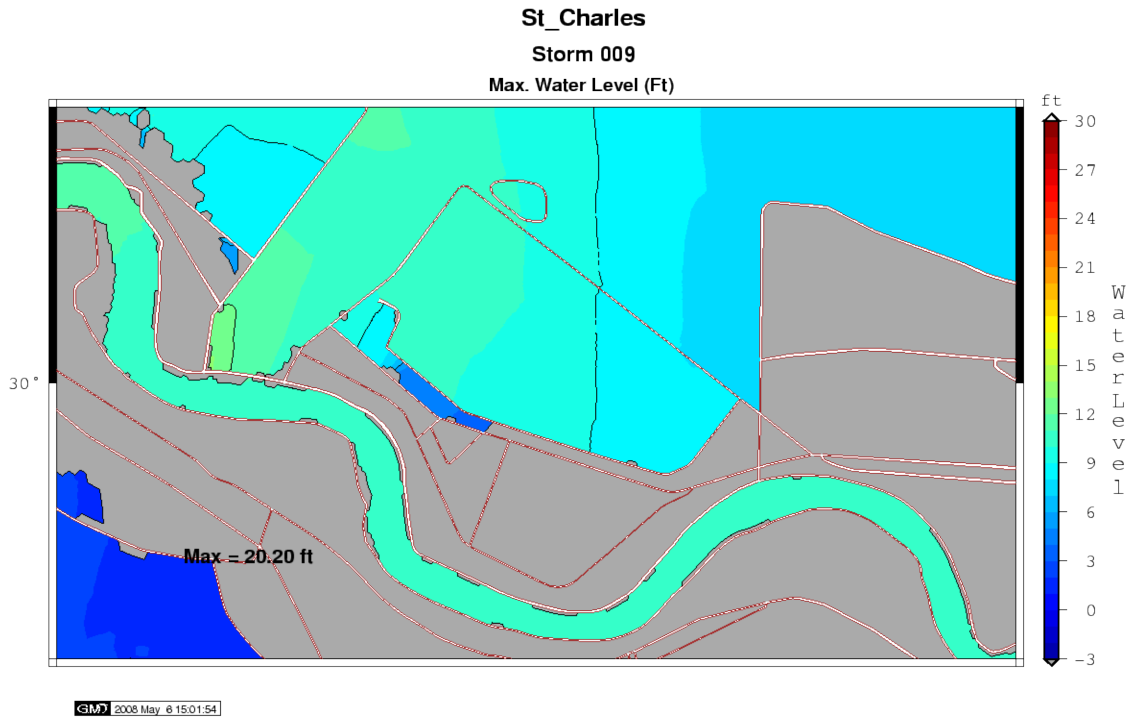


Figure 24. Maximum storm surge elevations (ft) for Storm 009 in St. Charles Parish.

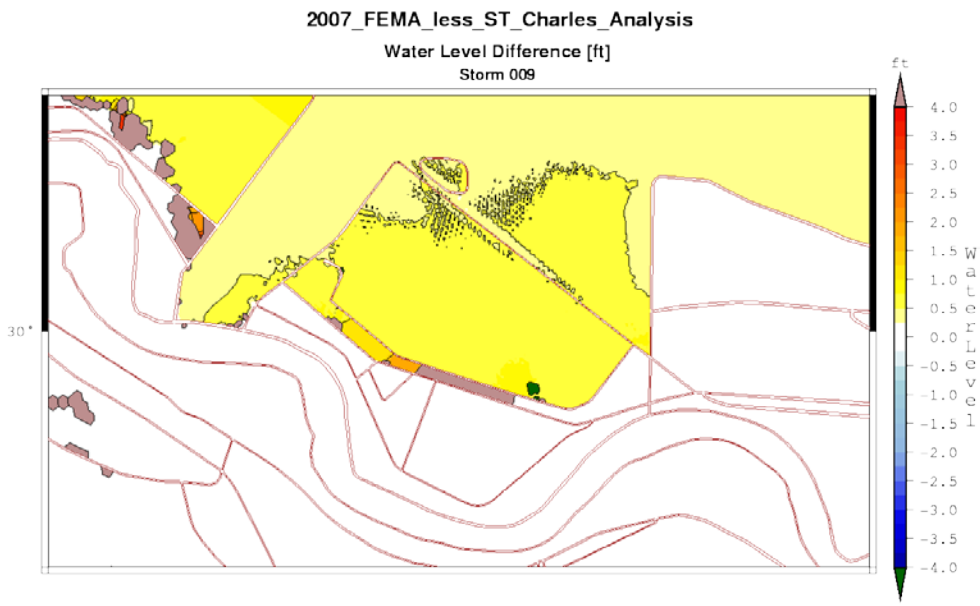


Figure 25. Differences in maximum storm surge elevations (ft) for Storm 009 in St. Charles Parish. Positive values represent lower surge values for the updated model analysis.

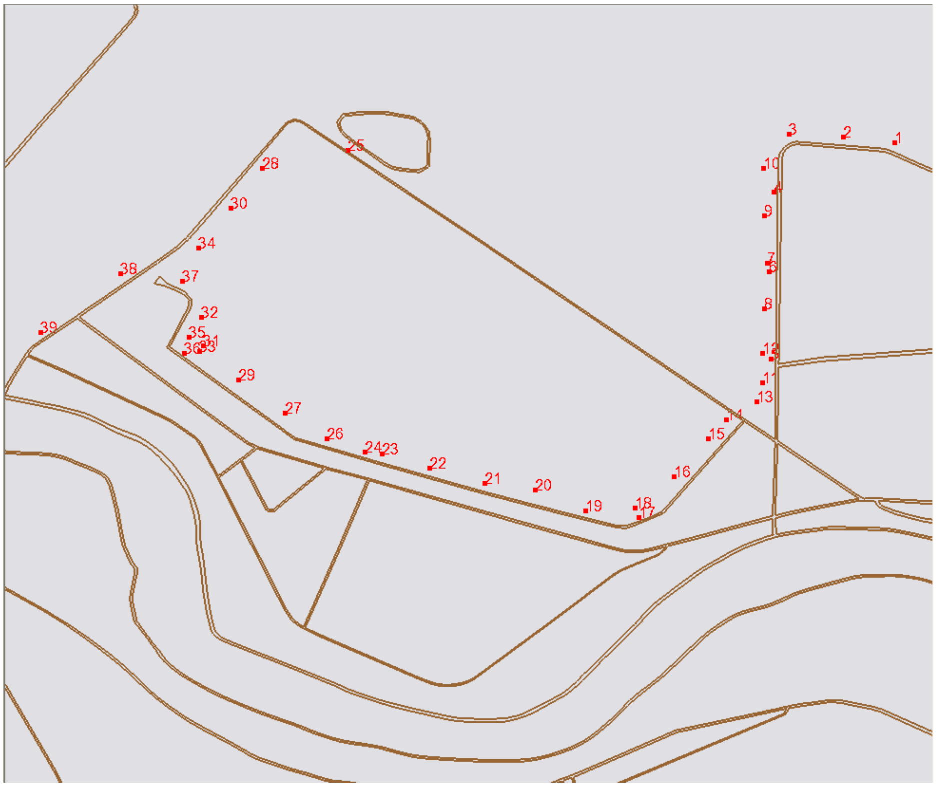


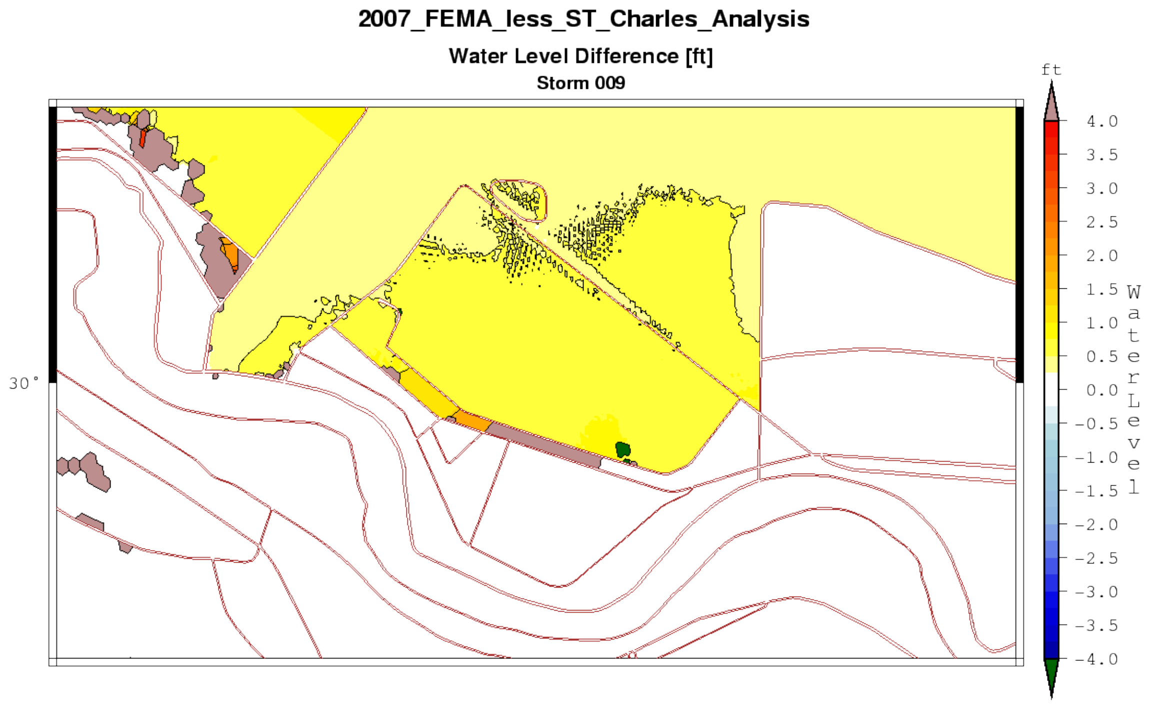
Figure 26. Location of point locations extracted for analysis of maximum surge values. A table of values can be found in Appendix H-B.

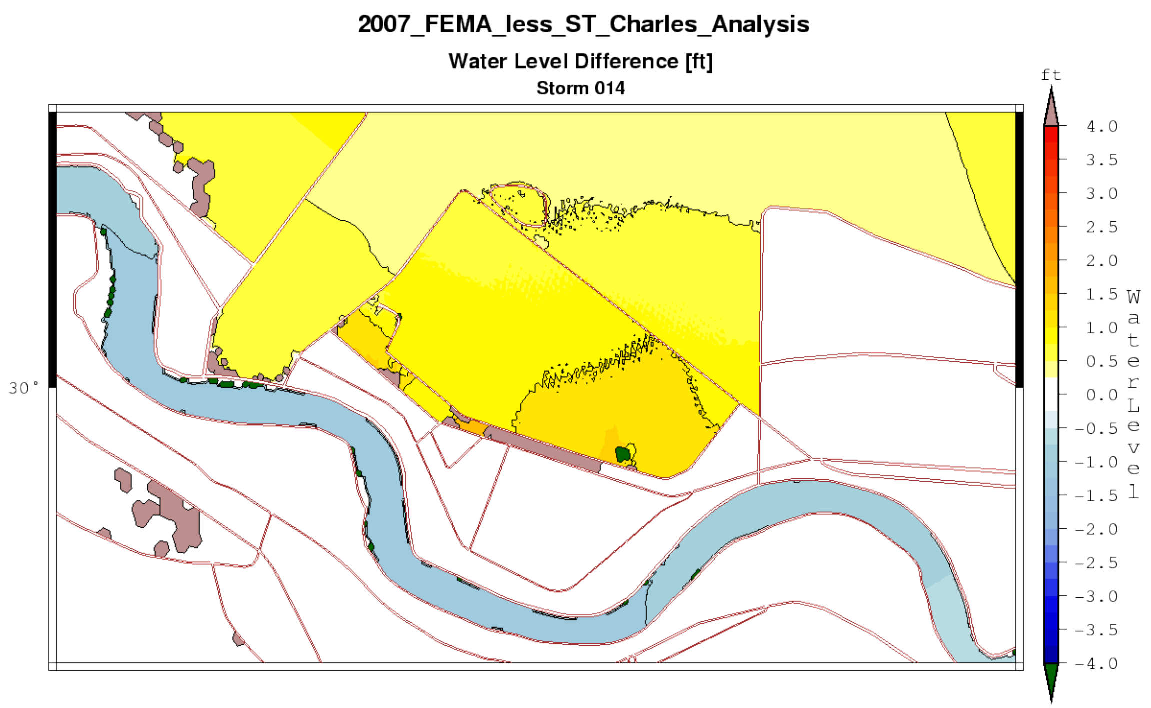
Summary and Conclusions

Physically pertinent alterations were made to the recent FEMA analysis production grid in St. Charles Parish. After an adjustment was made to the bathymetry in the region, the height of the CSX railroad, and the manning's n coefficients in the region, 34 storms were simulated in order to verify the surge levels reported in the region for the recent FEMA flood insurance study. In general, surge values were lowered between 0.25 and 1.50 feet throughout the region. It is our recommendation that the H&H Branch of the New Orleans District of the U.S. Army Corps of Engineers resume analysis in St. Charles Parish in order to quantify differences in the 100-year return period from the values previously reported in the recent FEMA flood insurance study.

Appendix H-A

Differences in 2007 FEMA Analysis and St. Charles Parish Analysis Maximum Water Elevation Plots

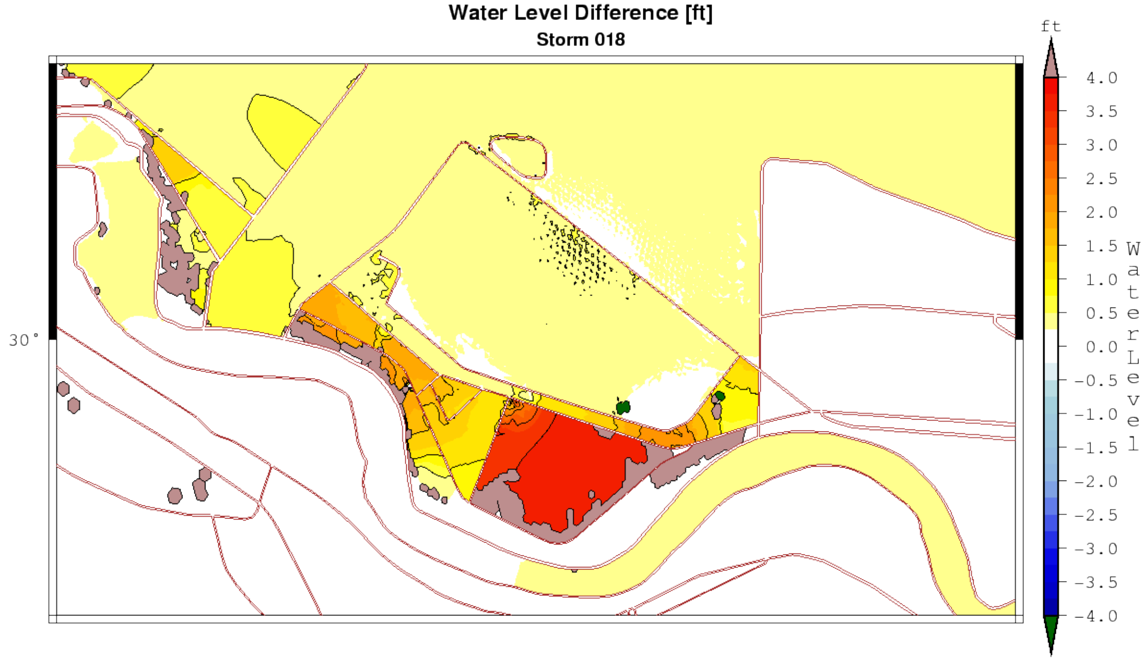

2007_FEMA_less_ST_Charles_Analysis
Water Level Difference [ft]
Storm 009
ft
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0
-3.5
-4.0
Water Level
30°


2007_FEMA_less_ST_Charles_Analysis
Water Level Difference [ft]
Storm 014
ft
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0
-3.5
-4.0
Water Level
30°

2007_FEMA_less_ST_Charles_Analysis

Water Level Difference [ft]

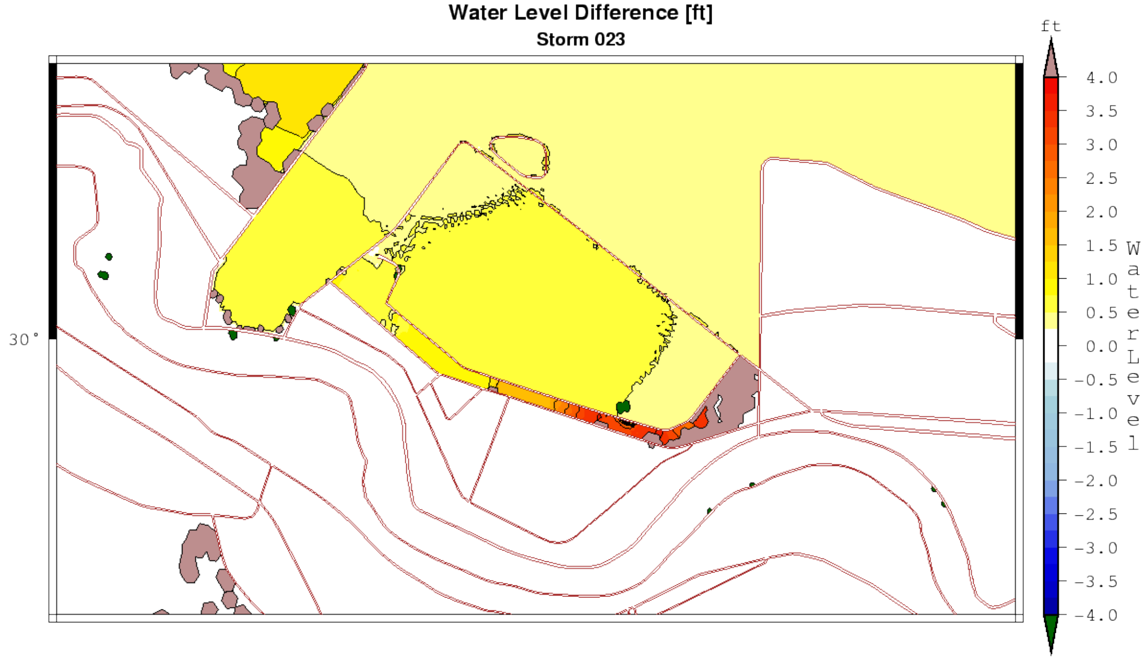
Storm 018



2007_FEMA_less_ST_Charles_Analysis

Water Level Difference [ft]

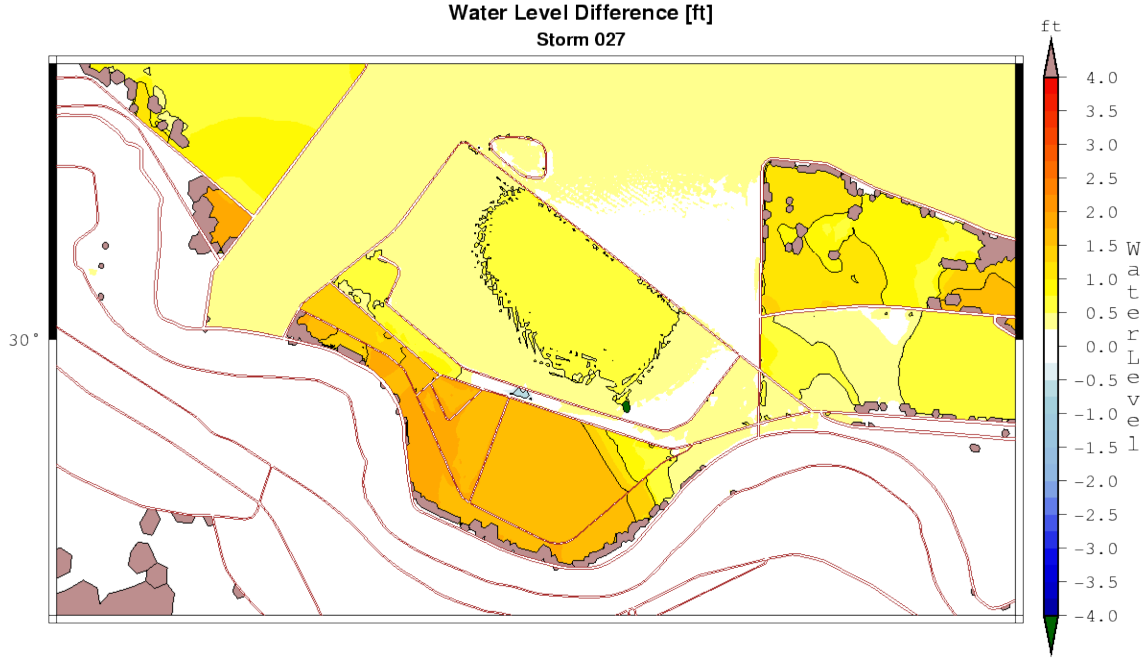
Storm 023



2007_FEMA_less_ST_Charles_Analysis

Water Level Difference [ft]

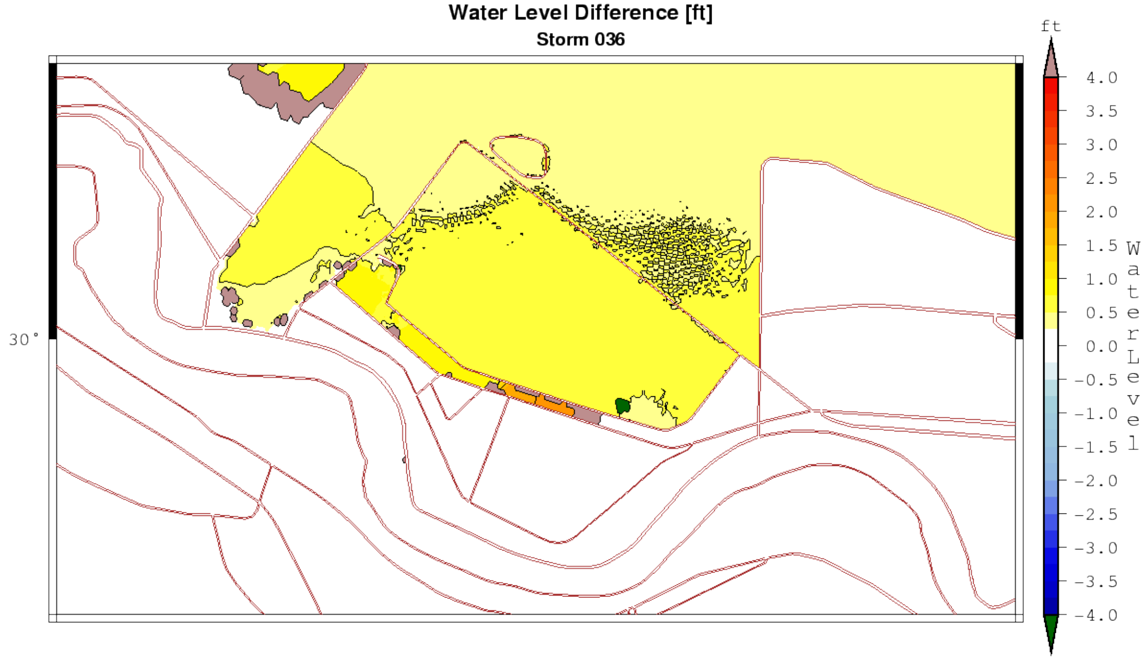
Storm 027



2007_FEMA_less_ST_Charles_Analysis

Water Level Difference [ft]

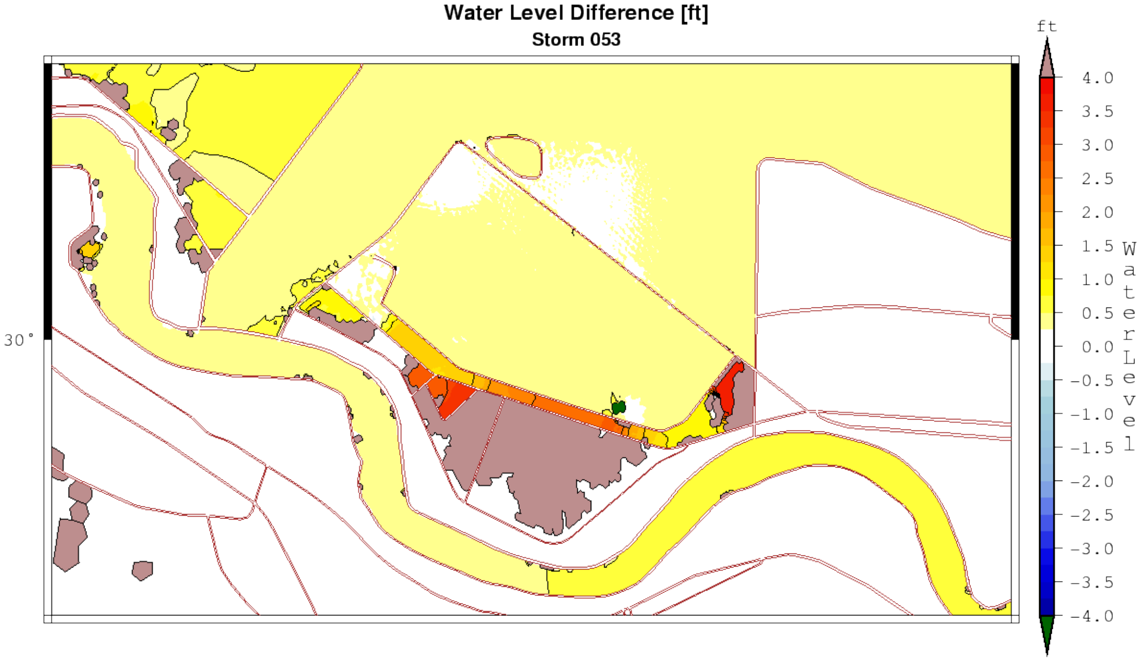
Storm 036



2007_FEMA_less_ST_Charles_Analysis

Water Level Difference [ft]

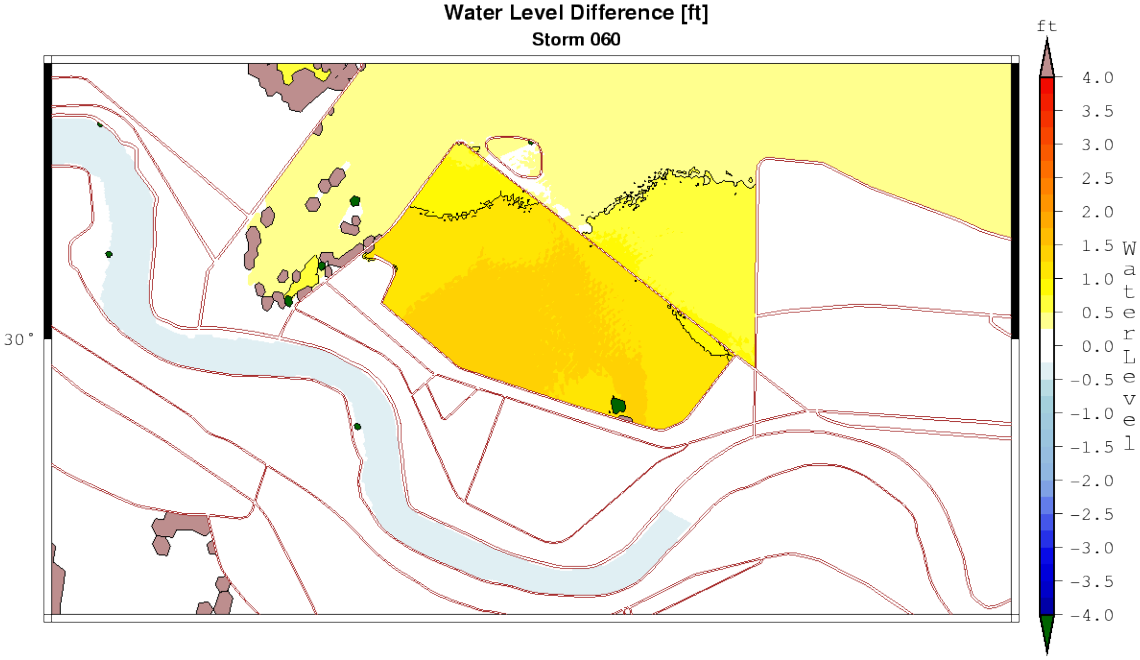
Storm 053

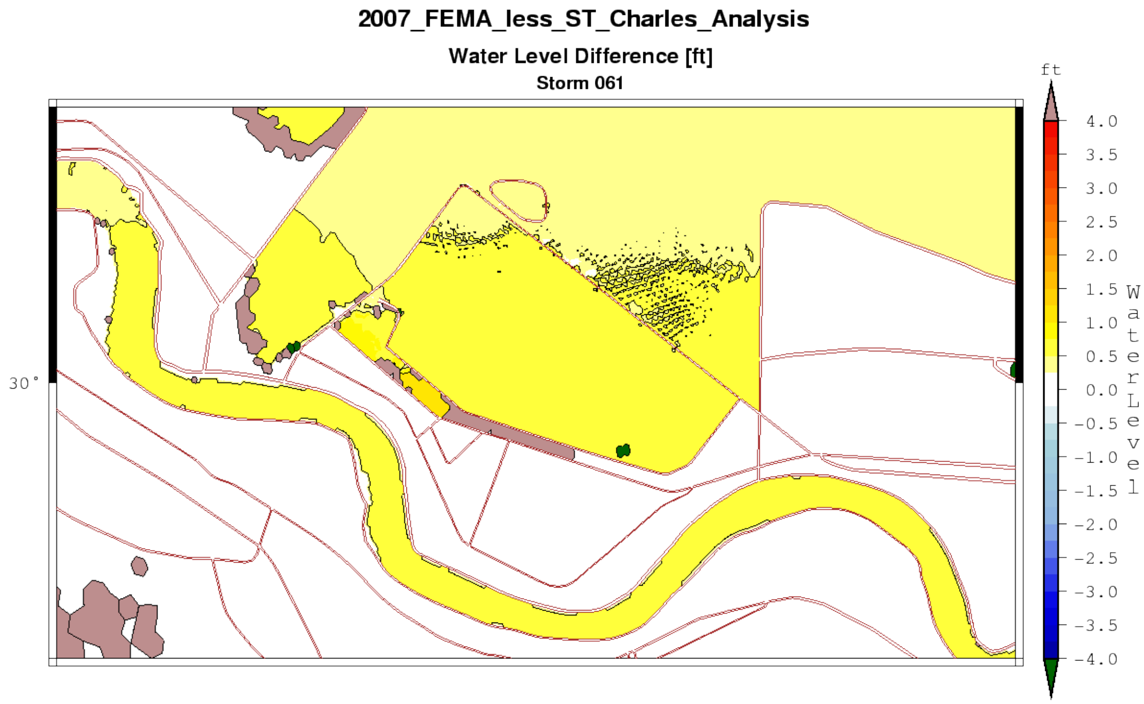


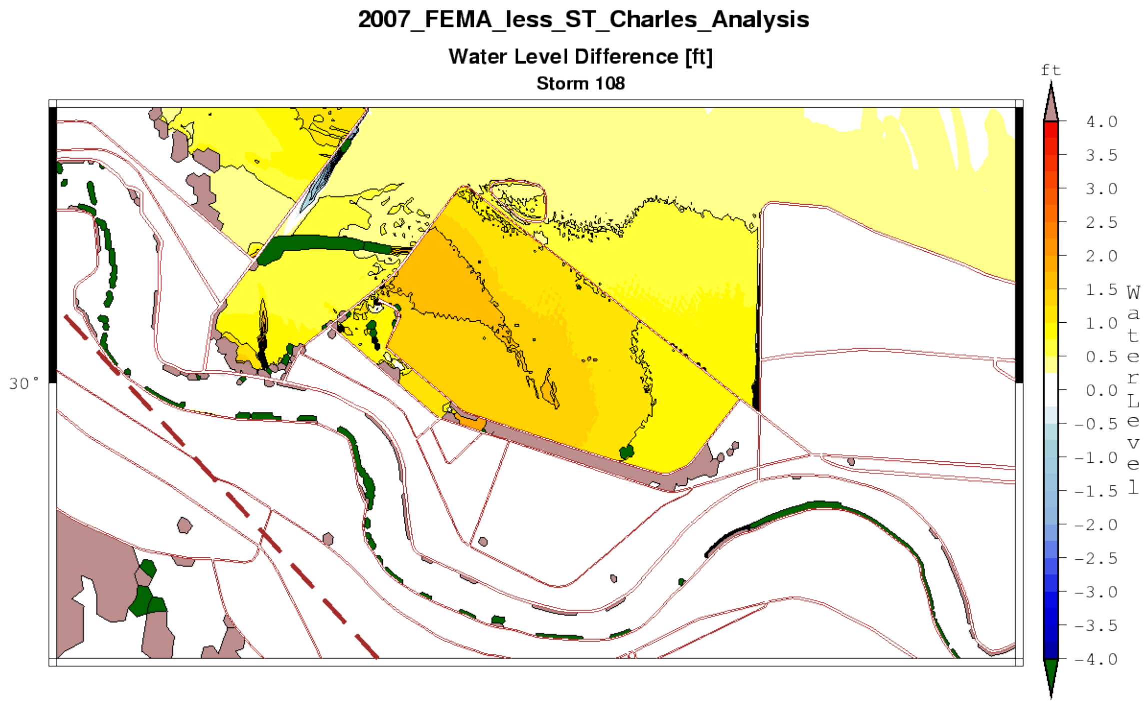
2007_FEMA_less_ST_Charles_Analysis

Water Level Difference [ft]

Storm 060




2007_FEMA_less_ST_Charles_Analysis
Water Level Difference [ft]
Storm 061
ft
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0
-3.5
-4.0
Water Level
30°


2007_FEMA_less_ST_Charles_Analysis
Water Level Difference [ft]
Storm 108
ft
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-2.5
-3.0
-3.5
-4.0
Water Level
30°

Appendix H-B

St Charles Parish East Bank

Peak Surge with updated modeling

ID	Point Set	Point Set ID	Longitude	Latitiude	Storm Number																																	
					9	14	15	17	18	22	23	24	25	26	27	35	36	49	52	53	56	57	60	61	72	73	77	83	85	89	93	94	98	99	102	103	108	109
					Maximum Surge (ft)																																	
1	Q835	164	-90.3623	30.0478	10.33	10.92	11.12	14.00	14.44	7.50	11.20	11.28	7.29	13.92	14.42	7.49	8.59	10.16	9.25	12.78	17.00	15.18	5.46	8.32	12.49	13.26	10.05	10.22	13.91	8.76	10.15	12.13	9.91	8.28	11.21	10.71	13.18	7.19
2	Q835	165	-90.3928	30.0117	10.14	10.60	11.09	13.96	14.41	6.42	11.90	11.99	6.10	14.24	14.54	7.85	9.58	10.16	8.55	13.39	15.53	14.96	5.64	9.57	13.32	13.68	10.94	9.98	14.07	9.75	10.65	12.65	10.31	8.97	9.00	9.92	12.04	5.84
3	Q835	263	-90.3900	30.0100	10.13	10.59	11.08	13.95	14.42	6.42	11.92	12.01	6.10	14.27	14.57	7.86	9.60	10.16	8.54	13.39	15.61	15.00	5.65	9.59	13.34	13.69	10.96	9.97	14.07	9.77	10.65	12.66	10.31	8.98	8.98	9.93	12.03	5.84
4	Q835	665	-90.2809	30.0075	7.56	7.87	8.92	10.91	11.73	9.07	12.01	11.63	8.94	15.18	15.05	8.83	10.19	7.96	7.09	10.93	13.88	14.26	9.43	11.52	14.93	12.55	11.17	8.01	11.91	10.10	8.89	11.52	9.32	8.87	7.17	10.80	11.20	8.99
5	Q835	666	-90.2812	30.0243	7.49	7.36	8.34	10.00	10.88	8.99	11.40	10.95	8.83	14.50	14.42	8.29	9.53	7.39	6.56	9.92	13.09	13.25	9.36	10.90	14.33	11.84	10.34	7.79	11.11	9.47	8.30	10.79	8.80	8.29	6.60	10.43	10.82	8.74
6	Q835	667	-90.2804	30.0396	7.53	7.44	8.19	9.71	10.57	8.49	10.57	10.22	8.30	13.58	13.52	7.53	8.66	7.24	6.30	9.24	12.46	12.33	8.40	9.77	13.17	10.99	9.37	7.84	10.85	8.72	8.03	10.12	8.23	7.60	6.97	9.92	10.31	8.09
7	Q835	673	-90.2837	29.9992	7.62	7.96	9.02	11.10	11.89	8.95	12.08	11.72	8.84	15.03	14.96	8.83	10.21	8.04	7.12	11.10	14.02	14.33	9.30	11.51	14.78	12.66	11.22	8.08	12.05	10.13	8.98	11.64	9.40	8.91	7.22	10.83	11.32	8.89
8	Q835	674	-90.3938	30.0086	10.14	10.60	11.09	13.96	14.41	6.42	11.91	12.00	6.10	14.24	14.53	7.86	9.59	10.17	8.55	13.39	15.49	14.94	5.64	9.58	13.33	13.68	10.95	9.98	14.07	9.76	10.66	12.66	10.31	8.98	9.00	9.92	12.04	5.84
9	Q835	675	-90.3589	29.9895	9.80	10.17	10.90	13.71	14.30	6.82	12.36	12.39	6.47	14.69	14.91	8.47	10.21	9.97	8.28	13.40	15.84	15.20	6.55	10.49	13.89	13.82	11.57	9.70	14.00	10.30	10.64	12.92	10.42	9.41	8.21	10.32	11.85	6.46
10	Q835	676	-90.3064	29.9770	8.35	8.54	9.76	12.05	12.82	8.01	12.52	12.28	7.64	14.59	14.61	9.01	10.61	8.77	6.95	12.15	14.43	14.38	8.54	11.63	14.31	13.18	11.80	8.65	12.83	10.56	9.72	12.42	10.00	9.46	6.25	10.88	11.67	7.81
11	Q835	678	-90.4213	30.0126	10.64	10.55	11.12	13.82	14.93	5.47	10.11	10.77	5.26	13.09	14.39	7.12	8.37	10.29	8.31	12.57	14.25	14.52	5.08	7.14	10.61	12.99	9.48	10.84	14.76	8.79	10.49	12.47	10.30	8.31	8.66	8.27	9.33	5.02
12	Q835	679	-90.4060	30.0240	10.62	10.60	11.05	13.80	14.74	5.69	9.84	10.49	5.49	12.68	13.96	6.64	7.90	10.15	8.36	12.28	14.28	14.32	4.98	6.71	10.38	12.57	9.02	10.74	14.39	8.39	10.24	12.19	9.93	7.92	9.60	8.42	10.25	5.29
13	D1479	220	-90.2825	30.0086	7.53	7.84	8.88	10.86	11.68	9.10	11.99	11.60	8.97	15.19	15.05	8.83	10.19	7.93	7.08	10.89	13.84	14.29	9.48	11.53	14.95	12.52	11.15	7.99	11.88	10.09	8.87	11.49	9.29	8.85	7.14	10.80	11.17	9.02
14	D1479	221	-90.2822	30.0172	7.48	7.47	8.48	10.22	11.10	9.18	11.69	11.24	9.03	14.95	14.80	8.61	9.89	7.54	6.77	10.25	13.31	13.75	9.63	11.31	14.76	12.13	10.73	7.77	11.36	9.80	8.49	11.06	8.99	8.56	6.74	10.65	10.94	9.00
15	D1479	222	-90.2817	30.0259	7.50	7.38	8.34	10.00	10.88	8.94	11.33	10.89	8.77	14.46	14.34	8.21	9.44	7.39	6.54	9.88	13.07	13.18	9.25	10.79	14.22	11.78	10.25	7.79	11.11	9.40	8.28	10.74	8.76	8.23	6.64	10.38	10.79	8.67
16	D1479	223	-90.2823	30.0351	7.52	7.45	8.29	9.89	10.75	8.62	10.85	10.47	8.44	13.90	13.82	7.75	8.93	7.33	6.40	9.54	12.77	12.66	8.65	10.12	13.52	11.30	9.69	7.82	11.00	8.95	8.14	10.38	8.45	7.83	6.87	10.08	10.54	8.28
17	D1479	224	-90.2824	30.0443	7.55	7.77	8.33	9.88	10.70	8.25	10.29	10.05	8.04	13.29	13.32	7.32	8.37	7.41	6.57	9.39	12.46	12.32	7.71	9.25	12.75	10.84	9.11	7.85	10.91	8.48	8.06	10.07	8.14	7.46	7.51	9.85	10.18	7.81
18	D1479	225	-90.2775	30.0509	7.58	7.91	8.33	9.79	10.61	7.96	9.85	9.66	7.74	12.75	12.85	7.01	7.96	7.42	6.70	9.31	12.31	12.04	6.95	8.54	12.11	10.41	8.76	7.89	10.77	8.12	8.01	9.83	7.91	7.20	7.78	9.67	10.31	7.43
19	D1479	226	-90.2671	30.0504	7.59	7.69	8.10	9.45	10.32	8.03	9.68	9.48	7.81	12.58	12.66	6.98	7.91	7.20	6.53	9.06	12.03	11.82	7.08	8.54	11.96	10.05	8.64	7.89	10.51	8.08	7.84	9.62	7.70	7.12	7.55	9.53	10.15	7.43
20	D1479	227	-90.2572	30.0493	7.59	7.48	7.88	9.17	10.04	8.11	9.54	9.33	7.91	12.45	12.49	6.96	7.88	7.15	6.37	8.83	11.77	11.63	7.26	8.58	11.85	9.74	8.55	7.89	10.28	8.06	7.67	9.44	7.51	7.06	7.32	9.40	10.00	7.46
21	D1479	238	-90.2894	29.9958	8.09	8.26	9.37	11.43	12.19	8.32	12.21	11.88	7.98	14.92	14.88	8.87	10.32	8.45	6.83	11.43	14.25	14.42	8.90	11.54	14.65	12.85	11.39	8.42	12.31	10.26	9.32	11.87	9.65	9.14	6.33	10.82	11.44	8.12
22	D1479	239	-90.2929	29.9922	8.14	8.31	9.46	11.59	12.36	8.23	12.29	11.98	7.89	14.81	14.79	8.89	10.39	8.51	6.84	11.62	14.29	14.39	8.80	11.55	14.54	12.95	11.49	8.46	12.45	10.33	9.41	12.01	9.74	9.20	6.32	10.84	11.52	8.03
23	D1479	240	-90.2995	29.9847	8.27	8.45	9.65	11.88	12.66	8.08	12.44	12.17	7.72	14.68	14.69	8.96	10.53	8.67	6.90	11.96	14.39	14.39	8.62	11.60	14.40	13.10	11.68	8.58	12.70	10.47	9.61	12.26	9.91	9.36	6.28	10.87	11.63	7.87
24	D1479	241	-90.3070	29.9787	8.35	8.54	9.75	12.05	12.82	8.01	12.51	12.28	7.65	14.62	14.64	9.01	10.61	8.77	6.95	12.15	14.47	14.41	8.54	11.62	14.33	13.18	11.80	8.65	12.83	10.56	9.72	12.42	10.00	9.46	6.25	10.88	11.67	7.81
25	D1479	242	-90.3166	29.9783	8.41	8.60	9.83	12.17	12.94	7.96	12.58	12.37	7.60	14.79	14.83	9.05	10.68	8.84	6.99	12.31	14.71	14.61	8.49	11.65	14.42	13.27	11.90	8.70	12.93	10.64	9.81	12.54	10.07	9.54	6.23	10.89	11.69	7.78
26	D1479	243	-90.3262	29.9823	8.88	9.10	10.20	12.70	13.42	7.76	12.65	12.51	7.39	15.09	15.20	9.06	10.72	9.25	7.48	12.69	15.42	15.17	8.15	11.53	14.53	13.51	12.00	9.04	13.29	10.71	10.10	12.82	10.21	9.66	6.62	10.85	11.73	7.60
27	D1479	244	-90.3359	29.9835	9.10	9.35	10.39	12.94	13.67	7.59	12.66	12.57	7.23	15.21	15.34	9.04	10.72	9.43	7.67	12.92	15.74	15.40	7.86	11.43	14.54	13.64	12.02	9.19	13.50	10.72	10.27	12.95	10.31	9.69	7.02	10.82	11.77	7.42
28	D1479	245	-90.3465	29.9865	9.52	9.83	10.72	13.45	14.15	7.29	12.61	12.58	6.93	15.08	15.26	8.87	10.59	9.78	8.06	13.32	16.02	15.47	7.32	11.08	14.34	13.82	11.94	9.50	13.88	10.62	10.53	13.05	10.42	9.66	7.70	10.67	11.81	7.08
29	D1479	246	-90.3558	29.9892	9.75	10.11	10.87	13.68	14.29	6.94	12.43	12.45	6.59	14.79	15.00	8.57	10.31	9.94	8.25	13.40	15.91	15.28	6.75	10.64	14.00	13.83	11.67	9.67	13.99	10.38	10.63	12.97	10.43	9.48	8.10	10.40	11.83	6.63
30	D1479	247	-90.3663	29.9920	9.89	10.29	10.94	13.76	14.31	6.59	12.20	12.26	6.26	14.53	14.77	8.24	9.99	10.01	8.34	13.40	15.71	15.08	6.14	10.16	13.70	13.79	11.35	9.77	14.01	10.10	10.64	12.81	10.40	9.26	8.47	10.14	11.91	6.08
31	D1479	248	-90.3744	29.9970	9.97	10.40	11.00	13.83	14.38	6.51	12.12	12.19	6.18	14.48	14.74	8.11	9.86	10.07	8.41	13.42	15.75	15.10	5.91	9.97	13.61	13.78	11.22	9.84	14.06	9.99	10.67	12.78	10.39	9.17	8.63	10.05	11.96	5.92
32	D1479	249	-90.3833	30.0035	10.11	10.55	11.07	13.95	14.44	6.44	11.99	12.07	6.12	14.34	14.63	7.94	9.68	10.15	8.52	13.41	15.74	15.07	5.70	9.71	13.44	13.72	11.05	9.95	14.08	9.84	10.66	12.71	10.34	9.05	8.88	9.96	12.02	5.85
33	D1479	250	-90.3908	30.0089	10.13	10.59	11.09	13.96	14.42	6.42	11.92	12.01	6.10	14.26	14.56	7.86	9.60	10.16	8.54	13.39	15.57	14.98	5.65	9.59	13.35	13.69	10.96	9.97	14.07	9.77	10.65	12.66	10.31	8.99	8.98	9.93	12.04	5.84
34	D1479	251	-90.3905	30.0155	10.15	10.60	11.08	13.95	14.43	6.41	11.88	11.97	6.09	14.24	14.56	7.82	9.55	10.17	8.55	13.37	15.67	15.02	5.63	9.52	13.28	13.66	10.90	9.99	14.07	9.72	10.65	12.63	10.30	8.95	9.03	9.90	12.02	5.83
35	D1479	252	-90.3942	30.0225	10.16	10.62	11.06	13.89	14.37	6.37	11.67	11.77	6.05	14.06	14.47	7.65	9.35	10.16	8.58	13.26	15.65	15.01	5.54	9.26	12.99	13.49	10.67	10.02	14.03	9.54	10.60	12.50	10.21	8.81	9.15	9.78	11.92	5.80
36	D1479	253	-90.2825	30.0029	7.59	7.92	8.97	11.01	11.82	9.00	12.05	11.68	8.88	15.10	15.01	8.83	10.20	8.00	7.11	11.03	13.96	14.32	9.36	11.52	14.86	12.62	11.20	8.05	11.99	10.12	8.94	11.59	9.36	8.90	7.21	10.82	11.27	8.93
37	D1479	1464	-90.3910	30.0289	10.22	10.68	11.11	13.99	14.48	6.36	11.68	11.79	6.04	14.14	14.54	7.63	9.33	10.20	8.60	13.30	16.00	15.14	5.53	9.24	13.01	13.55	10.66	10.05	14.07	9.53	10.62	12.52	10.22	8.80	9.23	9.78	11.94	5.80
38	D1479	1465	-90.3847	30.0367	10.34	10.85	11.21	14.14	14.61	6.32	11.65	11.76	6.00	14.15	14.56	7.51	9.25	10.28	8.67	13.31	16.43	15.28	5.46	9.11	12.95	13.56	10.60	10.15	14.12	9.46	10.63	12.50	10.21	8.73	9.53	9.73	12.02	5.77
39	D1479	1466	-90.3787	30.0443	10.48	11.03	11.30	14.26	14.68	6.24	11.55	11.67	5.93	14.09	14.53	7.34	9.09	10.36	8.77	13.29	16.72	15.31	5.33	8.87	12.79	13.52	10.45	10.27	14.14	9.32	10.63	12.45	10.18	8.62	9.97	9.65	12.52	5.71

St Charles Parish East Bank

Peak Surge 2007 modeling

ID	Point Set	Point Set ID	Longitude	Latitiude	9	14	15	17	18	22	23	24	25	26	27	35	36	49
					Maximum Surge (ft)													
1	Q835	164	-90.3623	30.0478	10.83	11.57	11.59	14.39	14.88	7.72	11.70	11.69	7.50	14.28	14.84	7.97	9.06	10.65
2	Q835	165	-90.3928	30.0117	10.68	11.43	11.68	14.24	14.64	7.76	12.45	12.40	7.42	14.51	14.81	8.84	10.12	10.88
3	Q835	263	-90.3900	30.0100	10.68	11.43	11.67	14.25	14.66	7.77	12.47	12.41	7.43	14.54	14.85	8.86	10.14	10.88
4	Q835	665	-90.2809	30.0075	8.26	8.72	9.47	11.23	12.01	9.56	12.43	11.96	9.26	15.34	15.23	9.56	10.72	8.63
5	Q835	666	-90.2812	30.0243	7.99	8.03	8.85	10.28	11.14	9.41	11.77	11.27	9.14	14.73	14.63	8.95	10.03	8.00
6	Q835	667	-90.2804	30.0396	8.03	7.97	8.66	10.01	10.86	8.83	10.91	10.52	8.58	13.79	13.76	8.05	9.10	7.77
7	Q835	673	-90.2837	29.9992	8.37	8.86	9.59	11.43	12.18	9.52	12.50	12.06	9.21	15.18	15.12	9.56	10.74	8.74
8	Q835	674	-90.3938	30.0086	10.68	11.44	11.68	14.25	14.63	7.76	12.46	12.41	7.43	14.50	14.78	8.85	10.13	10.88
9	Q835	675	-90.3589	29.9895	10.41	11.16	11.53	14.07	14.54	8.34	12.96	12.84	7.99	14.95	15.19	9.48	10.78	10.77
10	Q835	676	-90.3064	29.9770	8.98	9.58	10.22	12.38	12.99	9.39	12.91	12.62	9.06	14.62	14.68	9.87	11.11	9.35
11	Q835	678	-90.4213	30.0126	11.16	11.13	11.63	14.41	15.41	6.14	10.68	11.29	5.90	13.59	14.83	7.61	8.84	10.80
12	Q835	679	-90.4060	30.0240	11.11	11.16	11.55	14.36	15.21	6.18	10.39	11.01	5.94	13.18	14.39	7.17	8.40	10.64
13	D1479	220	-90.2825	30.0086	8.21	8.67	9.43	11.17	11.95	9.58	12.41	11.93	9.29	15.36	15.24	9.55	10.71	8.59
14	D1479	221	-90.2822	30.0172	7.97	8.19	9.01	10.52	11.36	9.60	12.08	11.57	9.32	15.13	14.99	9.29	10.40	8.18
15	D1479	222	-90.2817	30.0259	7.99	8.05	8.85	10.28	11.13	9.35	11.69	11.20	9.07	14.65	14.55	8.86	9.94	7.99
16	D1479	223	-90.2823	30.0351	8.01	8.04	8.78	10.18	11.02	8.97	11.20	10.78	8.71	14.09	14.04	8.33	9.39	7.90
17	D1479	224	-90.2824	30.0443	8.04	8.27	8.81	10.28	11.07	8.58	10.67	10.37	8.33	13.53	13.60	7.77	8.80	7.92
18	D1479	225	-90.2775	30.0509	8.07	8.28	8.74	10.23	11.01	8.32	10.25	10.04	8.07	13.04	13.18	7.42	8.38	7.83
19	D1479	226	-90.2671	30.0504	8.08	8.05	8.51	9.89	10.73	8.39	10.08	9.87	8.14	12.88	12.99	7.38	8.33	7.62
20	D1479	227	-90.2572	30.0493	8.08	7.84	8.29	9.60	10.46	8.47	9.94	9.73	8.24	12.76	12.83	7.36	8.31	7.61
21	D1479	238	-90.2894	29.9958	8.65	9.17	9.85	11.77	12.47	9.44	12.63	12.21	9.12	15.14	15.11	9.66	10.84	9.00
22	D1479	239	-90.2929	29.9922	8.74	9.29	9.95	11.95	12.62	9.42	12.71	12.33	9.10	14.99	14.98	9.71	10.90	9.10
23	D1479	240	-90.2995	29.9847	8.90	9.48	10.13	12.24	12.87	9.40	12.85	12.53	9.07	14.82	14.85	9.81	11.04	9.27
24	D1479	241	-90.3070	29.9787	8.97	9.57	10.21	12.38	12.99	9.39	12.91	12.62	9.06	14.71	14.77	9.87	11.11	9.34
25	D1479	242	-90.3166	29.9783	9.01	9.62	10.26	12.46	13.08	9.38	12.96	12.67	9.05	14.84	14.90	9.90	11.16	9.39
26	D1479	243	-90.3262	29.9823	9.73	10.41	10.89	13.23	13.83	9.06	13.23	12.99	8.71	15.45	15.60	10.03	11.32	10.12
27	D1479	244	-90.3359	29.9835	9.85	10.54	11.03	13.43	14.03	8.99	13.25	13.02	8.63	15.53	15.70	10.02	11.31	10.25
28	D1479	245	-90.3465	29.9865	10.19	10.92	11.36	13.91	14.45	8.73	13.21	13.03	8.39	15.38	15.59	9.87	11.18	10.60
29	D1479	246	-90.3558	29.9892	10.38	11.12	11.50	14.06	14.55	8.44	13.03	12.90	8.09	15.07	15.31	9.57	10.89	10.75
30	D1479	247	-90.3663	29.9920	10.47	11.22	11.56	14.09	14.54	8.12	12.80	12.70	7.77	14.80	15.05	9.26	10.56	10.80
31	D1479	248	-90.3744	29.9970	10.55	11.29	11.62	14.17	14.63	7.98	12.70	12.62	7.64	14.77	15.06	9.13	10.43	10.85
32	D1479	249	-90.3833	30.0035	10.66	11.41	11.65	14.25	14.69	7.84	12.55	12.49	7.49	14.63	14.94	8.95	10.23	10.87
33	D1479	250	-90.3908	30.0089	10.68	11.43	11.67	14.25	14.65	7.77	12.47	12.42	7.43	14.53	14.83	8.86	10.14	10.88
34	D1479	251	-90.3905	30.0155	10.68	11.43	11.67	14.25	14.68	7.74	12.42	12.37	7.40	14.53	14.86	8.81	10.08	10.87
35	D1479	252	-90.3942	30.0225	10.67	11.40	11.61	14.22	14.67	7.61	12.16	12.14	7.31	14.41	14.81	8.61	9.85	10.83
36	D1479	253	-90.2825	30.0029	8.32	8.80	9.54	11.34	12.10	9.54	12.48	12.02	9.23	15.25	15.18	9.56	10.73	8.69
37	D1479	1464	-90.3910	30.0289	10.73	11.47	11.66	14.32	14.81	7.59	12.18	12.16	7.29	14.49	14.92	8.58	9.84	10.86
38	D1479	1465	-90.3847	30.0367	10.84	11.59	11.71	14.49	14.99	7.51	12.14	12.12	7.20	14.52	14.99	8.47	9.73	10.89
39	D1479	1466	-90.3787	30.0443	10.95	11.69	11.74	14.59	15.07	7.39	12.00	12.01	7.08	14.46	14.97	8.29	9.54	10.90

ID	52	53	56	57	60	61	72	73	77	83	85	89	93	94	98	99	102	103	108	109
	Maximum Surge (ft)																			
1	9.78	13.13	17.27	15.56	5.60	8.70	13.14	13.64	10.58	10.73	14.38	9.20	10.61	12.60	10.34	8.77	11.74	11.15	13.92	7.54
2	9.96	13.66	15.70	15.17	6.87	10.18	13.83	13.98	11.61	10.44	14.37	10.16	10.99	13.10	10.74	9.61	10.85	11.30	13.53	7.44
3	9.95	13.66	15.78	15.22	6.88	10.21	13.86	13.99	11.63	10.44	14.39	10.18	10.99	13.10	10.75	9.62	10.83	11.31	13.52	7.44
4	7.56	11.30	14.17	14.41	10.15	12.11	15.23	12.71	11.66	8.47	12.24	10.57	9.27	11.90	9.69	9.44	6.86	11.71	12.01	9.32
5	6.92	10.28	13.19	13.37	10.05	11.44	14.64	12.02	10.84	8.31	11.49	9.93	8.65	11.15	9.18	8.82	6.52	11.22	11.57	9.00
6	6.73	9.60	12.57	12.49	8.99	10.21	13.48	11.15	9.80	8.38	11.24	9.13	8.40	10.50	8.60	8.08	7.14	10.48	10.89	8.36
7	7.67	11.45	14.30	14.44	10.05	12.10	15.06	12.81	11.74	8.55	12.37	10.59	9.37	12.02	9.77	9.50	6.99	11.75	12.15	9.30
8	9.96	13.66	15.60	15.12	6.87	10.19	13.84	13.99	11.62	10.44	14.37	10.17	10.99	13.10	10.75	9.62	10.85	11.31	13.53	7.44
9	9.80	13.68	16.13	15.39	7.93	11.16	14.37	14.13	12.28	10.15	14.31	10.75	10.99	13.34	10.85	10.09	10.22	11.67	13.22	8.22
10	8.33	12.44	14.50	14.40	9.78	12.26	14.46	13.27	12.35	9.02	13.06	10.97	10.01	12.72	10.29	10.00	7.74	11.97	12.61	9.26
11	8.92	13.08	15.00	15.07	5.87	7.72	11.13	13.41	9.95	11.29	15.16	9.20	10.90	12.93	10.69	8.77	9.47	8.88	10.08	5.54
12	8.92	12.76	14.94	14.85	5.47	7.25	10.86	12.98	9.50	11.19	14.79	8.83	10.63	12.63	10.32	8.39	10.16	8.97	10.80	5.74
13	7.51	11.25	14.14	14.40	10.20	12.12	15.26	12.68	11.64	8.43	12.19	10.57	9.24	11.87	9.66	9.41	6.81	11.69	11.98	9.33
14	7.09	10.63	13.41	13.85	10.31	11.87	15.08	12.31	11.23	8.29	11.67	10.27	8.85	11.43	9.37	9.11	6.44	11.48	11.72	9.24
15	6.92	10.23	13.16	13.30	9.94	11.32	14.53	11.96	10.74	8.32	11.48	9.85	8.63	11.10	9.13	8.76	6.55	11.15	11.53	8.94
16	6.85	9.88	12.87	12.80	9.29	10.60	13.84	11.47	10.14	8.36	11.38	9.38	8.51	10.76	8.81	8.33	6.91	10.70	11.18	8.54
17	7.01	9.79	12.71	12.58	8.10	9.62	13.06	11.02	9.54	8.40	11.34	8.89	8.46	10.47	8.51	7.93	7.88	10.28	10.72	8.12
18	7.07	9.69	12.65	12.33	7.31	8.89	12.42	10.69	9.12	8.44	11.22	8.54	8.40	10.25	8.29	7.65	8.11	10.03	10.65	7.79
19	6.90	9.45	12.38	12.12	7.44	8.88	12.27	10.37	9.00	8.44	10.95	8.50	8.24	10.04	8.09	7.58	7.89	9.88	10.48	7.79
20	6.74	9.23	12.13	11.93	7.62	8.92	12.16	10.07	8.90	8.44	10.72	8.49	8.08	9.87	7.91	7.52	7.66	9.75	10.33	7.82
21	7.97	11.76	14.51	14.56	9.90	12.14	15.00	12.97	11.91	8.77	12.62	10.70	9.62	12.23	9.95	9.67	7.40	11.82	12.27	9.26
22	8.07	11.96	14.50	14.51	9.85	12.16	14.84	13.05	12.03	8.84	12.75	10.77	9.73	12.37	10.05	9.76	7.52	11.85	12.38	9.25
23	8.25	12.28	14.56	14.49	9.81	12.23	14.66	13.21	12.24	8.96	12.96	10.90	9.92	12.60	10.21	9.92	7.68	11.93	12.54	9.26
24	8.32	12.44	14.57	14.46	9.78	12.26	14.54	13.28	12.35	9.02	13.06	10.97	10.01	12.71	10.28	10.00	7.74	11.97	12.61	9.26
25	8.36	12.54	14.76	14.62	9.75	12.28	14.62	13.34	12.43	9.05	13.14	11.02	10.07	12.80	10.32	10.05	7.77	11.98	12.64	9.25
26	9.20	13.15	15.88	15.54	9.20	12.16	15.03	13.83	12.72	9.60	13.75	11.19	10.60	13.30	10.67	10.37	9.03	12.04	12.90	9.04
27	9.32	13.31	16.12	15.71	9.06	12.09	15.04	13.95	12.72	9.69	13.90	11.20	10.70	13.38	10.72	10.38	9.21	12.01	13.01	8.98
28	9.66	13.63	16.32	15.74	8.60	11.73	14.83	14.15	12.64	9.96	14.24	11.09	10.92	13.47	10.84	10.34	9.80	11.91	13.19	8.73
29	9.78	13.68	16.15	15.51	8.09	11.30	14.49	14.15	12.38	10.12	14.32	10.83	10.98	13.39	10.85	10.16	10.14	11.73	13.20	8.36
30	9.84	13.66	15.88	15.29	7.55	10.84	14.19	14.09	12.06	10.22	14.32	10.55	10.99	13.24	10.83	9.93	10.39	11.55	13.27	7.89
31	9.90	13.70	15.97	15.35	7.30	10.63	14.12	14.10	11.93	10.29	14.38	10.43	11.02	13.23	10.83	9.84	10.54	11.48	13.36	7.68
32	9.93	13.67	15.93	15.30	6.99	10.35	13.95	14.04	11.73	10.41	14.41	10.26	11.00	13.15	10.78	9.69	10.76	11.37	13.47	7.51
33	9.95	13.66	15.72	15.19	6.89	10.21	13.86	13.99	11.63	10.44	14.38	10.18	10.99	13.11	10.75	9.63	10.84	11.31	13.53	7.44
34	9.94	13.64	15.90	15.28	6.84	10.13	13.81	13.97	11.57	10.45	14.39	10.13	10.98	13.08	10.73	9.58	10.86	11.27	13.52	7.42
35	9.92	13.52	16.03	15.32	6.66	9.81	13.53	13.81	11.29	10.47	14.36	9.92	10.91	12.93	10.63	9.42	10.89	11.09	13.40	7.31
36	7.62	11.39	14.25	14.44	10.09	12.11	15.14	12.77	11.71	8.51	12.32	10.59	9.33	11.97	9.74	9.48	6.92	11.73	12.10	9.31
37	9.94	13.56	16.34	15.50	6.63	9.78	13.56	13.88	11.28	10.51	14.44	9.91	10.93	12.95	10.65	9.40	10.98	11.10	13.46	7.29
38	9.96	13.56	16.82	15.70	6.44	9.63	13.52	13.90	11.20	10.62	14.53	9.83	10.93	12.93	10.64	9.33	11.25	11.05	13.58	7.20
39	9.97	13.53	17.16	15.75	6.20	9.34	13.36	13.87	11.01	10.73	14.57	9.66	10.92	12.87	10.60	9.19	11.59	10.97	13.83	7.07

St Charles Parish East Bank

Difference in Peak Surge updated modeling and 2007 modeling

Storm Number

ID	Point Set	Point Set ID	Longitude	Latitiude	9	14	15	17	18	22	23	24	25	26	27	35	36	49	52	53	56	57	60	61	72	73	77	83	85	89	93	94	98	99	102	103	108	109
					Difference in Maximum Surge (ft)																																	
1	Q835	164	-90.3623	30.0478	0.50	0.65	0.47	0.39	0.44	0.22	0.50	0.41	0.21	0.36	0.42	0.48	0.47	0.49	0.53	0.35	0.27	0.38	0.14	0.38	0.65	0.38	0.53	0.51	0.47	0.44	0.46	0.47	0.43	0.49	0.53	0.44	0.74	0.35
2	Q835	165	-90.3928	30.0117	0.54	0.83	0.59	0.28	0.23	1.34	0.55	0.41	1.32	0.27	0.27	0.99	0.54	0.72	1.41	0.27	0.17	0.21	1.23	0.61	0.51	0.30	0.67	0.46	0.30	0.41	0.34	0.45	0.43	0.64	1.85	1.38	1.49	1.60
3	Q835	263	-90.3900	30.0100	0.55	0.84	0.59	0.30	0.24	1.35	0.55	0.40	1.33	0.27	0.28	1.00	0.54	0.72	1.41	0.27	0.17	0.22	1.23	0.62	0.52	0.30	0.67	0.47	0.32	0.41	0.34	0.44	0.44	0.64	1.85	1.38	1.49	1.60
4	Q835	665	-90.2809	30.0075	0.70	0.85	0.55	0.32	0.28	0.49	0.42	0.33	0.32	0.16	0.18	0.73	0.53	0.67	0.47	0.37	0.29	0.15	0.72	0.59	0.30	0.16	0.49	0.46	0.33	0.47	0.38	0.38	0.37	0.57	-0.31	0.91	0.81	0.33
5	Q835	666	-90.2812	30.0243	0.50	0.67	0.51	0.28	0.26	0.42	0.37	0.32	0.31	0.23	0.21	0.66	0.50	0.61	0.36	0.36	0.10	0.12	0.69	0.54	0.31	0.18	0.50	0.52	0.38	0.46	0.35	0.36	0.38	0.53	-0.08	0.79	0.75	0.26
6	Q835	667	-90.2804	30.0396	0.50	0.53	0.47	0.30	0.29	0.34	0.34	0.30	0.28	0.21	0.24	0.52	0.44	0.53	0.43	0.36	0.11	0.16	0.59	0.44	0.31	0.16	0.43	0.54	0.39	0.41	0.37	0.38	0.37	0.48	0.17	0.56	0.58	0.27
7	Q835	673	-90.2837	29.9992	0.75	0.90	0.57	0.33	0.29	0.57	0.42	0.34	0.37	0.15	0.16	0.73	0.53	0.70	0.55	0.35	0.28	0.11	0.75	0.59	0.28	0.15	0.52	0.47	0.32	0.46	0.39	0.38	0.37	0.59	-0.23	0.92	0.83	0.41
8	Q835	674	-90.3938	30.0086	0.54	0.84	0.59	0.29	0.22	1.34	0.55	0.41	1.33	0.26	0.25	0.99	0.54	0.71	1.41	0.27	0.11	0.18	1.23	0.61	0.51	0.31	0.67	0.46	0.30	0.41	0.33	0.44	0.44	0.64	1.85	1.39	1.49	1.60
9	Q835	675	-90.3589	29.9895	0.61	0.99	0.63	0.36	0.24	1.52	0.60	0.45	1.52	0.26	0.28	1.01	0.57	0.80	1.52	0.28	0.29	0.19	1.38	0.67	0.48	0.31	0.71	0.45	0.31	0.45	0.35	0.42	0.43	0.68	2.01	1.35	1.37	1.76
10	Q835	676	-90.3064	29.9770	0.63	1.04	0.46	0.33	0.17	1.38	0.39	0.34	1.42	0.03	0.07	0.86	0.50	0.58	1.38	0.29	0.07	0.02	1.24	0.63	0.15	0.09	0.55	0.37	0.23	0.41	0.29	0.30	0.29	0.54	1.49	1.09	0.94	1.45
11	Q835	678	-90.4213	30.0126	0.52	0.58	0.51	0.59	0.48	0.67	0.57	0.52	0.64	0.50	0.44	0.49	0.47	0.51	0.61	0.51	0.75	0.55	0.79	0.58	0.52	0.42	0.47	0.45	0.40	0.41	0.41	0.46	0.39	0.46	0.81	0.61	0.75	0.52
12	Q835	679	-90.4060	30.0240	0.49	0.56	0.50	0.56	0.47	0.49	0.55	0.52	0.45	0.50	0.43	0.53	0.50	0.49	0.56	0.48	0.66	0.53	0.49	0.54	0.48	0.41	0.48	0.45	0.40	0.44	0.39	0.44	0.39	0.47	0.56	0.55	0.55	0.45
13	D1479	220	-90.2825	30.0086	0.68	0.83	0.55	0.31	0.27	0.48	0.42	0.33	0.32	0.17	0.19	0.72	0.52	0.66	0.43	0.36	0.30	0.11	0.72	0.59	0.31	0.16	0.49	0.44	0.31	0.48	0.37	0.38	0.37	0.56	-0.33	0.89	0.81	0.31
14	D1479	221	-90.2822	30.0172	0.49	0.72	0.53	0.30	0.26	0.42	0.39	0.33	0.29	0.18	0.19	0.68	0.51	0.64	0.32	0.38	0.10	0.10	0.68	0.56	0.32	0.18	0.50	0.52	0.31	0.47	0.36	0.37	0.38	0.55	-0.30	0.83	0.78	0.24
15	D1479	222	-90.2817	30.0259	0.49	0.67	0.51	0.28	0.25	0.41	0.36	0.31	0.30	0.19	0.21	0.65	0.50	0.60	0.38	0.35	0.09	0.12	0.69	0.53	0.31	0.18	0.49	0.53	0.37	0.45	0.35	0.36	0.37	0.53	-0.09	0.77	0.74	0.27
16	D1479	223	-90.2823	30.0351	0.49	0.59	0.49	0.29	0.27	0.35	0.35	0.31	0.27	0.19	0.22	0.58	0.46	0.57	0.45	0.34	0.10	0.14	0.64	0.48	0.32	0.17	0.45	0.54	0.38	0.43	0.37	0.38	0.36	0.50	0.04	0.62	0.64	0.26
17	D1479	224	-90.2824	30.0443	0.49	0.50	0.48	0.40	0.37	0.33	0.38	0.32	0.29	0.24	0.28	0.45	0.43	0.51	0.44	0.40	0.25	0.26	0.39	0.37	0.31	0.18	0.43	0.55	0.43	0.41	0.40	0.40	0.37	0.47	0.37	0.43	0.54	0.31
18	D1479	225	-90.2775	30.0509	0.49	0.37	0.41	0.44	0.40	0.36	0.40	0.38	0.33	0.29	0.33	0.41	0.42	0.41	0.37	0.38	0.34	0.29	0.36	0.35	0.31	0.28	0.36	0.55	0.45	0.42	0.39	0.42	0.38	0.45	0.33	0.36	0.34	0.36
19	D1479	226	-90.2671	30.0504	0.49	0.36	0.41	0.44	0.41	0.36	0.40	0.39	0.33	0.30	0.33	0.40	0.42	0.42	0.37	0.39	0.35	0.30	0.36	0.34	0.31	0.32	0.36	0.55	0.44	0.42	0.40	0.42	0.39	0.46	0.34	0.35	0.33	0.36
20	D1479	227	-90.2572	30.0493	0.49	0.36	0.41	0.43	0.42	0.36	0.40	0.40	0.33	0.31	0.34	0.40	0.43	0.46	0.37	0.40	0.36	0.30	0.36	0.34	0.31	0.33	0.35	0.55	0.44	0.43	0.41	0.43	0.40	0.46	0.34	0.35	0.33	0.36
21	D1479	238	-90.2894	29.9958	0.56	0.91	0.48	0.34	0.28	1.12	0.42	0.33	1.14	0.22	0.23	0.79	0.52	0.55	1.14	0.33	0.26	0.14	1.00	0.60	0.35	0.12	0.52	0.35	0.31	0.44	0.30	0.36	0.30	0.53	1.07	1.00	0.83	1.14
22	D1479	239	-90.2929	29.9922	0.60	0.98	0.49	0.36	0.26	1.19	0.42	0.35	1.21	0.18	0.19	0.82	0.51	0.59	1.23	0.34	0.21	0.12	1.05	0.61	0.30	0.10	0.54	0.38	0.30	0.44	0.32	0.36	0.31	0.56	1.20	1.01	0.86	1.22
23	D1479	240	-90.2995	29.9847	0.63	1.03	0.48	0.36	0.21	1.32	0.41	0.36	1.35	0.14	0.16	0.85	0.51	0.60	1.35	0.32	0.17	0.10	1.19	0.63	0.26	0.11	0.56	0.38	0.26	0.43	0.31	0.34	0.30	0.56	1.40	1.06	0.91	1.39
24	D1479	241	-90.3070	29.9787	0.62	1.03	0.46	0.33	0.17	1.38	0.40	0.34	1.41	0.09	0.13	0.86	0.50	0.57	1.37	0.29	0.10	0.05	1.24	0.64	0.21	0.10	0.55	0.37	0.23	0.41	0.29	0.29	0.28	0.54	1.49	1.09	0.94	1.45
25	D1479	242	-90.3166	29.9783	0.60	1.02	0.43	0.29	0.14	1.42	0.38	0.30	1.45	0.05	0.07	0.85	0.48	0.55	1.37	0.23	0.05	0.01	1.26	0.63	0.20	0.07	0.53	0.35	0.21	0.38	0.26	0.26	0.25	0.51	1.54	1.09	0.95	1.47
26	D1479	243	-90.3262	29.9823	0.85	1.31	0.69	0.53	0.41	1.30	0.58	0.48	1.32	0.36	0.40	0.97	0.60	0.87	1.72	0.46	0.46	0.37	1.05	0.63	0.50	0.32	0.72	0.56	0.46	0.48	0.50	0.48	0.46	0.71	2.41	1.19	1.17	1.44
27	D1479	244	-90.3359	29.9835	0.75	1.19	0.64	0.49	0.36	1.40	0.59	0.45	1.40	0.32	0.36	0.98	0.59	0.82	1.65	0.39	0.38	0.31	1.20	0.66	0.50	0.31	0.70	0.50	0.40	0.48	0.43	0.43	0.41	0.69	2.19	1.19	1.24	1.56
28	D1479	245	-90.3465	29.9865	0.67	1.09	0.64	0.46	0.30	1.44	0.60	0.45	1.46	0.30	0.33	1.00	0.59	0.82	1.60	0.31	0.30	0.27	1.28	0.65	0.49	0.33	0.70	0.46	0.36	0.47	0.39	0.42	0.42	0.68	2.10	1.24	1.38	1.65
29	D1479	246	-90.3558	29.9892	0.63	1.01	0.63	0.38	0.26	1.50	0.60	0.45	1.50	0.28	0.31	1.00	0.58	0.81	1.53	0.28	0.24	0.23	1.34	0.66	0.49	0.32	0.71	0.45	0.33	0.45	0.35	0.42	0.42	0.68	2.04	1.33	1.37	1.73
30	D1479	247	-90.3663	29.9920	0.58	0.93	0.62	0.33	0.23	1.53	0.60	0.44	1.51	0.27	0.28	1.02	0.57	0.79	1.50	0.26	0.17	0.21	1.41	0.68	0.49	0.30	0.71	0.45	0.31	0.45	0.35	0.43	0.43	0.67	1.92	1.41	1.36	1.81
31	D1479	248	-90.3744	29.9970	0.58	0.89	0.62	0.34	0.25	1.47	0.58	0.43	1.46	0.29	0.32	1.02	0.57	0.78	1.49	0.28	0.22	0.25	1.39	0.66	0.51	0.32	0.71	0.45	0.32	0.44	0.35	0.45	0.44	0.67	1.91	1.43	1.40	1.76
32	D1479	249	-90.3833	30.0035	0.55	0.86	0.58	0.30	0.25	1.40	0.56	0.42	1.37	0.29	0.31	1.01	0.55	0.72	1.41	0.26	0.19	0.23	1.29	0.64	0.51	0.32	0.68	0.46	0.33	0.42	0.34	0.44	0.44	0.64	1.88	1.41	1.45	1.66
33	D1479	250	-90.3908	30.0089	0.55	0.84	0.58	0.29	0.23	1.35	0.55	0.41	1.33	0.27	0.27	1.00	0.54	0.72	1.41	0.27	0.15	0.21	1.24	0.62	0.51	0.30	0.67	0.47	0.31	0.41	0.34	0.45	0.44	0.64	1.86	1.38	1.49	1.60
34	D1479	251	-90.3905	30.0155	0.53	0.83	0.59	0.30	0.25	1.33	0.54	0.40	1.31	0.29	0.30	0.99	0.53	0.70	1.39	0.27	0.23	0.26	1.21	0.61	0.53	0.31	0.67	0.46	0.32	0.41	0.33	0.45	0.43	0.63	1.83	1.37	1.50	1.59
35	D1479	252	-90.3942	30.0225	0.51	0.78	0.55	0.33	0.30	1.24	0.49	0.37	1.26	0.35	0.34	0.96	0.50	0.67	1.34	0.26	0.38	0.31	1.12	0.55	0.54	0.32	0.62	0.45	0.33	0.38	0.31	0.43	0.42	0.61	1.74	1.31	1.48	1.51
36	D1479	253	-90.2825	30.0029	0.73	0.88	0.57	0.33	0.28	0.54	0.43	0.34	0.35	0.15	0.17	0.73	0.53	0.69	0.51	0.36	0.29	0.12	0.73	0.59	0.28	0.15	0.51	0.46	0.33	0.47	0.39	0.38	0.38	0.58	-0.29	0.91	0.83	0.38
37	D1479	1464	-90.3910	30.0289	0.51	0.79	0.55	0.33	0.33	1.23	0.50	0.37	1.25	0.35	0.38	0.95	0.51	0.66	1.34	0.26	0.34	0.36	1.10	0.54	0.55	0.33	0.62	0.46	0.37	0.38	0.31	0.43	0.43	0.60	1.75	1.32	1.52	1.49
38	D1479	1465	-90.3847	30.0367	0.50	0.74	0.50	0.35	0.38	1.19	0.49	0.36	1.20	0.37	0.43	0.96	0.48	0.61	1.29	0.25	0.39	0.42	0.98	0.52	0.57	0.34	0.60	0.47	0.41	0.37	0.30	0.43	0.43	0.60	1.72	1.32	1.56	1.43
39	D1479	1466	-90.3787	30.0443	0.47	0.66	0.44	0.33	0.39	1.15	0.45	0.34	1.15	0.37	0.44	0.95	0.45	0.54	1.20	0.24	0.44	0.44	0.87	0.47	0.57	0.35	0.56	0.46	0.43	0.34	0.29	0.42	0.42	0.57	1.62	1.32	1.31	1.36

Appendix R

Sensitivity Analysis On Storm Surge Modeling Results for Lake Pontchartrain and Vicinity (LPV), West Bank and Vicinity (WBV) and New Orleans to Venice (NOV)/Non-Federal Levee (NFL) Incorporation into NOV Hurricane Storm Damage and Risk Reduction (HSDRRS) Projects

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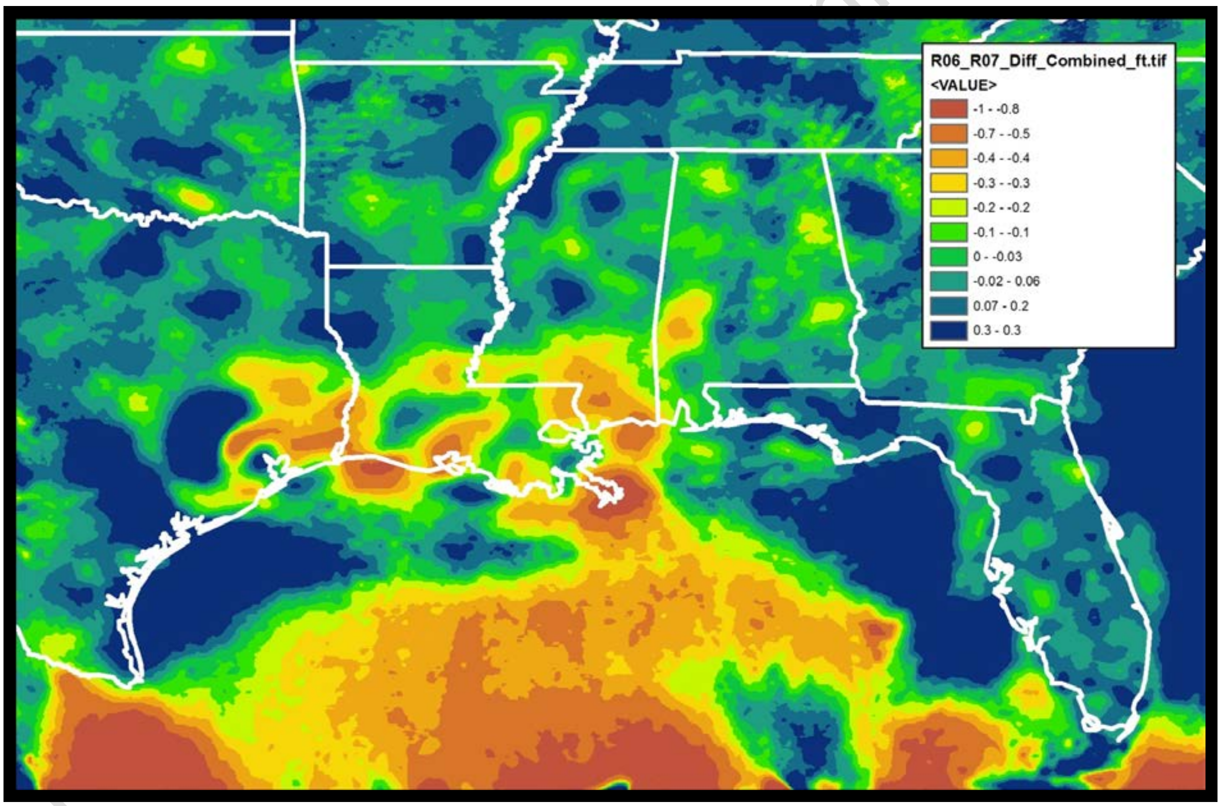
Sensitivity Analysis on Storm Surge Modeling Results

for the Lake Pontchartrain and Vicinity (LPV),
West Bank and Vicinity (WBV) and
New Orleans to Venice (NOV)/Non-Federal Levee (NFL) Incorporation into NOV
Hurricane Storm Damage and Risk Reduction (HSDRRS) Projects

due to the Vertical Datum Update
from NAVD88 (2004.65) to NAVD88 (2009.55)

FINAL
July 2014



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<u>On the Cover:</u> Graphic depicting change in elevation between Geoid03 and Geoid12A

<u>Bottom Line Up Front (BLUF):</u>

- <u>Purpose:</u> A sensitivity analysis on storm surge modeling results was performed, using a subset of 18 storms in ADCIRC, to determine the potential impact of the vertical datum update from NAVD88 2004.65 to NAVD88 2009.55 to the published design elevations for LPV, WBV and NOV/NFL Projects.
- A major consideration in the sensitivity analysis was definition of the appropriate starting water surface elevation for the Gulf of Mexico in the ADCIRC simulations. It was determined that the appropriate starting water surface elevation is +0.97 ft NAVD88 2009.55. This is 0.23 ft lower than the original ADCIRC modeling which used +1.2 ft NAVD88 2004.65.
- <u>Results:</u>
  - o The simulation of 18 storms on the NAVD88 2009.55 grid computes surge elevations that are lower in direct relationship to the lower starting water surface elevation. Specifically, surge elevations relative to NAVD88 2009.55 are approximately -0.2 to -0.4 ft different (lower) than surge elevations relative to NAVD88 2004.65 coastwide.
  - o When the datum difference at a specific location is similar to the shift in starting water surface elevation (as is the case for LPV, WBV and the NFL portion of NOV), there is no requirement to add height to the published design elevations to achieve the intended level of risk reduction (LORR).
  - o When the datum difference at a specific location is greater than the shift in starting water surface elevation (as is the case for some of NOV), there is a required to add height to the published design elevations to achieve the intended level of risk reduction (LORR).
- <u>Recommendations:</u>
  - o Since this analysis was not a comprehensive update considering all changed parameters, MVN H&H Branch does not endorse revising the published design elevations for the WBV, LPV or NOV/NFL projects from their values published in the October 2007 Design Elevation Report (DER) and updates to the DER.
    - ▪ For LPV, WBV and the NFL portion of NOV, the published design elevations achieve the intended LORR.
    - ▪ For some NOV Project features; however, this is not the case. MVN H&H Branch strongly recommends adding height to the structures (at an amount equal to the negative change in datum) to ensure that the 2063 intended LORR is achieved.
  - o For NOV/NFL Projects, add a table to the 2014 update to the Design Elevation Report to convert the published design elevations for design and construction contract reaches from NAVD88 2004.65 to NAVD88 2009.55.
  - o Given the large amount of uncertainty in any surge hazard analysis, it is important to re-analyze the complete system and conduct new analyses using the latest data, the latest technology and the most sound methodology on a recurring interval of 10 years or after a major storm event.

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1. Executive Summary

This document presents the results of a sensitivity analysis, completed by MVN H&H Branch (CEMVN-ED-H) and Design Services Branch, Survey Section (CEMVN-ED-S), on Storm Surge Modeling Results for the Lake Pontchartrain and Vicinity (LPV), West Bank and Vicinity (WBV) and New Orleans to Venice (NOV)/Non-Federal Levee (NFL) Incorporation into NOV Hurricane Storm Damage and Risk Reduction (HSDRRS) Projects due to the Vertical Datum Update from NAVD88 (2004.65) to NAVD88 (2009.55). The analysis and findings were technically reviewed by subject matter experts with the Army Geospatial Center (AGC), the Engineering Research and Development Center (ERDC) Coastal Hydraulics Laboratory (CHL), the University of North Carolina (UNC), and ARCADIS U.S., Inc.

The purpose of the sensitivity analysis was to run a subset of storms in ADCIRC to determine the potential impacts of the vertical datum update from NAVD88 2004.65 to NAVD88 2009.55 to the published design elevations for LPV, WBV and NOV/NFL projects. The published design elevations for LPV, WBV and NOV/NFL are the design elevations published in the October 2007 Design Elevation Report (DER), including updates to the DER, and utilized for design and construction of the LPV, WBV and NOV/NFL projects.

For the hydraulic design of hurricane and storm damage risk reduction system (HSDRRS) projects and other coastal projects in Southeast Louisiana, the vertical reference of the various versions of the SL15 ADCIRC grid were set to NAVD88 2004.65. Recently, a new vertical reference NAVD88 2009.55 was established. In order to assess the impacts to stage frequency curves and design elevations, the SL15v7 ADCIRC grid was adjusted to the NAVD88 2009.55 datum/epoch. The adjustment to bathymetry/topography and raised features elevations was based on the difference between Geoid03 and Geoid12a, which is roughly the same as the difference between the NAVD88 2004.65 and NAVD88 2009.55 datum/epochs because Geoid03 aligns with NAVD88 2004.65 and Geoid12a aligns with NAVD88 2009.55.

The starting water surface elevation was also adjusted from NAVD88 2004.65 to the NAVD88 2009.55 datum/epoch. For the original ADCIRC modeling done to developed the published design elevations for LPV, WBV and NOV/NFL, the starting water surface elevation in the 2004.65 epoch was set to +1.2 ft NAVD88, based on analysis of 11 coastal water level gages. Re-computing Local Mean Sea Level (LMSL) in NAVD88 2009.55 at the same gages yields a lower LMSL and therefore a lower value for the starting water surface elevation. Because there are several methods for adjusting the starting water surface elevation to the NAVD88 2009.55 datum/epoch and there is considerable uncertainty in the original computation, multiple values were calculated and applied to assess the sensitivity of the ADCIRC modeling results to the starting value. The values selected were +0.82 ft, +0.97 ft, and +1.2 ft NAVD88 2009.55, with +0.97 ft as the best approximation of the starting water surface that is most consistent with the approach used in the original modeling.

The results show that surge elevations tend to lower along with, but not necessarily 1:1 with, the generally downward shift between the 2004.65 to the 2009.55 epochs. The simulation of 18 storms on the NAVD88 2009.55 grid computes surge elevations that are lower in direct relationship to the lower starting water surface elevation. Specifically, surge elevations relative to NAVD88 2009.55 are approximately -0.2 to -0.4 ft different (lower) than surge elevations relative to NAVD88 2004.65 coastwide.

Since this analysis only updated surge modeling results from a subset of storms using this single changed parameter (and was not a comprehensive update considering all changed parameters), MVN H&H Branch does not endorse revising the published design elevations for the WBV, LPV or NOV/NFL projects from their published values. Until a comprehensive approach is undertaken to update all changed parameters, the published design elevations for LPV, WBV and NOV/NFL will remain as published. For some NOV Project features, however, MVN H&H Branch strongly recommends adding height to the structures (at an amount equal to the change in datum) to ensure that the 2063 intended LORR is achieved.

When assessing the findings of this analysis, it is most important to consider sources of error and other assumptions in the overall JPM-OS surge hazard analysis conducted for design of LPV, WBV and NOV/NFL. There are many sources of uncertainty which are most likely much larger than the changes to design elevation estimated in this analysis. For example, the ADCIRC model does not have the ability to reliably predict surge accurately to within a few tenths of a foot. Hindcasts of Katrina, Rita, Gustav, Ike and Isaac reveal that ADCIRC usually predicts surge to within +/- 1.5 ft. There are many sources of error in the ADCIRC modeling including: errors in the applied wind-field and wind-stress equations, errors in bathymetric elevations/datums, errors in the nodal attributes (bottom friction, wind canopy coefficients, etc), errors due to lack of resolution, and many more limitations.

In the overall H&H levee design approach, conservancy in estimates such as overbuild, subsidence rates, sea level rise rates, surveying accuracy, steric water level adjustment, local bathymetric changes, shoreline erosion, vegetation changes and other issues are just as criticial, if not more critical, than the new geoid update. Other conservancy concerning the surge hazard analysis and design procedures could potentially result in changes to required design elevation that are greater than the changes calculated in this analysis. Given the large amount of uncertainty in any surge hazard analysis, it is important to re-analyze the complete system and conduct a new analysis for coastal Louisiana using the latest data, the latest technology and the most sound methodology on a recurring interval of 10 years or after a major storm event, as expressed in the district's Engineering Division Datum Policy Memo #2.

2. Introduction and Background

Figure 1 displays the elevation difference between the NAVD88 2004.65 and NAVD88 2009.55 datum-epochs at NGS benchmarks in Southern Louisiana. At points with warm colors (red, orange and yellow), the 2009.55 datum sits above the 2004.65 datum, so elevations referenced to the 2009.55 datum are lower than those referenced to the 2004.65 datum for the same land surface. It is important to note that the datum shift varies spatially considerably. For example, the shift is -1.05 ft in Lower Plaquemines and +0.16 ft at Caernarvon Marsh. If the datum shift were constant spatially, there would be no cause for concern, because the new datum would only change relative elevations, leaving the absolute levee or structures vertical locations the same.

Figure 1 also displays a surface of the difference between Geoid03 and Geoid12A[1]. The updated geoid is the primary factor which contributes to the difference between the epochs. As can be seen in the graphic, the difference between NAVD88 2004.65 and NAVD88 2009.55 is approximately the same as the difference between the geoids. The difference between NAVD88 2004.65 and NAVD88 2009.55 includes a small amount of subsidence or monument shift, which is not expressed in the difference between the old and new geoids. In Figure 1, at the approximate location of Venice, the difference between NAVD88 2004.65 and NAVD88 2009.55 is labeled as -1.00 to -1.05 ft, while the difference in geoids at the same location is labeled as -0.9 to -0.99 ft.

The new realization of NAVD88 has caused concern for USACE-MVN since LPV, WBV and NOV/NFL Projects have published design elevations in the NAVD88 2004.65 datum-epoch. For example, as pictured in Figure 2, a levee as part of a HSDRRS might have published design elevation of 17.0 ft NAVD88 2004.65. The initial response by MVN was to reset the elevation "value" the same for both epochs. For example, if the design elevation was 17.0 ft NAVD88 2004.65, the design elevation would become 17.0 ft NAVD88 2009.55. Using this approach for an example where the epoch difference is -0.5 ft, the design elevation would actually become equivalent to 17.5 ft NAVD88 2004.65, and the levee would have to be raised 0.5 ft, as can be seen in the right portion of Figure 2. In areas where the NAVD88 2009.55 is lower than NAVD88 2004.65, this approach would require the structures to be raised by the negative difference between the datums. Since the starting water surface elevation also shifts (but not necessarily at the same magnitude in all locations) with the new datum, this conservative approach will result in a LORR equal to or greater than the intended LORR. This conservative approach is the one the NOV PDT is using to most expeditiously move forward with some structural features already under advanced design/construction.

Figure 3 displays the same example levee with an original published design elevation of 17.0 ft NAVD88 2004.65. With the datum adjustment at this location, the new design elevation becomes 16.5 ft NAVD88 2009.55 because the water level and land are both

[1] The GEOID03 and GEOID12A grids were downloaded from the National Geodetic Survey respective geoid pages: http://www.ngs.noaa.gov/GEOID/GEOID12A/GEOID12A_CONUS.shtml and http://www.ngs.noaa.gov/PC_PROD/GEOID03/
The difference between the geoid surfaces were calculated and are displayed in Figure 1.

shifted with the datum shift. Note that in this example, the starting water surface elevation and land are shifted the same amount, which is not always the case. In other words, the applied starting water surface elevation might not have exactly the same shift as the datum. In this example, there is no requirement to raise the levee since the same freeboard exists for both datums and the levee provides the same LORR. The analysis described in this report will estimate how design levels shift at various locations when the new epoch conversion is applied.

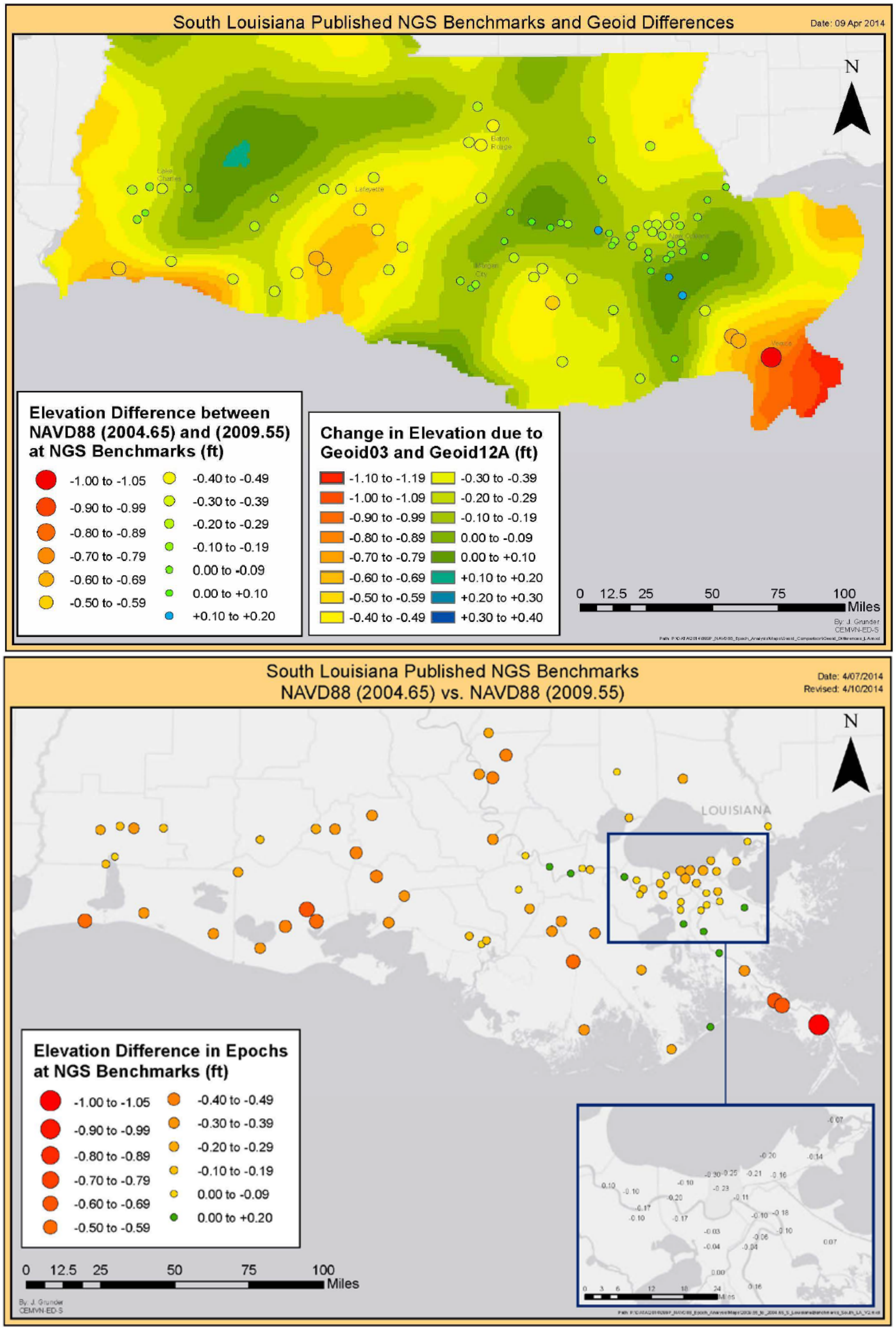

Figure 1 Elevation Differences Between NAVD88 2004.65 and NAVD88 2009.55 at NGS Benchmarks (the difference is computed by subtracting 2004.65 from 2009.55)

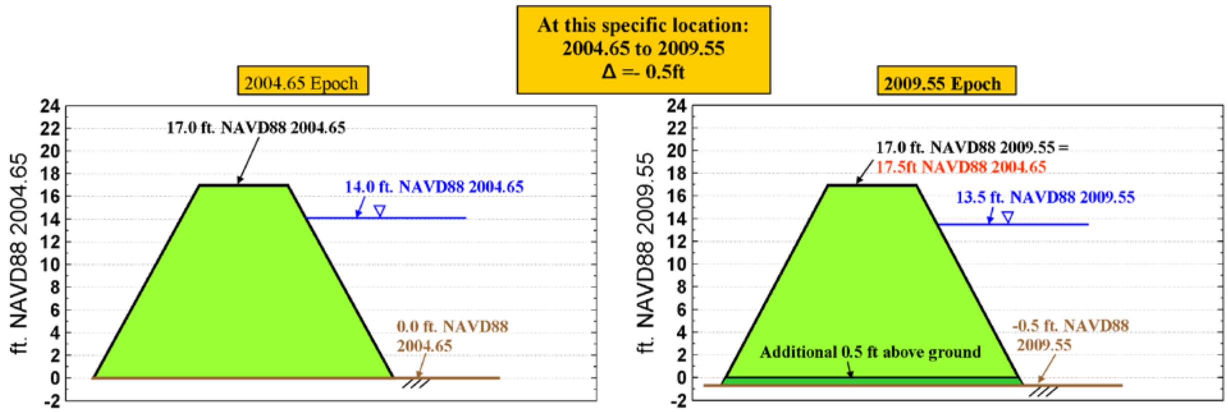

Figure 2 Levee design comparison NAVD88 2004.65 vs NAVD88 2009.55 assuming design elevation 17.0 ft NAVD88 2004.65 = 17.0 ft NAVD88 2009.55.

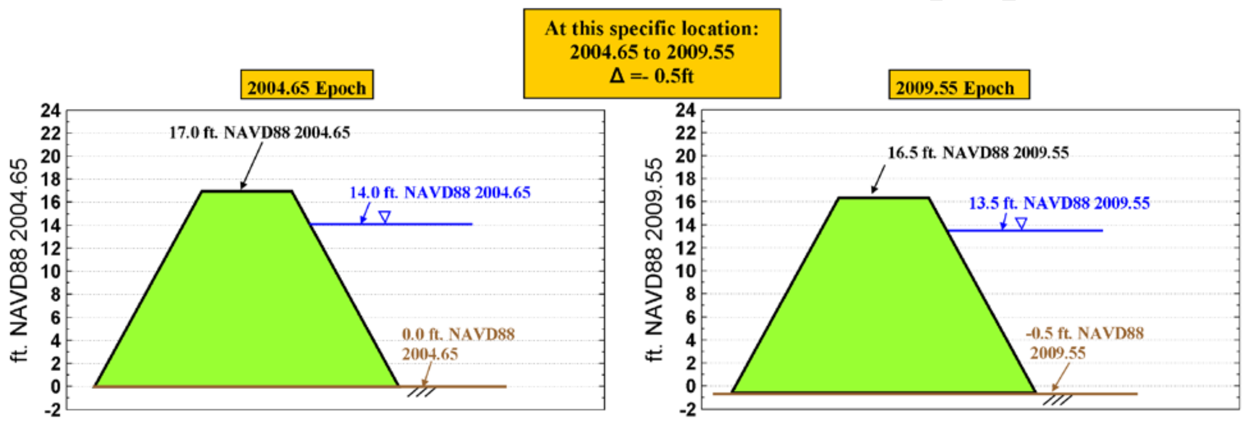

Figure 3 Levee design comparison NAVD88 2004.65 vs NAVD88 2009.55 assuming the water level and land shift with the new datum-epoch. Note that in this example, the water and land are shifted by the same amount, which is not always the case. In some cases, the datum shift is a greater amount than the shift in starting water level.

3. Methodology

3.1 Synthetic Storms

In this analysis, we have selected and evaluated an 18 storm subset of the original 152 synthetic storms used in design of LPV and WBV projects and other projects in coastal Louisiana. The selected storms are: S014, S015, S017, S018, S023, S024, S026, S027, S032, S035, S052, S053, S056, S057, S069, S073, S077, and S085. These storms were selected because they produce a range of stages from 2% (50-year) to 0.5% (500-yr) throughout the LPV, WBV and NOV/NFL Projects. This same subset of storms was processed for the original analysis of the collocated LPV-MRL and WBV-MRL and the NOV/NFL levees and floodwalls. The tracks of these selected synthetic storms are plotted as grey lines along with tracks of major historical storms, which are colored lines in Figure 4. Figure 5 displays windspeed versus distance from landfall for the subset of synthetic and historical storms.

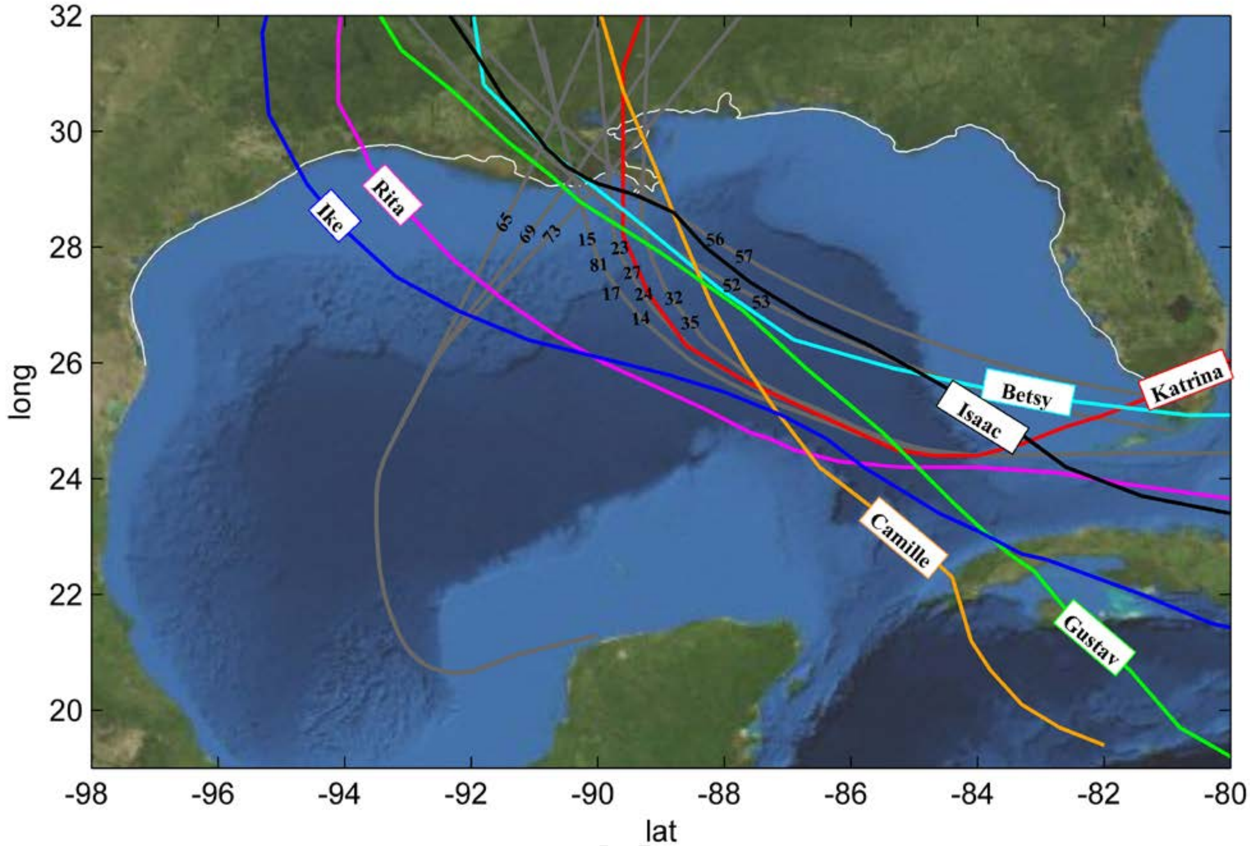

Figure 4 Comparison of 18 synthetic storm tracks and historical storm tracks. (Grey lines represent synthetic storm tracks, colored lines represent historical storms)

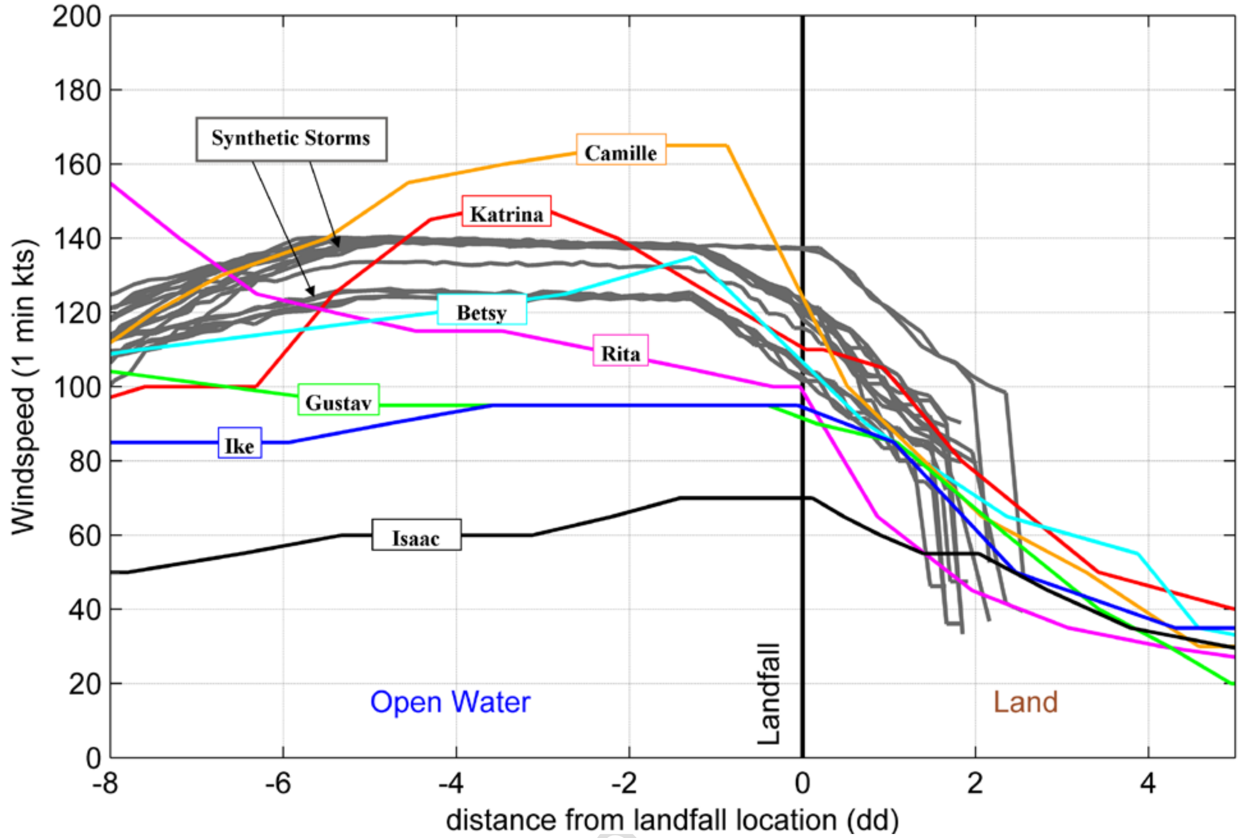

Figure 5 Distance from landfall vs windspeed of synthetic storms and historical storms. (Grey lines represent synthetic storm tracks, colored lines represent historical storms)

3.2 Grid Bathymetry, Topography and Raised Feature Modifications

For the hydraulic design of hurricane and storm damage risk reduction system (HSDRRS) projects and other coastal projects in Southeast Louisiana, the vertical reference of the various versions of the SL15 ADCIRC grid were set to NAVD88 2004.65. Recently, a new vertical reference NAVD88 2009.55 was established. In order to assess the impacts to stage frequency curves and design elevations, the SL15v7 ADCIRC grid was adjusted to the NAVD88 2009.55 datum/epoch. The adjustment to bathymetry/topography and raised features elevations was based on the difference between Geoid03 and Geoid12a, which is roughly the same as the difference between the NAVD88 2004.65 and NAVD88 2009.55 datum/epochs because Geoid03 aligns with NAVD88 2004.65 and Geoid12a aligns with NAVD88 2009.55.

The SL15v7 ADCIRC grid has been updated to include LPV and WBV Project features including the IHNC Surge Barrier, the WCC, the 3 Lakefront canal Interim Closure Structures, the MRGO Closure Structure, and the Seabrook Floodgate. The most recent levee elevation surveys in NAVD88 2004.65 were applied to the mesh using a nearest node approach. For previous LPV and WBV ADCIRC modeling, the levee elevations for various project features were set to non-overtopping. In this analysis we have set the levee heights to the most recent survey elevations, which may produce slightly different surge results on a storm by storm basis when comparing to the older LPV/WBV

modeling results, depending on whether or not the levees are overtopped. This updated version of the grid represents existing 2014 conditions in the NAVD88 2004.65 datum. This updated version of SL15v7 is also labeled simply as the "MRL grid" in some of the figures in this document. The NAVD88 2004.65 grid bathymetry/topography is displayed in Figure
6.

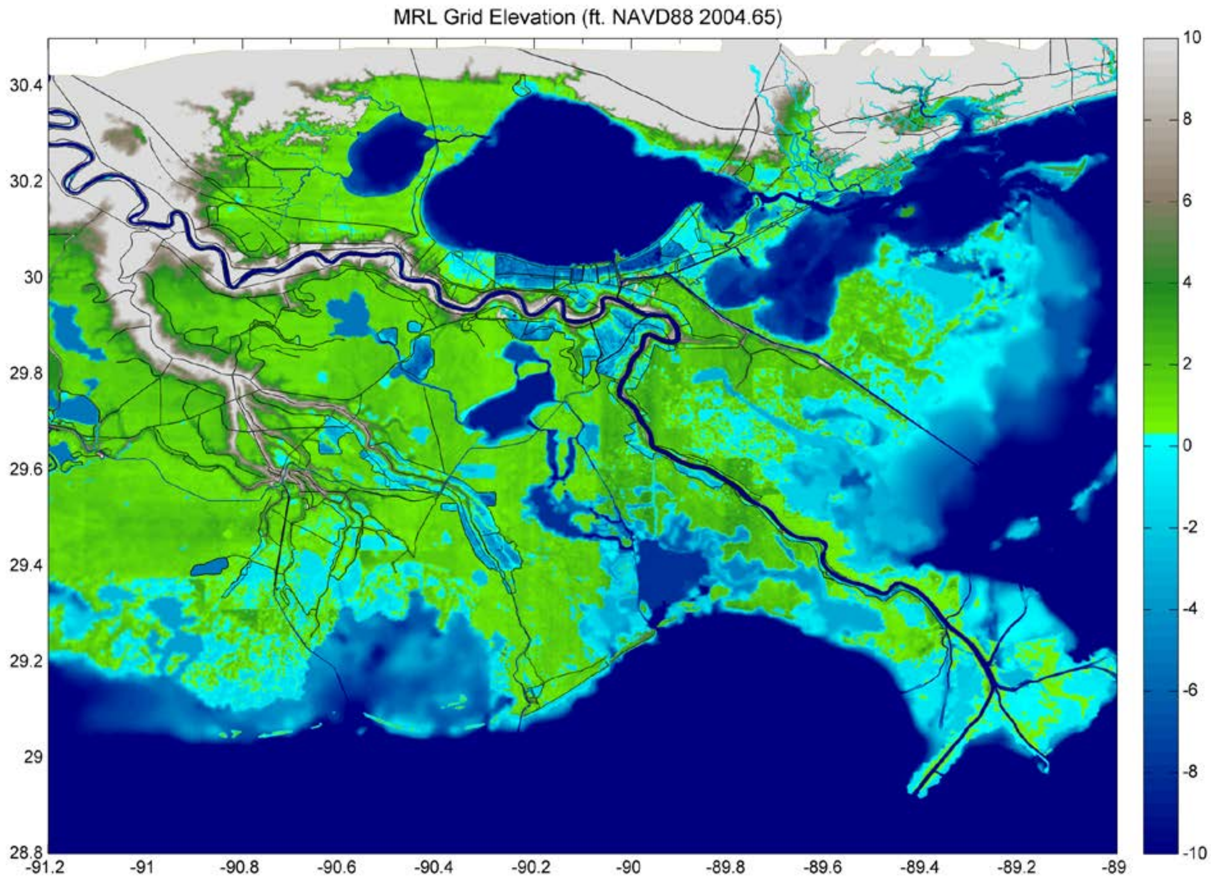

Figure 6 SL15v7 ADCIRC grid elevation in ft NAVD88 2004.65

The updated SL15v7 grid was then approximately adjusted to the NAVD88 2009.55 datum. The ADCIRC model bathymetry/topography and raised feature elevations were adjusted to the new NAVD88 2009.55 using the difference in geoid for the entire Gulf of Mexico. The change in geoid is displayed in Figure 7 (geoid differences shown in Figure 1 are subset of differences shown in Figure 7). Warmer colors indicate that the 2004.65 datum is higher than the 2009.55 datum. While the change in geoid is not exactly the same as the change in datum, the updated geoid accounts for the majority of the datum shift. It would be difficult to warp the grid using the benchmark data because there are not enough points to warp the entire surface without significant extrapolation and interpolation. Therefore, some very mirror error in the new elevations is expected in the 2009.55 grid. The NAVD88 2009.55 grid bathymetry/topography is displayed in Figure 8. A difference plot between the NAVD88 2004.65 and NAVD88 2009.55 grid elevations is displayed in Figure 9. Figure 9 also displays the location of the various levee design segments which are analyzed. The datum shift varies in different locations, and each project feature can be affected differently.

No changes to nodal attributes (bottom friction, canopy coefficients, etc) were made. The nodal attributes, including bottom friction and canopy, are predominately assigned using land coverage data, which should not be impacted by the new epoch.

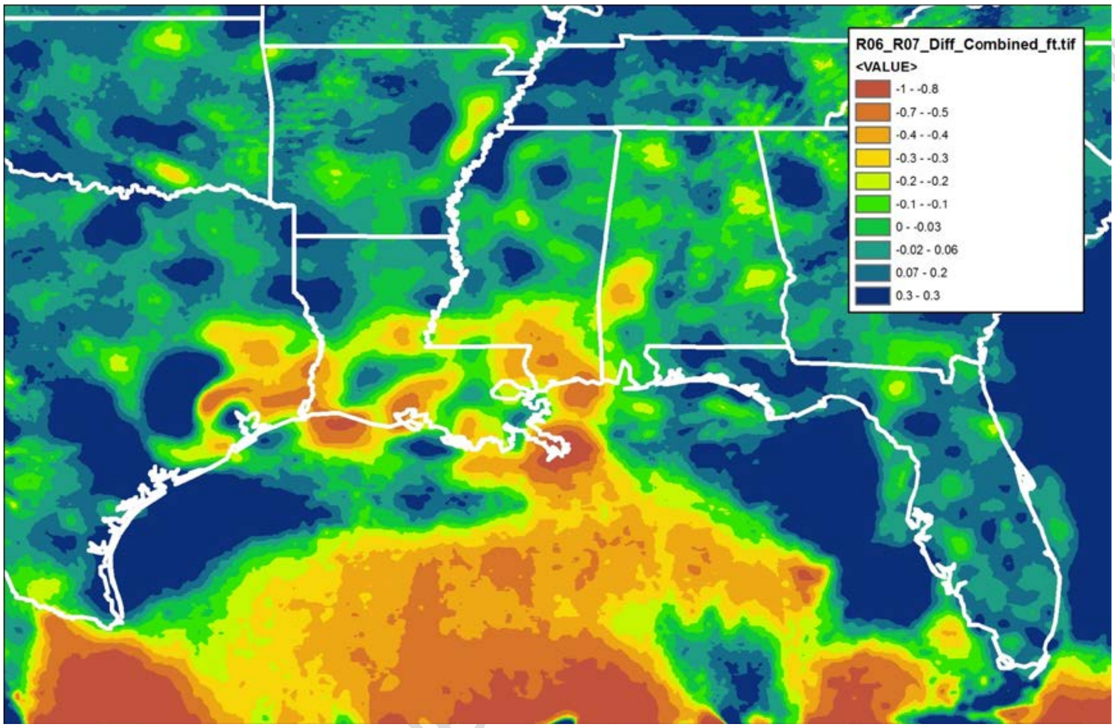

Figure 7 Change in elevation due to Geoid03 and Geoid12A (ft) (the difference is computed by subtracting Geoid12A from Geoid03)

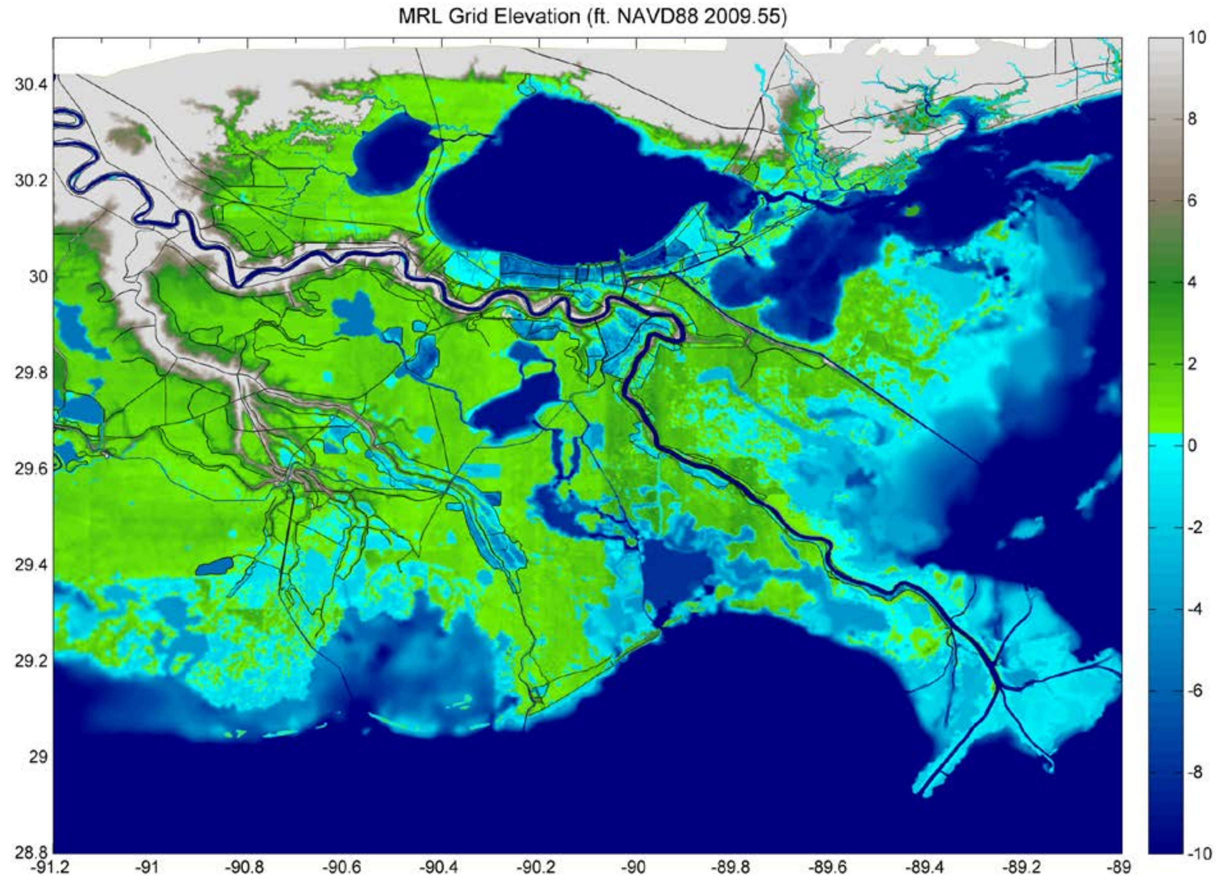

Figure 8 SL15v7 ADCIRC grid elevation in ft NAVD88 2009.55

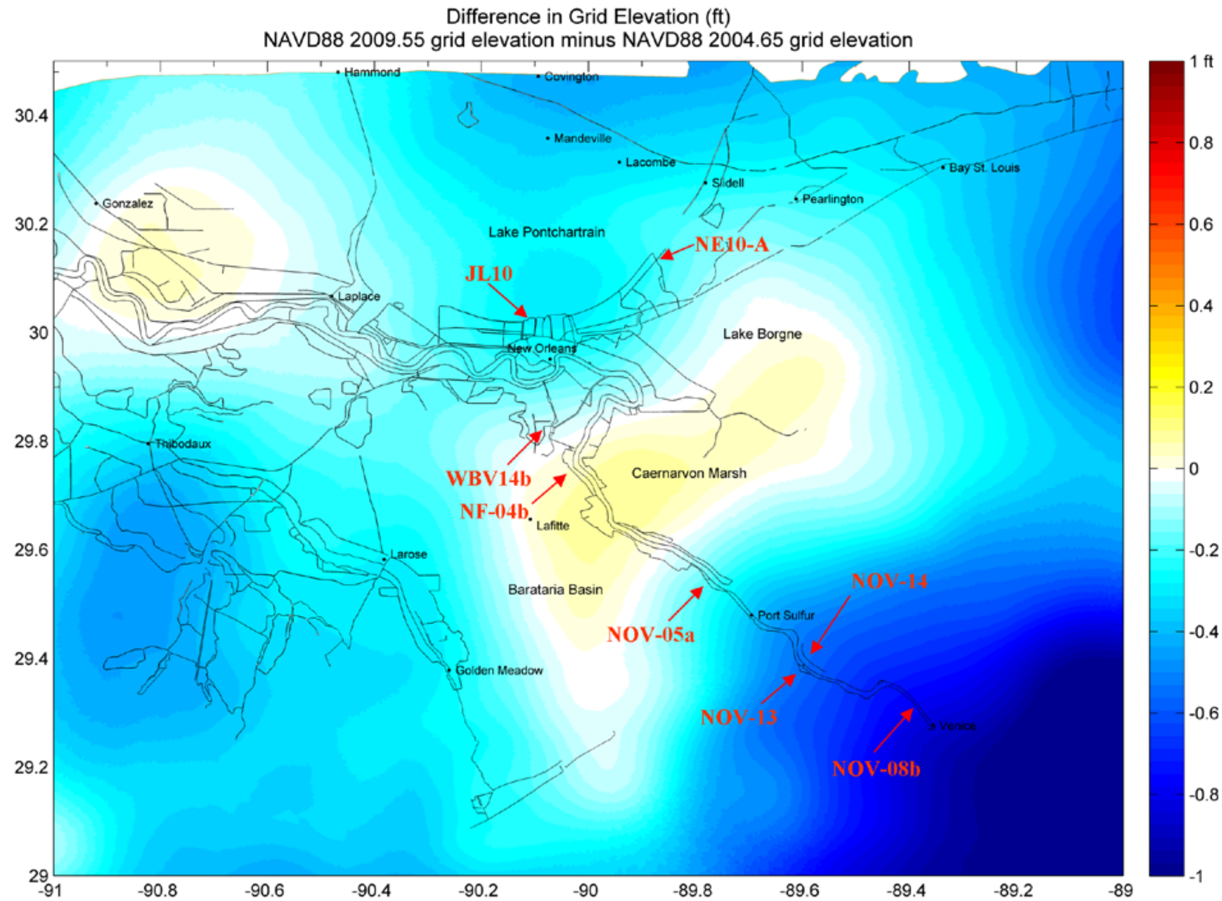

Figure 9 Difference (ft) in elevation between NAVD88 2004.65 and NAVD88 2009.55 ADCIRC grids (the difference is computed by subtracting the NAVD88 2004.65 grid from the NAVD88 2009.55 grid).

3.3 ADCIRC Starting Water Surface Modifications

The starting water surface elevation used in ADCIRC was also adjusted to the NAVD88 2009.55 datum in the updated grid. For the original ADCIRC modeling, a +1.2 ft NAVD88 2004.65 starting water surface elevation was assigned based on analysis of 11 of the 12 coastal water level gages as listed in Table 1 (the Shell Beach gage was excluded because its MSL of 0.99 was considered anomalous). The average local mean sea level above NAVD88 2004.65 at the selected gages was determined to be 0.44 ft. Using a 95% non-exceedance steric water level adjustment of 0.66 ft, the elevation became +1.1 ft NAVD88 2004.65, which was then increased to +1.2 ft NAVD88 2004.65. For this modeling effort, the starting water surface elevation was determined by analyzing the same gages as before, however this time the local mean sea level was computed in NAVD88 2009.55. The 3rd column of Table 1 displays the LMSL above NAVD88 2004.65 at the 11 gages which were used in the original modeling. The 5th column of Table 1 displays the LMSL above NAVD88 2009.55 at the same gages, where available. Three of the gages, Florida Ave, Paris Road, and Seabrook were not assessed because the tidal analysis referenced in the IPET report could not be found. Furthermore, these gages are now inside the LPV Project perimeter and may no longer be relevant for sea level computations. Shell Beach was excluded as in the original analysis, while MV Petrol Dock was excluded because no nearby benchmark exists to establish a tie between tidal and geodetic datums at that location. Using the latest published mean sea level at the remaining 6 gages yielded an average elevation of 0.19 ft above NAVD88 2009.55, a reduction of 0.23 ft as compared to the original analysis. Using the average difference between the 6 available gages, the starting water surface was adjusted to +1.2 - 0.23 = +0.97 ft NAVD88 2009.55. Adjustments for the steric effect, uncertainty, and rounding were kept unchanged, with the only difference in starting water surface elevation compared to the original analysis being as a result of estimating local mean sea level in terms of the updated NAVD88 datum. In this +analysis we evaluate a +0.97 ft NAVD88 2009.55 starting water surface elevation for the 2009.55 simulations. For sensitivity purposes, we also evaluated a +1.2 ft NAVD88 2009.55 starting water surface as a maximum starting water surface elevation and +0.82 ft. NAVD88 2009.55 as a reasonable minimum starting water surface. ADCIRC results using the +0.82 ft and +1.2 ft NAVD88 2009.55 starting water surface elevations are included in the Appendix A and B, respectively.

		IPET Report		2014 Tidal Calculations			
Gage ID	Gage Name	LMSL above NAVD88 (2004.65)	Tidal Datum Epoch	LMSL above NAVD88 (2009.55)	Tidal Datum Epoch	Delta	Notes
8761927	New Canal Station	0.51	2001-2005	0.04	1983-2001	-0.47	
8761402	US Hwy 90, The Rigolets	0.46	2001-2005	0.08	1983-2001	-0.38	
8761799	MV Petrol Dock	0.39	2001-2005	-	2002-2006*	-	Cannot compute. Does not have current benchmark elevation to adjust tidal datums.
8761602	Lake Judge Perez	0.18	2001-2005	0.16	2002-2006*	-0.02	NAVD88 relationship based on NAVD88 (OPUS 2014).
8761724	Grand Isle	0.29	2001-2005	0.35	2002-2006	0.06	
8762372	East Bank 1, Norco, B Labranche	0.58	2001-2005	0.42	2002-2006	-0.16	NAVD88 relationship based on NAVD88 (OPUS 2011).
8761487	Chef Menteur, Chef Menteur Pass	0.34	2001-2005	-	1983-2001	-	Cannot compute. Does not have current benchmark elevation to adjust tidal datums.
8747437	Bay Waveland Yacht Club	0.53	2001-2005	0.10	1983-2001	-0.43	
85800	Shell Beach		2001-2005	-		-	This gage is inactive, and was not considered for the ADCIRC starting water surface.
76120	Florida Ave	0.57	2001-2005	-		-	Cannot compute. Cannot find tidal analysis referenced in IPET report to adjust.
76040	Paris Road	0.35	2001-2005	-		-	
76060	Seabrook	0.67	2001-2005	-		-	
	Average:	**0.44 ft**		**0.19 ft**		**-0.23 ft**	

Table 1 Local mean sea level to NAVD88 2004.65 and NAVD88 2009.55

The 2014 LMSL relationships are referenced to a new realization of NAVD88. Sea level trends cannot be determined by comparing these to the IPET values. During technical review of this document, several reviewers commented on the methods used to adjust the starting water surface for the updated datum. In particular, multiple reviewers questioned using a mixture of 5 year and 19 year tidal epochs for the purpose of computing mean sea level at the tidal gages included in the analysis. MVN's justification for using this mixture was to use as much of the most recent published NOAA tide data as possible, which requires a mixture of tidal epochs because NOAA uses 5 year modified epochs only where necessary, and the standard 19 year National Tidal Datum Epoch elsewhere. However, after consideration of the review comments we received, we concurred that using a methodology that is as consistent as possible with the original ADCIRC modeling should take priority over using updated information, as there are countless components of the design that could have been updated at this time, but we only intended to address/access the impact of the update to the NAVD88 datum.

Following a suggestion of the reviewers, we again re-computed mean sea level in NAVD88 2009.55 using two techniques. The first technique applied the geoid difference (Geoid12A versus Geoid03) at the location of each gage to each mean sea level in the IPET report, which were computed based on data collected from 2001-2005 and were used to determine the starting water surface elevation in the original LPV and WBV Project design. This technique isolates the change in the geoid model and ignores other changes such as subsidence or uncertainty in the NAVD88 epochs. The second technique applies the difference in elevation (NAVD88 epoch 2009.55 versus 2004.65) at the benchmark nearest to each gage (except for NOAA gage 8761799, "MV Petrol Dock," which has no nearby benchmark). This technique applies the best possible estimate for the entire datum update at each location, including the effects of subsidence and uncertainty. As shown in Table 2, both techniques yield the same result: an overall average mean sea level of +0.34 ft NAVD88 2009.55 when all 12 gages are included, or +0.27 ft when the Shell Beach gage is excluded (it was excluded from the original LPV and WBV Project design due to its apparently anomalous mean sea level of +0.99 ft NAVD88 2004.65). A mean sea level of 0.27 ft is 0.17 ft lower than the previously computed value of 0.44, so 0.17 ft was subtracted from the previous water surface elevation of 1.2 ft (all other considerations, including steric adjustment, uncertainty, rounding, etc. were again left unchanged), yielding a starting water surface elevation of +1.03 ft NAVD88 2009.55. This water surface elevation is very close (well within survey error and dwarfed in its effect on modeled surge heights by many other sources of model uncertainty) to the +0.97 ft used in this report. Therefore, the re-computation confirms the appropriateness of +0.97 ft NAVD88 2009.55 as the starting water surface elevation used in the datum update analysis.

LMSL - NAVD88 Relationships							
Gages are from IPET Report, Vol 2, Table 11							
		IPET Report		LMSL Update			
Gage ID	Gage Name	LMSL above NAVD88 (2004.65)	Tidal Datum Epoch	Geoid difference	Elevation difference at closest BM	LMSL above NAVD88 (2009.55) (Geoid difference)	LMSL above NAVD88 (2009.55) (BM difference)
8761927	New Canal Station	0.51	2001-2005	-0.30	-0.3	0.21	0.21
8761402	US Hwy 90, The Rigolets	0.46	2001-2005	-0.14	-0.07	0.32	0.39
8761799	MV Petrol Dock	0.39	2001-2005	0.04		0.43	
8761602	Lake Judge Perez	0.18	2001-2005	-0.03	-0.32	0.15	-0.14
8761724	Grand Isle	0.29	2001-2005	-0.06	0.03	0.23	0.32
8762372	East Bank 1, Norco, B Labranche	0.58	2001-2005	-0.13	-0.1	0.45	0.48
8761487	Chef Menteur, Chef Menteur Pass	0.34	2001-2005	-0.11	-0.14	0.23	0.2
8747437	Bay Waveland Yacht Club	0.53	2001-2005	-0.35	-0.29	0.18	0.24
85800	Shell Beach	0.99	2001-2005	0.04	0.03	1.03	1.02
76120	Florida Ave	0.57	2001-2005	-0.26	-0.17	0.31	0.4
76040	Paris Road	0.35	2001-2005	-0.22	-0.16	0.13	0.19
76060	Seabrook	0.67	2001-2005	-0.30	-0.24	0.37	0.43
NOTES:	(1) The updated LMSL values are referenced to the 2001-2005 update that was referenced in IPET report				Avg of all values	0.34	0.34
	(2) There is no benchmark located near gage 8761799				Avg w/o 85800	0.27	0.27

Table 2 2001-2005 mean sea levels adjusted to NAVD epoch 2009.55

Review comments also mentioned adding more gages to the mean sea level analysis. This was a compelling suggestion, but it was not consistent with the purpose of this analysis, which was to update the previous analysis for the change in datum epoch. Other updates, such as updating the sea level, revising the wind drag equation from the older Garratt coefficient to the more modern formula by Mark Powell, updating wetland land cover maps, using the latest revision to the JPM-OS methodology, etc., should be performed as parts of a comprehensive periodic design update, rather than as piecemeal changes made at various times. Nevertheless, we were interested in the potential impacts of adding additional gages, so we investigated the possibility.

The area around the LPV, WBV and NOV/NFL Project perimeters includes approximately 20 USACE gages and approximately 20 USGS gages that could reasonably be added to this analysis, though not all of these include data from 2001-2005 (indeed, many USACE gages were installed to support the LPV and WBV Projects). There are also approximately 30 NOAA stations around the perimeter beyond those included in the original analysis (all subordinate to either Grand Isle or Bay Waveland), though again only a subset of these would have data collected from 2001-2005. Clearly, the choice of which gages to add to the average would have a significant impact on the sea level computed. We identified three areas that seem to be missing from the original set of gages: near the WBV perimeter, Breton Sound, and on/along the Mississippi River south of Lake Judge Perez. Adding gages in each of these areas would introduce its own challenges related to data availability, benchmark accuracy, and removal of the effects of river discharge, but it could probably be done. Before doing so, however, we did a sensitivity analysis to determine the possible benefit of the exercise. We added four hypothetical gages to the analysis, each with a mean sea level of 0.0 ft (an unreasonably low number to find for all four gages). We then changed the mean sea level for these four gages to 1.0 ft (unreasonably high). Neither of these additions changed the overall mean enough to move it outside the range of +0.82 to +1.2 ft NAVD88 2009.55 that were analyzed in this report. Therefore, we concluded that there would be no benefit to subjectively adding gages to the dataset for the purposes of re-computing 2001-2005 mean sea level at this time, though this question should certainly be considered as a part of the next systematic reanalysis.

The reanalysis to confirm the appropriateness of +0.97 ft NAVD88 2004.65 as the starting water surface elevation and the analysis that resulted in the decision to not include additional gages was re-reviewed by the technical reviews. The reviewers concurred with MVN's reanalysis and results.

3.4 Limitations and Assumptions

The most significant assumption in this analysis is the selected starting water surface elevation for the NAVD88 2009.55 datum. There is a relatively high amount of variation in the local mean sea level above NAVD88 at the selected gages. To evaluate the sensitivity of the starting water surface elevation parameter, we have evaluated three possible starting water surface elevations for the 2009.55 grid: +0.82 ft NAVD88 2009.55, +0.97 ft NAVD88 2009.55 and +1.2 ft NAVD88 2009.55. The best estimate of the starting water surface elevation is +0.97 ft NAVD88 2009.55.

Results of re-running ADCIRC using a starting water surface elevation of +0.82 ft NAVD88 2009.55) are provided in Appendix A. The +0.82 ft NAVD88 2009.55 is a reasonable estimate of the starting water surface elevation based on the mean adjustment of the geoid for the entire LA coast. The average adjustment between the 2009.55 and 2004.65 datums is -0.38 ft. Using this adjustment, the original +1.2 ft NAVD 2004.65 starting water surface elevations becomes +0.82 ft NAVD88 2009.55.

Results of re-running ADCIRC using a starting water surface elevation of +1.2 ft NAVD88 2009.55 are provided in Appendix A. This provides the variability of ADCIRC results when only lowering the topography/bathymetry while leaving the starting water surface elevation unchanged (+1.2 ft NAVD88 2009.55). This is a reasonable sensitivity check, but is overly conservative. We have attempted to capture some of the uncertainty in the variation in LMSL above NAVD88 by modeling two possible starting water surface elevations for the NAVD88 2009.55 simulations. In the original ADCIRC modeling, the +1.2 ft NAVD88 2004.65 starting water surface elevation is conservative because it uses a 95% non exceedance steric water level adjustment and it was increased arbitrarily by +0.1 ft. It should also be noted that this set of runs only included adjusting the bathymetry/topography for the State of Louisiana while the +0.82 ft and +0.97 ft runs used adjusted grids for the entire Gulf of Mexico.

The effect of the datum change on wave heights and wave periods was not evaluated. Waves were not analyzed because it was assumed the impacts would be negligible. In areas where wave are depth limited, the wave heights could possibly increase (or decrease) by 60% of the surge difference. For example, if the surge increases by +0.1 ft relative to the land, the wave heights may increase by approximately +0.06 ft.

The Mississippi and Atchafalaya Rivers were assumed to follow the 70/30 split in river discharge. The Mississippi river flux boundary condition was set to a flow of 400,000 cfs. No attempts were made to re-calibrate the model.

Simulation of a subset of 18 storms is also a limitation of the approach. However, a decent fit is observed in the regression analysis which shows that the datum change is a small perturbation with very predictable impacts to stage-frequency. The regression analysis will be presented in the next section of this report.

4. Results

4.1 ADCIRC Simulations NAVD88 2004.65 and NAVD88 2009.55

Peak surge results from the 18 synthetic storms are processed to estimate the impacts to stage-frequency and associated design elevations. Figure 10 displays an example of the maximum surge elevation produced by synthetic storm 027 using the NAVD88 2004.65 grid. Figure 11 displays the maximum surge elevation produced for synthetic storm 027 using the NAVD88 2009.55 grid with a starting water surface elevation of +0.97 ft NAVD88 2009.55. A difference between the two peak surge surfaces is plotted in Figure 12. The difference plot reveals that for this particular storm, the peak surge is changed by approximately -0.2 to -0.3 ft. The global reduction in surge elevation is similar to the reduction in starting water surface elevation (which was adjusted to account for the conversion between LMSL and NAVD88 2009.55).

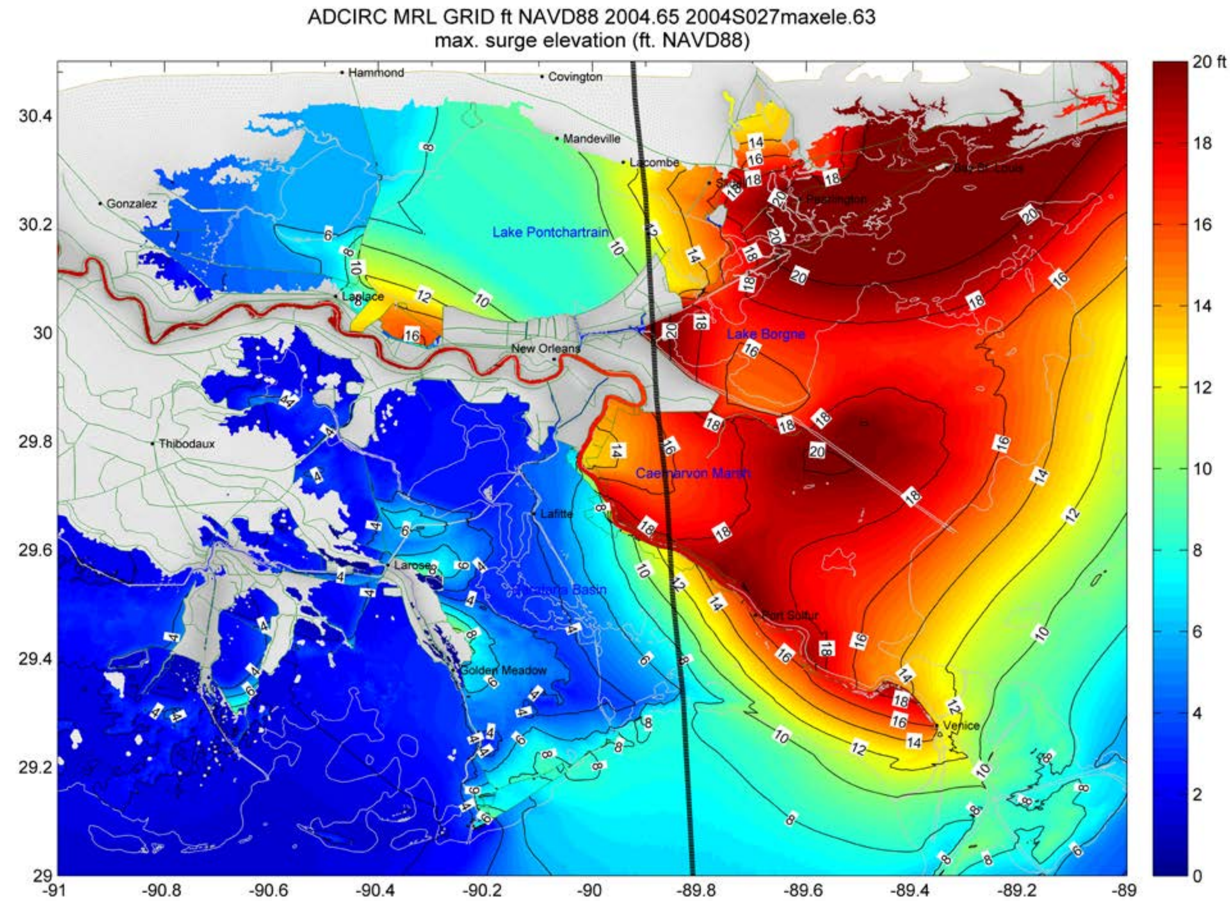

Figure 10 Maximum surge elevation (ft NAVD88 2004.65) for synthetic storm 027 in ft NAVD88 2004.65

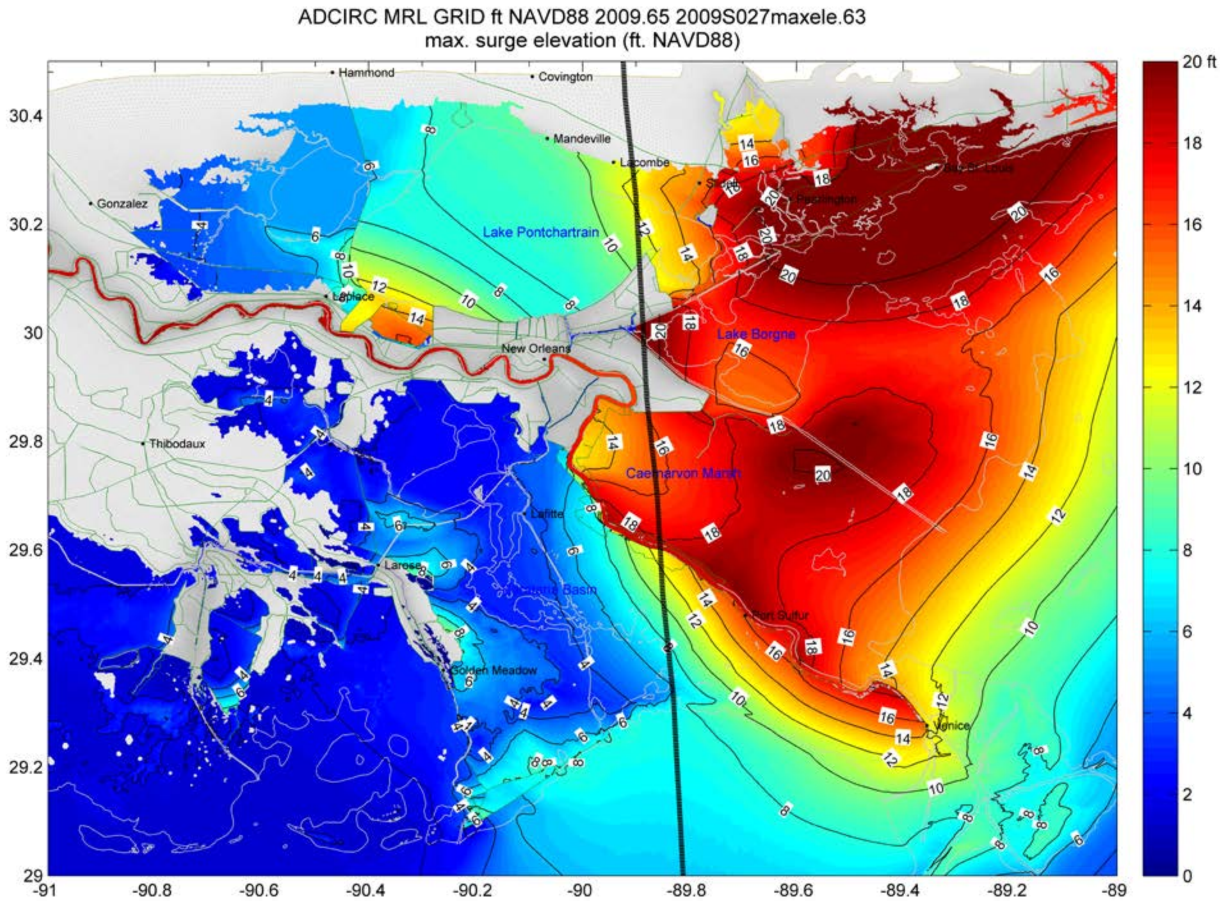

Figure 11 Maximum surge elevation (ft NAVD88 2009.55) for synthetic storm 027 in ft NAVD88 2009.55 with a starting water surface elevation of +0.97 ft NAVD88 2009.55.

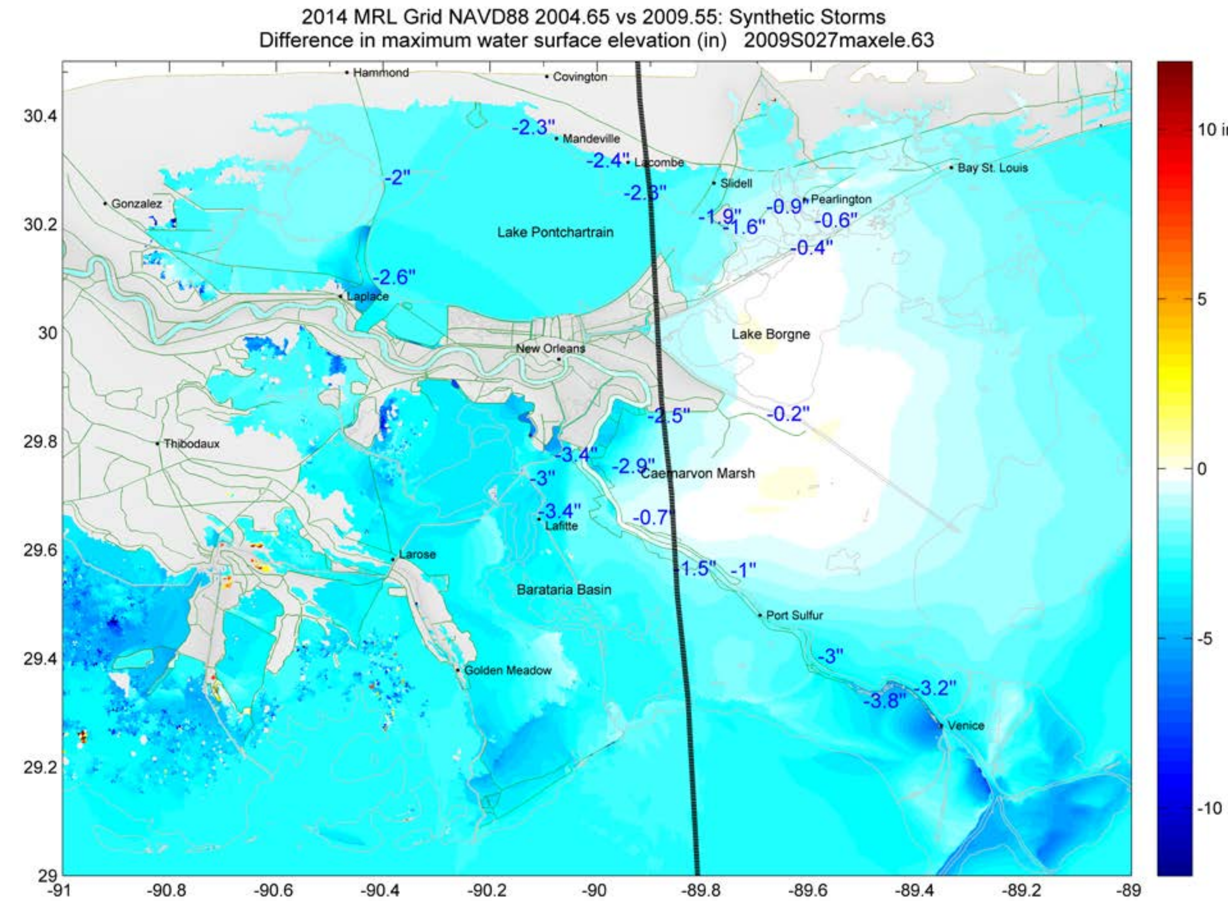

Figure 12 Difference (inches) in peak surge between the NAVD88 2004.65 and the NAVD88 2009.55 grids (the difference is computed by subtracting the NAVD88 2004.65 maximum surge elevation from the NAVD88 2009.55 maximum surge elevation for synthetic storm 027)

4.2 Impacts to Stage Frequency and LPV, WBV and NOV/NFL HSDRRS Designs

Simulation of 18 hypothetical storms shows that the surge elevation values on the mesh based on the 2009.55 epoch tend to lower relative to surge elevation values calculated on the 2004.65 epoch mesh along with the generally downward shift between the 2004.65 and the 2009.55 epochs. For example, the 1% (100-year) surge elevation produced by the NAVD88 2004.65 grid at the LPV Lakefront location is +7.6 ft NAVD88 2004.65. At the same location on the NAVD88 2009.55 grid, the 1% (100-year) surge is estimated to be +7.4 ft NAVD88 2009.55 through evaluation of an 18 storm subset. In this example, while the surge elevation lowers, the surge height above the geoid or mean water elevation is nearly the same; the difference between the datums themselves is 0.3 ft. With the datum adjustment considered in this example, the design elevations based on the surge elevations computed on the NAVD88 2009.55 grid are nearly equivalent to the design elevations based on surge elevations computed on the NAVD88 2004.65 grid. In general, the simulation of 18 storms on the NAVD88 2009.55 grid compute surge elevations that are lower in direct relationship to the lower starting water surface elevation. Specifically, surge elevations relative to NAVD88 2009.55 are approximately -0.2 to -0.4 ft different (lower) than surge elevations relative to NAVD88 2004.65 coastwide. If the datum difference at a specific project location is within this range, as is the case for the LPV, WBV and NFL portion of NOV projects, adding height to published design elevations is not needed to achieve the intended LORR. If the datum difference at a specific projects location is greater than this range, as is the case for some of the NOV project, there could be a requirement to add height to the levee in order to achieve the intended LORR.

The impact of the datum shift has been determined at a sample of the 1% (100-year) LPV and WBV and the 2% (50-year) NOV/NFL levee and structure segments. The datum shift has negligible impact on the three sample design segments of 1% (100-year) LPV and WBV. The datum shift for LPV and WBV range from +0.1 to - 0.3 ft, which is similar to the shift in starting water surface elevation. Since the datum difference at this location is similar to the shift in starting water surface elevation, there is no requirement to add height to the published design elevations in order to achieve the intended LORR. In short, the effects of the datum appear to be a wash for LPV and WBV.

For NOV/NFL, the datum shift varies from -1.05 to +0.16 ft. For some NOV/NFL reaches, especially areas in the lower half of the birds foot delta (NOV features), the datum shift has a greater shift downward than the shift in starting water surface elevation. In areas where the datum shift has a greater magnitude than the shift in starting water surface elevation, such as NOV-08b, NOV-13, and NOV-14, there would be a requirement to raise the levees and structures to achieve the intended LORR. For most of the NOV structural features the NOV Project Delivery Team (PDT) has already proceeded with setting the required construction elevation in the 2009.55 epoch as the same value as determined originally in the 2004.65 epoch (i.e. adding height equal to negative datum shift). For example, if the published elevation is 17.0 ft NAVD88 2004.65, the new construction elevation would be set at 17.0 ft NAVD88 2009.55. If the datum shift in this area is -1.0 ft, this approach requires the structures to be re-designed with an additional 1.0 ft of height. At other segments of NOV and all of NFL, the datum shift is positive, and there is no requirement to add height to the levees or structures to achieve the intended LORR.

The peak surge results for the NAVD88 2004.65 and NAVD88 2009.55 grids are post-processed using a regression analysis to determine impacts to the 2% (50-year), 1% (100-year) and 0.5% (500-year) surges and associated designs at a few locations within the LPV, WBV and NOV/NFL projects. Figure 13 displays the peak surge regression plot at a location near Venice, LA. The right portion of this plot compares peak surge from the NAVD88 2004.65 grid against the peak surge from the NAVD88 2009.55 grid. The peak surges are plotted as red dots. A second order polynomial trendline is plotted through the data. JPM-OS surge statistics developed on the SL15v7 NAVD88 2004.65 grid are plugged into the trendline equation to estimate surge statistics in the NAVD88 2009.55 datum. Although the regression analysis uses only a subset of storms, a reasonable trend is developed, giving some confidence in the NAVD88 2009.55 surge-frequency estimates. The same regression analysis is performed at other locations of interest. Figure 14 displays the regression trendline near St. Jude, about halfway up the birdsfoot and Figure 15 displays the regression trendline near Ollie Pump Station. These 3 examples display the NAVD88 2009.55 results using the 0.97 ft starting water surface elevation. A summary table which compares the output of the regression analysis at multiple locations using the 0.97 ft starting water surface elevation is provided in Table 3. The overall summary of the regression analysis shows that statistical surge values (2% (50-year), 1% (100-year) and 0.5% (500-year)) lower in the NAVD88 2009.55 datum. The lowering is primarily caused by the adjustment to the starting water surface elevation. Using a starting water surface elevation of +0.97 ft NAVD88 2009.55 changes the surge-frequency curves by approximately -0.2 to -0.4 ft at most locations. Although the surge values appear to decrease, the datum shift must be considered in order to have a proper comparison. If the datum difference at a specific project location is within this range, there is no requirement to add height to published design elevations in order to achieve the intended LORR.

In order to illustrate this effect, a few design examples have been selected for evaluation. In these tests, we first compute the design elevations to the nearest tenth of a foot using the original NAVD88 2004.65 surge elevation. Computations of levee elevations in this document used the same methodologies described in the design guidelines and the October 2007 Design Elevation Report (DER), as well as the same standard deviations, wave heights and wave periods for the selected levee reaches. Table 4 displays the surge elevation and design elevation in the NAVD88 2004.65 datum highlighted in blue. Next the required design elevation is computed in the NAVD88 2009.55 datum using a +0.97 ft starting water surface elevation. The green columns show the surge elevation and the required design elevation in NAVD88 2009.55. Using the nearest NGS benchmark, the conversion between the NAVD88 2004.65 and NAVD88 2009.55 datums is assigned to each levee design reach. This conversion is applied to the NAVD88 2009.55 required elevation to determine an equivalent elevation in the NAVD88 2004.65 datum. Once this conversion is applied, the true difference in design height is determined. The actual difference is listed in the last column of Table 4. A negative value indicates a lowering in levee height and a positive value indicates an increase in levee height.

In many locations, including all of the 1% (100-year) LPV and WBV levee design reaches and the NFL portion of NOV, the required design heights may be slightly lower with the NAVD88 2009.55 datum update. In these locations, the analysis shows that the original designs built to NAVD88 2004.65 will provide the intended LORR.

The locations where the published design elevation actually increase would be lower Plaquemines Parish (NOV features), where the datum changes by up to -1.05 ft, while the water elevation was changed based on new analysis by -0.2 to -0.4 ft. For example, near Venice (Duvic Pump Station, NOV-08b), the published 2% design elevation (2063) is +17.0 NAVD88 2004.65. The 2009.55 surge analysis at this location shows that the design becomes +16.8 ft NAVD88 2009.55 (using the +0.97 ft starting water surface elevation). Near Venice, the conversion between the NAVD88 2009.65 and NAVD88

2009.55 datum is -1.05 ft. With this conversion applied, the design requirement in the new datum could be 0.85 ft higher.

Table 4 is provided for sensitivity analysis purposes only. Published design elevations are provided to the nearest 0.0 or 0.5 ft (rounding up). Published design elevations for structures consider future conditions (2057 for LPV and WBV, 2063 for NOV/NFL) accounting for subsidence and relative sea level rise. The values in these tables should not be directly compared to the published design elevations for current or future conditions. The conversion (NAVD88 2004.65 to NAVD88 2009.55) noted in Table 4 is taken from the ADCIRC grid adjustments made to the topography/bathymetry.

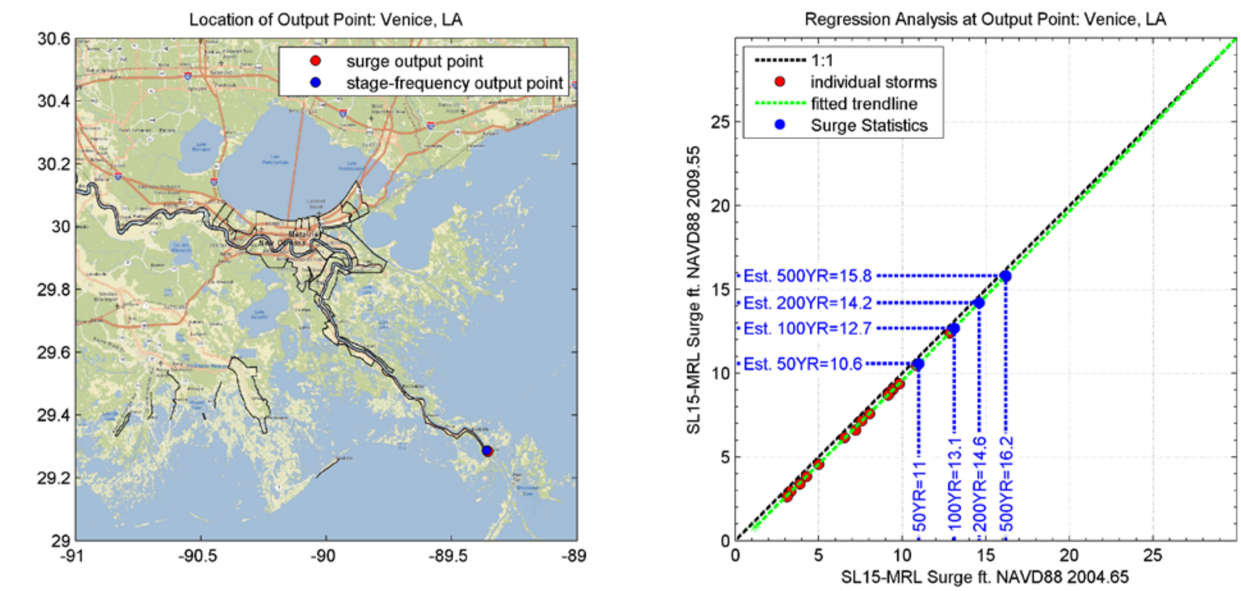

Figure 13 Regression analysis comparing peak surge in NAVD88 2004.65 and NAVD88 2009.55 at a location near Venice, LA (MRL side), using +0.97 ft NAVD88 2009.55 starting water surface elevation

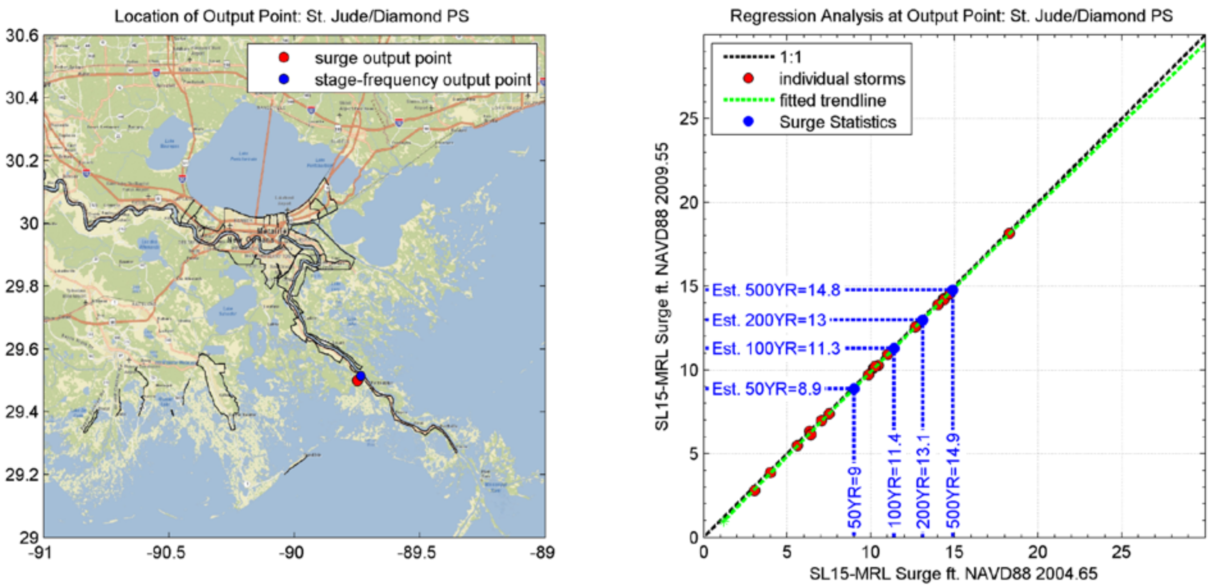

Figure 14 Regression analysis comparing peak surge in NAVD88 2004.65 and NAVD88 2009.55 at a location near NOV-05a Diamond Levee (backlevee side), using +0.97 ft starting water surface elevation

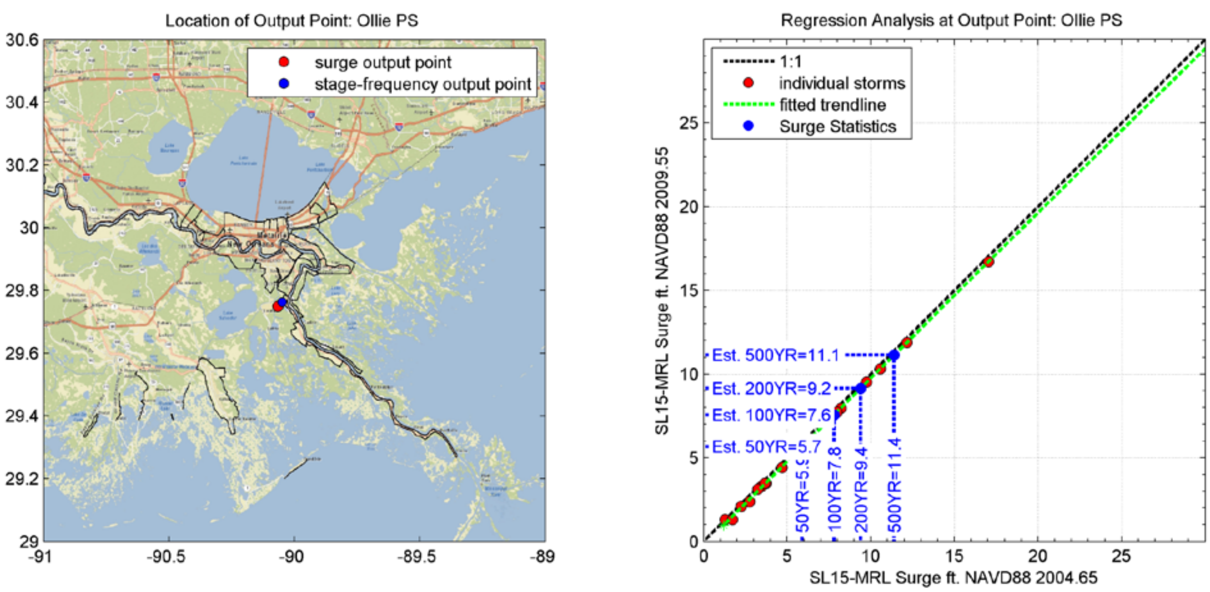

Figure 15 Regression analysis comparing peak surge in NAVD88 2004.65 and NAVD88 2009.55 at a location near NF-04b Ollie Pump Station (NFL backlevee), using +0.97 ft NAVD88 2009.55 starting water surface elevation

	Surge Elevation (ft. NAVD88 2004.65) using a starting water surface elevation of 1.2 ft. NAVD88 2004.65			Updated Surge Elevation (ft. NAVD88 2009.55) using a starting water surface elevation of 0.97 ft NAVD88 2009.55			Difference (ft.)		
	50 YR	100 YR	500YR	50 YR	100 YR	500YR	50 YR	100 YR	500YR
NF-04b Ollie PS	5.9	7.8	11.4	5.7	7.6	11.1	-0.2	-0.2	-0.3
NF-05b Wilkinson Canal PS	7.2	9.2	13.3	7.1	9.1	13.2	-0.1	-0.1	-0.1
NOV-05a Diamond Levee	9.0	11.4	14.9	8.9	11.3	14.8	-0.1	-0.1	-0.1
NOV-09 (~RM46, St. Jude)	14.1	16.5	19.4	13.9	16.4	19.3	-0.2	-0.1	-0.1
NOV-13 Empire Floodgate	8.1	10.3	14.1	8.0	10.2	13.9	-0.1	-0.1	-0.2
NOV-14 Empire Lock (~RM 30)	13.5	16.1	19.5	13.3	15.9	19.3	-0.2	-0.2	-0.2
NOV-08b Duvic PS	8.2	10.4	14.3	8.1	10.2	14.0	-0.1	-0.2	-0.3
NOV-12 (~RM 15)	11.0	13.1	16.2	10.7	12.8	16.0	-0.3	-0.3	-0.2
NOV-02 (Bellevue PS)	13.7	16.1	19.8	13.7	16.1	19.7	0.0	0.0	-0.1
WBV-77 WTI (Davis Pond Div)	5.4	6.8	9.2	5.1	6.5	8.8	-0.3	-0.3	-0.4
WBV-90 WCC	4.8	6.8	10.1	4.5	6.5	9.8	-0.3	-0.3	-0.3
WBV-MRL 4.2 (~RM 78)	14.4	16.3	19.0	14.2	16.2	18.9	-0.2	-0.1	-0.1
LPV-149 (Caernarvon)	14.4	17.5	21.6	14.3	17.4	21.5	-0.1	-0.1	-0.1
IHNC-02 Surge Barrier	14.6	17.5	21.5	14.4	17.4	21.4	-0.2	-0.1	-0.1
LPV 109.02a (NE-10-A)	7.9	9.4	12.1	7.7	9.2	11.9	-0.2	-0.2	-0.2
LPV Lakefront, Orleans Parish, UNO	7.6	8.9	11.6	7.4	8.7	11.4	-0.2	-0.2	-0.2
LPV-07b2 St. Charles	10.2	12.0	14.7	10.0	11.8	14.5	-0.2	-0.2	-0.2
WSLP (I-10 near I-55)	6.9	9.0	12.3	6.6	8.7	12.0	-0.3	-0.3	-0.3
WSLP (I-10 west of Hwy 3188)	3.2	3.7	4.4	3.0	3.5	4.2	-0.2	-0.2	-0.2

Table 3 Summary of regression analysis results showing difference between the NAVD88 2004.65 surge-frequency and NAVD88 2009.55 surge-frequency (+0.97 ft NAVD88 2009.55 starting water surface elevation)

		With Starting Water Surface Elevation = 1.2 ft. NAVD88 2004.65		With Starting Water Surface Elevation = 0.97 ft. NAVD88 2009.55		Conversion NAVD88 2004.65 to NAVD88 2009.55	Actual Difference in Design Height
Design Segment	Authorized LORR	surge (ft NAVD88 2004.65)	design (ft NAVD88 2004.65)	surge (ft NAVD88 2009.55)	design (ft NAVD88 2009.55)	(ft)	(ft)
NF-04b Ollie PS	50YR	8.00	11.30	7.80	11.10	0.16	-0.36
NOV-05a Diamond Levee	50YR	9.10	12.70	9.00	12.60	0.16	-0.26
NOV-08b Duvic PS	50YR	11.30	17.00	11.20	16.80	-1.05	0.85
NOV-13 Empire Floodgate	50YR	11.20	16.40	11.10	16.30	-0.63	0.53
NOV-14 Empire Lock (RM 30)	50YR	13.10	19.50	12.90	19.20	-0.63	0.33
LPV 109.02a (NE10-A)	100YR	9.50	16.80	9.30	16.60	-0.20	0.00
LPV-JL10 Jefferson Lakefront	100YR	8.70	13.50	8.50	13.30	-0.30	0.10
WBV14b	100YR	7.30	9.90	7.00	9.60	-0.043	-0.26

Table 4 Summary of design elevation differences using a +0.97 ft starting water surface elevation in the NAVD88 2009.55 simulations

5. Conclusions and Recommendations

In summary, the sensitivity analysis shows that there are very modest impacts to the overall design elevations when the NAVD88 2009.55 epoch adjustment is considered, with the exception of lower Plaquemines Parish. For the LPV, WBV and NFL portions of NOV, the analysis shows that the published design elevations are adequate to achieve the intended LORR with the geoid update considered. At the LPV, WBV and NFL portions of NOV locations, the datum shift is similar to the shift in starting water surface elevation. Since the datum difference at this location is similar to the shift in starting water surface elevation, there is no requirement to add height to the published design elevations in order to achieve the intended LORR.

Since this analysis only updated surge modeling results from a subset of storms using this single changed parameter (and was not a comprehensive update considering all changed parameters), MVN H&H Branch does not endorse revising the published design elevations for the WBV, LPV or NOV/NFL projects from their published values. For LPV, WBV and the NFL portion of NOV, the published design elevations achieve the intended LORR. For some NOV Project features; however, this is not the case.

For lower Plaquemines (some of NOV) and specifically those locations where the datum difference is greatest (up to -1.05 ft), the analysis shows that published elevations may need to increase by up to 1 ft in locations because the datum shift is more than the shift in starting water surface elevation. The assumptions being used by the NOV PDT, to add height to most NOV structure designs equal to the negative datum difference at that specific location, are conservative and will result in structural features that will be built to their intended (or in some cases, slightly higher than their intended) LORR. Since levees are more easily adaptable in the future, the NOV PDT decided to construct the levees to the same height (from levee toe to crown) as planned under the previous epoch. This means that for levees, the PDT is not requiring the levees be raised by the negative difference between the datums. These project-specific adaptations are being memorialized in project-specific memorandums for record. MVN H&H Branch strongly recommends adding height to the structures (at an amount equal to the negative change in datum) to ensure that the 2063 intended LORR is achieved.

For NOV/NFL Projects, a table will be added to the 2014 update to the DER to convert the published design elevations for design and construction contract reaches from NAVD88 2004.65 to NAVD88 2009.55. This is being done for constructability purposes, as outlined in Addendum to Revised Vertical Control Requirements for USACE Projects (Engineering Division Datum Policy Memo #2), dated 15 May 2014. This memo set the project datum/epoch for the NOV/NFL project to be NAVD88 2009.55.

When assessing the findings of this analysis, it is most important to consider sources of error and other assumptions in the overall JPM-OS surge hazard analysis conducted for design of LPV, WBV and NOV/NFL. There are many sources of uncertainty which are most likely much larger than the changes to design elevation estimated in this analysis. For example, the ADCIRC model does not have the ability to reliably predict surge accurately to within a few tenths of a foot. Hindcasts of Katrina, Rita, Gustav, Ike and

Isaac reveal that ADCIRC usually predicts surge to within +/- 1.5 ft. There are many sources of error in the ADCIRC modeling including: errors in the applied wind-field and wind-stress equations, errors in bathymetric elevations/datums, errors in the nodal attributes (bottom friction, wind canopy coefficients, etc), errors due to lack of resolution, and many more limitations.

In the overall H&H levee design approach, conservancy in estimates such as overbuild, subsidence rates, sea level rise rates, surveying accuracy, steric water level adjustment, local bathymetric changes, shoreline erosion, vegetation changes and other issues are just as criticial, if not more critical, than the new geoid update. Other issues concerning the surge hazard analysis and design procedures could potentially result in changes to required design elevation that are greater than the changes calculated in this analysis. Given the large amount of uncertainty in any surge hazard analysis, it is important to re-analyze the complete system and conduct a new analysis for coastal Louisiana using the latest data, the latest technology and the most sound methodology on a recurring interval of 10 years or after a major storm event, as expressed in the district's Engineering Division Datum Policy Memo #2.

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Appendix A - NAVD88 2009.55 Results with Starting Water Surface Elevation of +0.82 ft NAVD88 2009.55.

A summary table which compares the output of the regression analysis using the 0.82 ft starting water surface elevation is provided in Table 5. Using the lower starting water surface elevation of +0.82 ft NAVD88 2009.55 lowers the surge-frequency curve values by approximately 0.3 to 0.4 ft at most locations. Table 6 displays the same design elevation calculations described previously, but with the +0.82 ft NAVD88 2009.55 starting water surface elevation.

	Surge Elevation (ft. NAVD88 2004.65) using a starting water surface elevation of 1.2 ft. NAVD88 2004.65			Updated Surge Elevation (ft. NAVD88 2009.55) using a starting water surface elevation of 0.82 ft NAVD88 2009.55			Difference (ft.)		
	50 YR	100 YR	500YR	50 YR	100 YR	500YR	50 YR	100 YR	500YR
NF-04b Ollie PS	5.9	7.8	11.4	5.5	7.4	10.9	-0.4	-0.4	-0.5
NF-05b Wilkinson Canal PS	7.2	9.2	13.3	7.0	9.0	13.0	-0.2	-0.2	-0.3
NOV-05a Diamond Levee	9.0	11.4	14.9	8.7	11.1	14.6	-0.3	-0.3	-0.3
NOV-09 (~RM46, St. Jude)	14.1	16.5	19.4	13.8	16.2	19.2	-0.3	-0.3	-0.2
NOV-13 Empire Floodgate	8.1	10.3	14.1	7.8	10.0	13.8	-0.3	-0.3	-0.3
NOV-14 Empire Lock (~RM 30)	13.5	16.1	19.5	13.1	15.7	19.2	-0.4	-0.4	-0.3
NOV-08b Duvic PS	8.2	10.4	14.3	7.9	10.1	13.9	-0.3	-0.3	-0.4
NOV-12 (~RM 15)	11.0	13.1	16.2	10.6	12.7	15.8	-0.4	-0.4	-0.4
NOV-02 (Bellevue PS)	13.7	16.1	19.8	13.4	15.9	19.5	-0.3	-0.2	-0.3
WBV-77 WTI (Davis Pond Div)	5.4	6.8	9.2	5.0	6.3	8.6	-0.4	-0.5	-0.6
WBV-90 WCC	4.8	6.8	10.1	4.4	6.4	9.7	-0.4	-0.5	-0.4
WBV-MRL 4.2 (~RM 78)	14.4	16.3	19.0	14.1	16.0	18.7	-0.3	-0.3	-0.3
LPV-149 (Caernarvon)	14.4	17.5	21.6	13.9	17.1	21.3	-0.5	-0.4	-0.3
IHNC-02 Surge Barrier	14.6	17.5	21.5	14.3	17.2	21.2	-0.3	-0.3	-0.3
LPV 109.02a (NE-10-A)	7.9	9.4	12.1	7.5	9.0	11.7	-0.4	-0.4	-0.4
LPV Lakefront, Orleans Parish, UNO	7.6	8.9	11.6	7.2	8.5	11.2	-0.4	-0.4	-0.4
LPV-07b2 St. Charles	10.2	12.0	14.7	9.8	11.6	14.4	-0.4	-0.4	-0.3
WSLP (I-10 near I-55)	6.9	9.0	12.3	6.3	8.5	11.9	-0.6	-0.5	-0.4
WSLP (I-10 west of Hwy 3188)	3.2	3.7	4.4	2.9	3.4	4.1	-0.3	-0.3	-0.3

Table 5 Summary of regression analysis results showing difference between the NAVD88 2004.65 surge-frequency and NAVD88 2009.55 surge-frequency (+0.82 ft NAVD88 2009.55 starting water surface elevation)

		With Starting Water Surface Elevation = 1.2 ft. NAVD88 2004.65		With Starting Water Surface Elevation = 0.82 ft. NAVD88 2009.55		Conversion NAVD88 2004.65 to NAVD88 2009.55	Actual Difference in Design Height
Design Segment	Authorized LORR	surge (ft NAVD88 2004.65)	design (ft NAVD88 2004.65)	surge (ft NAVD88 2009.55)	design (ft NAVD88 2009.55)	(ft)	(ft)
NF-04b Ollie PS	50YR	8.00	11.30	7.60	11.00	0.16	-0.46
NOV-05a Diamond Levee	50YR	9.10	12.70	8.90	12.50	0.16	-0.36
NOV-08b Duvic PS	50YR	11.30	17.00	11.00	16.70	-1.05	0.75
NOV-13 Empire Floodgate	50YR	11.20	16.40	10.90	16.10	-0.63	0.33
NOV-14 Empire Lock (RM 30)	50YR	13.10	19.50	12.70	19.00	-0.63	0.13
LPV 109.02a (NE10-A)	100YR	9.50	16.80	9.10	16.40	-0.20	-0.20
LPV-JL10 Jefferson Lakefront	100YR	8.70	13.50	8.30	13.10	-0.30	-0.10
WBV14b	100YR	7.30	9.90	6.90	9.50	-0.043	-0.36

Table 6 Summary of design elevation differences using a +0.82 ft starting water surface elevation in the NAVD88 2009.55 simulations

Appendix B - NAVD88 2009.55 Results with Starting Water Surface Elevation of +1.2 ft NAVD88 2009.55.

Simulations were conducted with a starting water surface elevation of +1.2 ft NAVD88 2009.55 (assuming there was no change in the starting water surface elevation value, i.e. +1.2 ft NAVD88 2004.65 = +1.2 ft NAVD88 2009.55). In these initial simulations, the SL 15v7 grid was approximately adjusted to the NAVD88 2009.55 datum for the state of Louisiana only. For all subsequent simulations, the grid was approximately adjusted for the entire Gulf of Mexico. Through recent analysis of coastal water level gages, the conversion between the NAVD88 2004.65 and NAVD88 2009.55 starting water surface elevations of -0.23 ft, resulting in a starting water surface elevation of +0.97 ft NAVD88 2009.55 was found to be the best approximation of starting water surface that is consistent with the original analysis. A +1.2 ft NAVD88 2009.55 water surface elevation on the NAVD88 2009.55 grid is not a realistic estimate and is overly conservative. It is useful for information purposes, as it demonstrates the sensitivity of the ADCIRC surge modeling results to changes in topography/bathymetry only, with starting water surface left unchanged. The following table compares surge-frequency data computed using the NAVD88 2004.65 grid with a starting water surface of +1.2 ft NAVD88 2004.65 to surge frequency data computed using the NAVD88 2009.55 grid with a starting water surface elevation of +1.2 ft NAVD88 2009.55.

	Surge Elevation (ft. NAVD88 2004.65) using a starting water surface elevation of 1.2 ft. NAVD88 2004.65			Updated Surge Elevation (ft. NAVD88 2009.55) using a starting water surface elevation of 1.2 ft NAVD88 2009.55			Difference (ft.)		
	50 YR	100 YR	500YR	50 YR	100 YR	500YR	50 YR	100 YR	500YR
NF-04b Ollie PS	5.9	7.8	11.4	5.9	7.8	11.4	0.0	0.0	0.0
NF-05b Wilkinson Canal PS	7.2	9.2	13.3	7.3	9.4	13.4	0.1	0.2	0.1
NOV-05a Diamond Levee	9.0	11.4	14.9	9.1	11.5	15.0	0.1	0.1	0.1
NOV-09 (~RM46, St. Jude)	14.1	16.5	19.4	14.0	16.5	19.5	-0.1	0.0	0.1
NOV-13 Empire Floodgate	8.1	10.3	14.1	8.2	10.4	14.1	0.1	0.1	0.0
NOV-14 Empire Lock (~RM 30)	13.5	16.1	19.5	13.4	16.0	19.4	-0.1	-0.1	-0.1
NOV-08b Duvic PS	8.2	10.4	14.3	8.2	10.4	14.1	0.0	0.0	-0.2
NOV-12 (~RM 15)	11.0	13.1	16.2	10.9	13.0	16.2	-0.1	-0.1	0.0
NOV-02 (Bellevue PS)	13.7	16.1	19.8	14.1	16.4	19.9	0.4	0.3	0.1
WBV-77 WTI (Davis Pond Div)	5.4	6.8	9.2	5.4	6.8	9.2	0.0	0.0	0.0
WBV-90 WCC	4.8	6.8	10.1	4.8	6.8	10.1	0.0	0.0	0.0
WBV-MRL 4.2 (~RM 78)	14.4	16.3	19.0	14.4	16.3	19.2	0.0	0.0	0.2
LPV-149 (Caernarvon)	14.4	17.5	21.6	14.9	17.9	21.9	0.5	0.4	0.3
IHNC-02 Surge Barrier	14.6	17.5	21.5	14.7	17.6	21.6	0.1	0.1	0.1
LPV 109.02a (NE-10-A)	7.9	9.4	12.1	8.0	9.5	12.2	0.1	0.1	0.1
LPV Lakefront, Orleans Parish, UNO	7.6	8.9	11.6	7.7	9.0	11.8	0.1	0.1	0.2
LPV-07b2 St. Charles	10.2	12.0	14.7	10.2	11.9	14.6	0.0	-0.1	-0.1
WSLP (I-10 near I-55)	6.9	9.0	12.3	6.8	8.9	12.2	-0.1	-0.1	-0.1
WSLP (I-10 west of Hwy 3188)	3.2	3.7	4.4	3.2	3.7	4.5	0.0	0.0	0.1

Table 7 Summary of regression analysis results showing difference between the NAVD88 2004.65 surge-frequency and NAVD88 2009.55 surge-frequency (+1.2 ft NAVD88 2009.55 starting water surface elevation)

Appendix S

LPV-149 HSDRRS Floodwall tie-in at the MRL

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The following memo concerns the LPV-149 HSDRRS floodwall tie-in at the MRL. The floodwall forms a connection between the St. Bernard floodwall and the East Bank MRL levee at approximately river mile 82. Figure 1 displays an aerial photo of the tie-in. The floodwall steps down rapidly from 26.0 ft to 19.96 ft NAVD88 2004.65 at the tie-in. Concern was expressed over the design of this floodwall by the SLFPA-East:

“We are also puzzled by the geometric configuration used to tie the LPV 149 HSDRRS Floodwall into the MRL in Caernarvon. The height of the floodwall is stepped down in the last few monoliths to meet the existing elevation of the MRL. Has anyone calculated the 2057 overtopping rates for this segment of the floodwall? If so, will the overtopping cause problems with flooding in the area bounded by the new floodwall and the old LPV levee? The “transition” armor at this location appears to be severely inadequate.

A much better tie-in solution was constructed in other areas of the HSDRRS. We think the Corps should consider a configuration where the required 26’ wall height is carried all the way to the crest of the MRL. At that point, the wall should turn and run parallel to the MRL, and gradually transition down the MRL crest elevation. Such a configuration would move the focal point of the wave energy away from the sharp corner at the LPV 149-MRL tie in, and direct the overtopping into the MRL and away from the interior of the St. Bernard polder” – Robert Turner, Regional Director SLFPA-E. 12/12/2011

BLUF

As constructed, with a portion of the tie-in at 19.96 to 20.3 ft NAVD88 2004.65, the LPV-149 tie-in does not meet HSDRRS wave overtopping design criteria for 1% existing conditions. However, the additional volume of overtopping for the 1% existing condition will not considerably impact interior stage levels, and can be handled by the interior drainage system with minimal increase in water level. There are other portions of the HSDRRS system, such as I-310 and I-10, where overtopping criteria are not met; the design for these areas includes scour protection and the additional volume entering the polder does not adversely impact interior water levels.

Scour protection exceeds generalized HSDRRS design recommendations, and is adequate to reduce scour for the 1% and 0.2% events for existing and future conditions. The scour protection may need to be keyed in, so as not to incur scour under the concrete should the adjacent Mississippi River levee experience scour from overtopping.

At the 0.2% resiliency event for existing condition, significant increase in interior water levels will occur (> 2 ft) because of the large scale free-flow overtopping and the size of the storage area bounded by the HSDRRS system and the original levee. The increase in interior water level is even greater in the future for the resiliency event. This is different than other portions of the HSDRRS system, where the interior polder can handle the additional volume of water with minimal increase in water level.

For the future, at roughly 2030, the additional volume of overtopping will add at least 0.5 ft to interior water levels within the storage area bounded by the HSDRRS system and the original levee for the 1% annual exceedence exterior surge elevation and associated waves.

The Mississippi River levee section 82E-L meets existing condition requirements. However, over time, 82E-L will also become deficient, as the required future design elevation of 21.0 ft

NAVD88, exceeds the existing levee elevation of 20.0 ft NAVD88. Analysis has shown that the 82E-L levee section will become deficient in 2056, near the end of the project life.

Analysis

The P&S drawings were referenced for the elevations and lengths of the step-down construction of the tie-in.

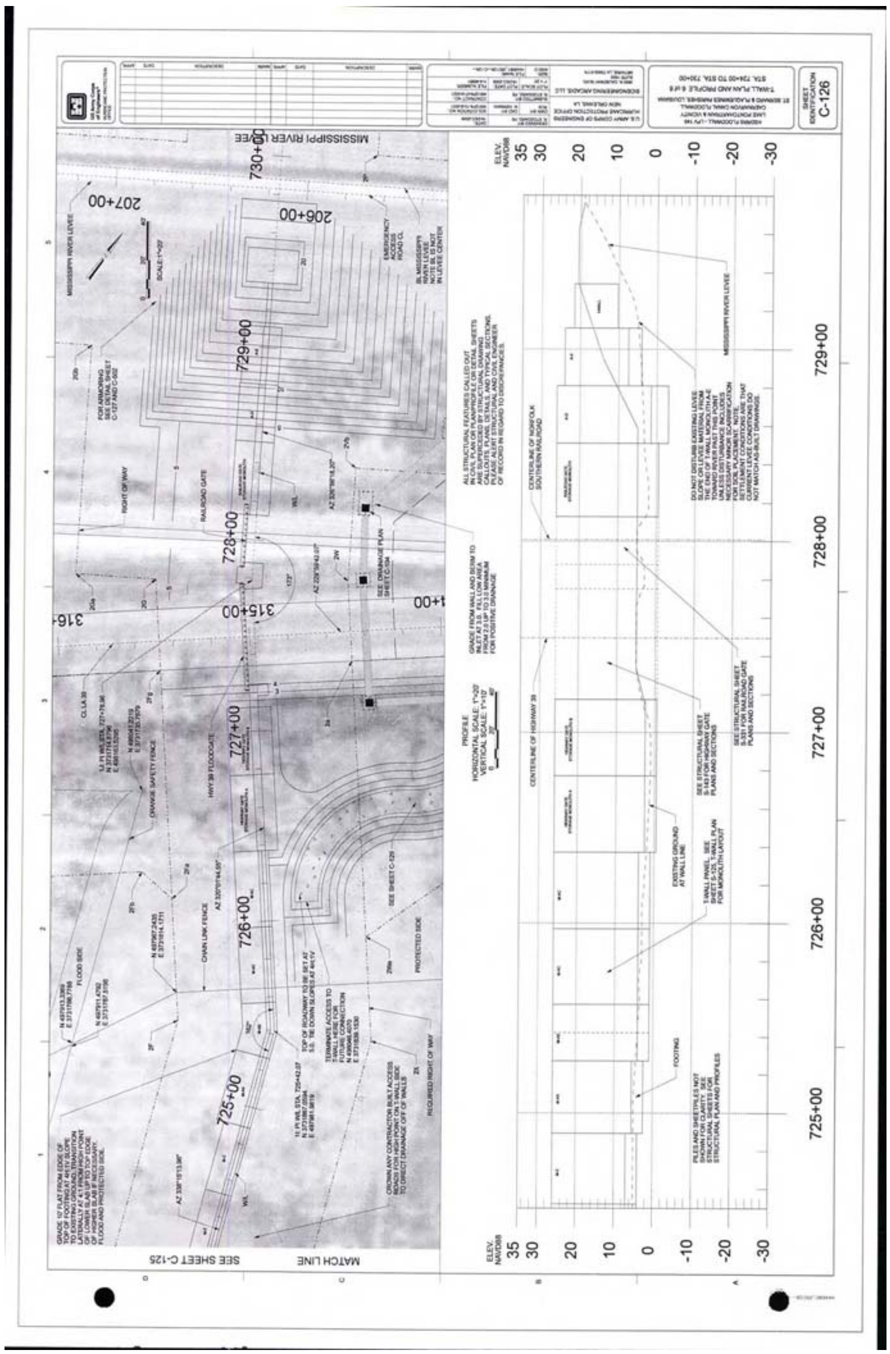

Figure 1 P&S of Tie-In

For the overtopping calculations, the tie-in was divided into 4 sections.

Table 1 summarizes the four sections of tie-in.

Type	Elevation (ft. NAVD88 2004.65)	Length (ft.)
T-Wall	24.00	30.00
I-Wall	21.50	22.50
Levee	20.30	40.00
Levee	19.96	6.00

Table 1 Levee and Floodwall Sections at LPV-149 Tie-in

Figure 2 Aerial photo with structure elevations as built

As a response to this concern from the levee board, we have taken a closer look at the hydraulic boundary conditions used for design of the LPV149 tie-in. Figure 2 shows the hydraulic boundary conditions that were used for design of the tie-in and surrounding levees/floodwalls. Labels and alignments colored purple are meant to represent floodwalls, which are designed and built for future 1% surge and wave conditions. Labels and alignments colored blue represent levee alignments, which are currently designed and built for 1% existing condition hydraulic boundary conditions. Future levee lifts will ensure the system meets 1% risk reduction, as time goes on.

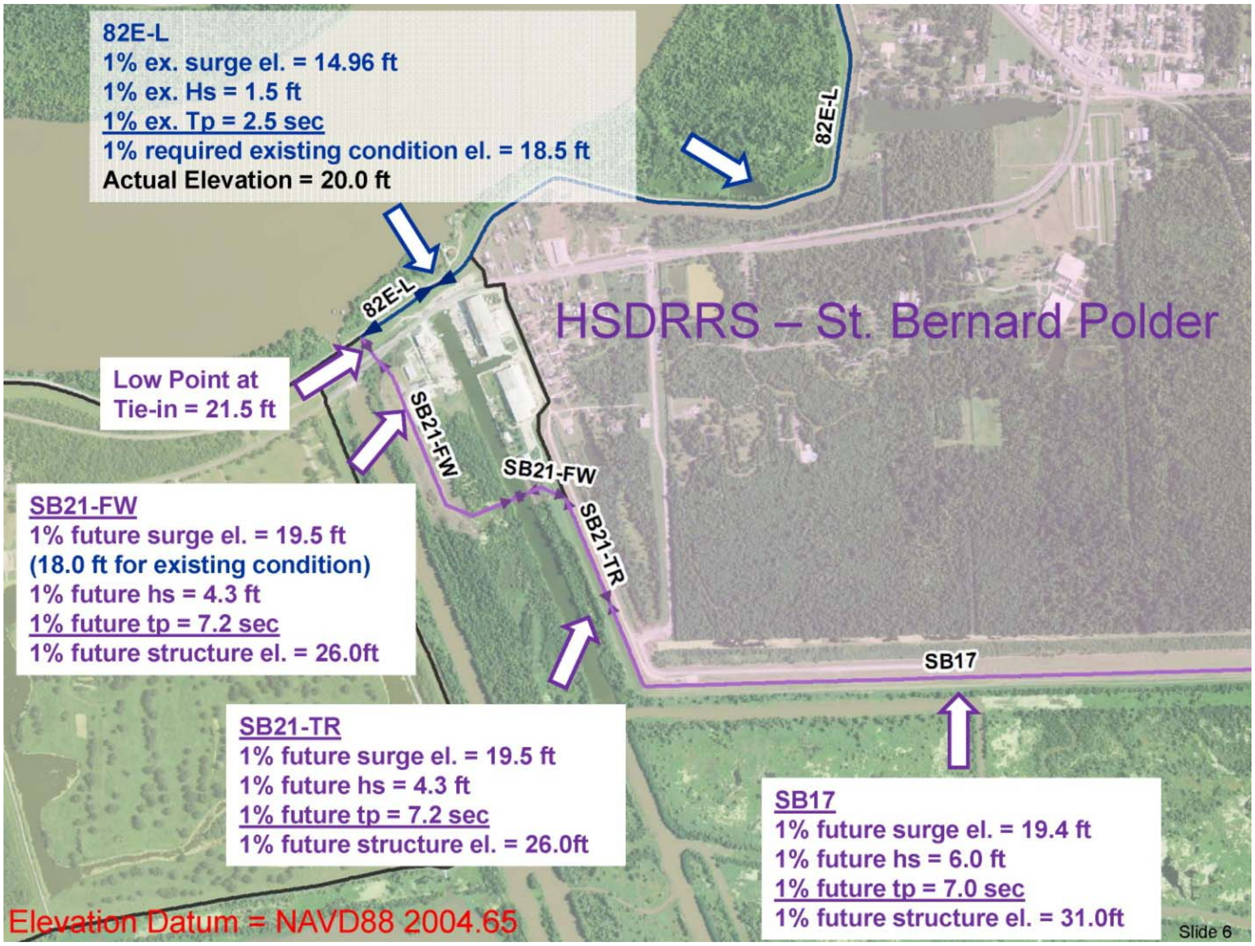

Figure 3 Hydraulic boundary conditions at LPV149

The crucial issue with the design of the tie-in concerns the propagation of waves and surge deep within the pocket that is formed by the tie-in, the MRL levee, and the non-federal back levee. A SWAN wave model was used to calculate the existing and future 1% wave conditions within the pocket. With this local modeling approach, we apply 1% wave conditions along the southern boundary of the mesh domain, and allow the waves to propagate through the domain. No wind has been applied in the model. We are simply propagating the large waves that develop in the inner Caernarvon Marsh, to the tie-in. The model accounts for wave breaking / depth limitation that occurs as the waves propagate to the tie-in, but does not account for the influence of vegetation, or reflection at the adjacent floodwalls and levees. Figures 4 through 7 display the 2D SWAN results.

Table 2 summarizes existing and future 1% wave conditions from the recent SWAN wave modeling at the tie-in. Using the waves calculated with SWAN, overtopping rates for 1% existing and future conditions have been calculated at each individual section of the tie-in. Overtopping rates at each individual section of tie-in are also presented in Table 2.

Condition	Tie-In Section	1% SWAN Significant Wave Height (ft.)	1% SWAN Peak Wave Period (sec.)	1% Surge Elevation (ft. NAVD88 2004.65)*	Design Elevation at tie-in (ft. NAVD88 2004.65)	Overtop. Rate (cfs/ft) Q50 Q90	Meets HSDRRS Overtopping Criterion (Q50<0.01 cfs/ft for levees, Q50<0.03 cfs/ft for floodwalls, Q90<0.10 cfs/ft)
Existing	24’ T-Wall	2.4	5.9	18.0*	24	Q50=0.0002 Q90=0.0023	Yes
Existing	21.5’ I-Wall	2.4	5.9	18.0*	21.5	Q50=0.0091 Q90=0.0718	Yes
Existing	20.3’ Levee	2.4	5.9	18.0*	20.3	Q50=0.3615 Q90=1.5184	No
Existing	19.96’ Levee	2.4	5.9	18.0*	19.96	Q50=0.5157 Q90=2.0937	No
Future	24’ T-Wall	2.99	7.81	19.5	24	Q50=0.0109 Q90=0.0818	Yes
Future	21.5’ I-Wall	2.99	7.81	19.5	21.5	Q50=0.227 Q90=1.4338	No
Future	20.3’ Levee	2.99	7.81	19.5	20.3	Q50=3.0021 Q90=10.901	No
Future	19.96’ Levee	2.99	7.81	19.5	19.96	Q50=3.9632 Q90=14.895	No
* From SB21-TR hydraulic boundary conditions in 1% design report							

Table 2 Summary of updated 1% hydraulic boundary conditions at LPV-149 tie-in

For the 0.2% resiliency check, the existing and future 0.2% surge elevations of 21.6 ft and 23.6 ft, respectively, are greater than the lowest as-built elevation of 19.96 ft. These hydraulic conditions provide free-flow overtopping and potential for scour and considerable interior flooding during a 0.2% surge event, for both existing and future conditions. Table 2 provides a summary of the 0.2% resiliency check.

Condition	Percent Chance Exceedance (% / Return Period)	Surge Elevation (ft. NAVD88 2004.65)*	Minimum Design Elevation at tie-in (ft. NAVD88 2004.65)	Overtop. Rate	Meets HSDRRS Resiliency Criterion
Existing 2007	0.2% / 500YR	21.6**	19.96	Free-flow	No
Future 2057	0.2% / 500YR	23.6*	19.96	Free-flow	No
* From SB21-TR hydraulic boundary conditions in 1% design report ** from ADCIRC output point Q237 (IHNC 152)					

Table 3 Summary of 0.2% resiliency check at LPV-149 tie-in

Overtopping of Selected Design Storms Approximating 1% and 0.2% Surge and Waves

Overtopping volumes were calculated for 3 storms that produce approximately 1% surge and waves at the tie in, as well as 3 storms that produce 0.2% surge and waves. These calculations provide an estimate of overtopping volume for 1% and 0.2% design conditions.

Wave overtopping calculations were performed for each section using the Van-der-Meer equations for levees, and the Franco and Franco equations for floodwalls. In the case of free-flow overtopping, the broad-crested weir equations were used for levee sections, and the rectangular weir equation was used for the floodwall sections. Table 4 summarizes the overtopping volume at each individual section, and the total overtopping volume for each storm.

Condition	Synthetic Storm ID	Peak Surge (ft. NAVD88 2004.65)	Normalized Peak Surge (ft. NAVD88 2004.65)	Overtopping volume (acre-ft)				
				24' T-wall	21.5' I-wall	20.3' levee	19.96' levee	Total
1% Existing	009	19.9	18	0.0	0.1	2.0	0.5	**2.6**
1% Existing	027	19.8	18	0.0	0.0	1.0	0.2	**1.3**
1% Existing	056	19.7	18	0.0	0.0	1.1	0.3	**1.4**
1% Future	009	19.9	19.5	0.2	1.8	27.3	5.7	**34.9**
1% Future	027	19.8	19.5	0.1	0.9	13.8	2.9	**17.7**
1% Future	056	19.7	19.5	0.1	0.9	13.8	2.8	**17.5**
0.2% Existing	094	22.6	21.6	1.1	12.6	63.1	11.2	**88.0**
0.2% Existing	057	22.5	21.6	0.8	8.8	45.4	8.1	**63.1**
0.2% Existing	146	22.2	21.6	1.5	16.5	82.1	14.5	**114.6**
0.2% Future	094	22.6	23.6	9.3	84.5	147.6	25.1	**266.5**
0.2% Future	057	22.5	23.6	6.4	59.9	107.9	18.4	**192.6**
0.2% Future	146	22.2	23.6	12.2	110.3	189.3	31.8	**343.7**

Table 4   Overtopping Volumes for 1% and 0.2% Synthetic Storms

Figure 4 displays the area bounded by the LPV-149 floodwall. Figure 5 displays the elevation-volume curve for the area bounded by the LPV-149 floodwall.

The overtopping volume for the 1% existing condition calculations is on the order of 1.3 to 2.5 acre-ft. Examination of the elevation-volume curve, presented in Figure 5, shows that additional volume flowing into the polder at the tie-in will not cause a substantial increase in interior stages for the 1% existing condition. For 1% existing conditions, the wave overtopping volume will flow into a small basin near the tie-in, labeled in Figure 4, that is formed by the high-ground of the MRL levee, the tie-in floodwall, an access road, and the railroad tracks. This small basin is approximately 1 to 1.5 acres, and approximately 1 to 2 ft deep. 1% existing condition overtopping, of 1.3-2.5 acre-ft will initially flow directly into this basin before being carried away by the interior drainage system. The increase in stage will be largest in the small basin, and relatively minor in the project area.

Overtopping calculations for the 1% future condition show volumes of approximately 16.5 to 32.9 acre-ft enter the polder at the tie-in. Using the elevation-volume curve presented in Figure 5, the additional volume will raise interior stages in the project area by 1.5 to 2.5 ft.

For the 0.2% percent existing and future events, the overtopping volume (largely free-flow) will likely raise water levels considerably at the project area, depending on the interior drainage of the system. Flow entering the project area at the tie-in will ultimately have to be drained by St. Mary #8 pump station, which is approximately 7 miles from the tie-in. Water overtopping the tie-in could also flow down highway 39 and enter the adjacent polder through the opening where the original tie-in meets the MRL. Without modeling, the potential negative effects of water taking this pathway cannot be determined. In this analysis, we assume water will stay within the project area presented in Figure 4.

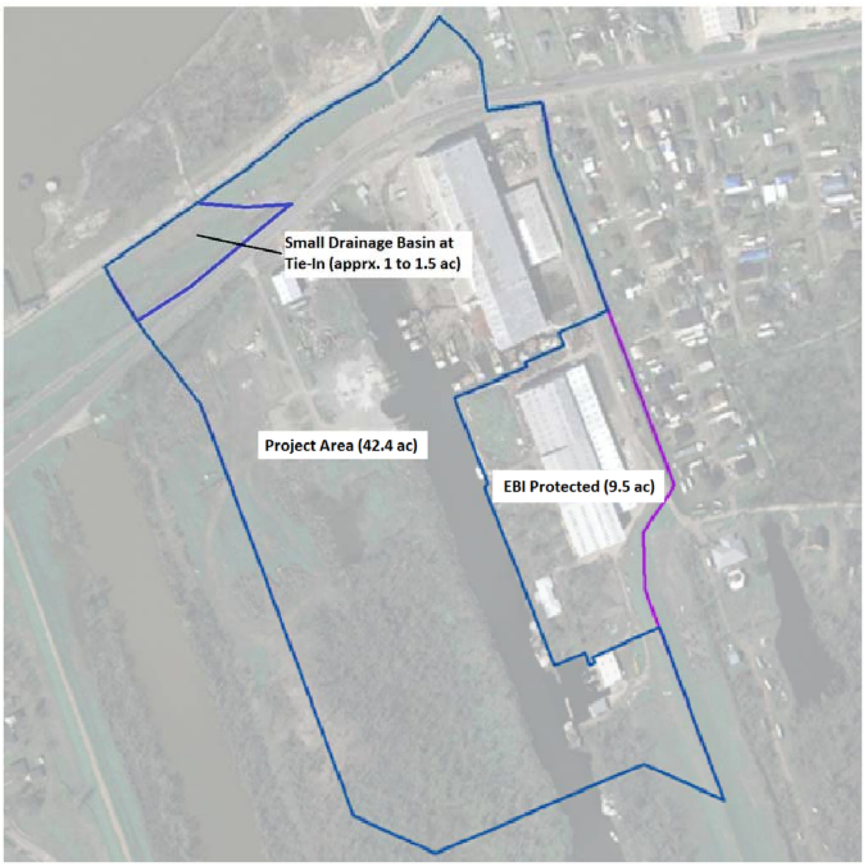

Figure 4 Project area omitting portion of EBI protected by existing floodwall (42.4 acres)

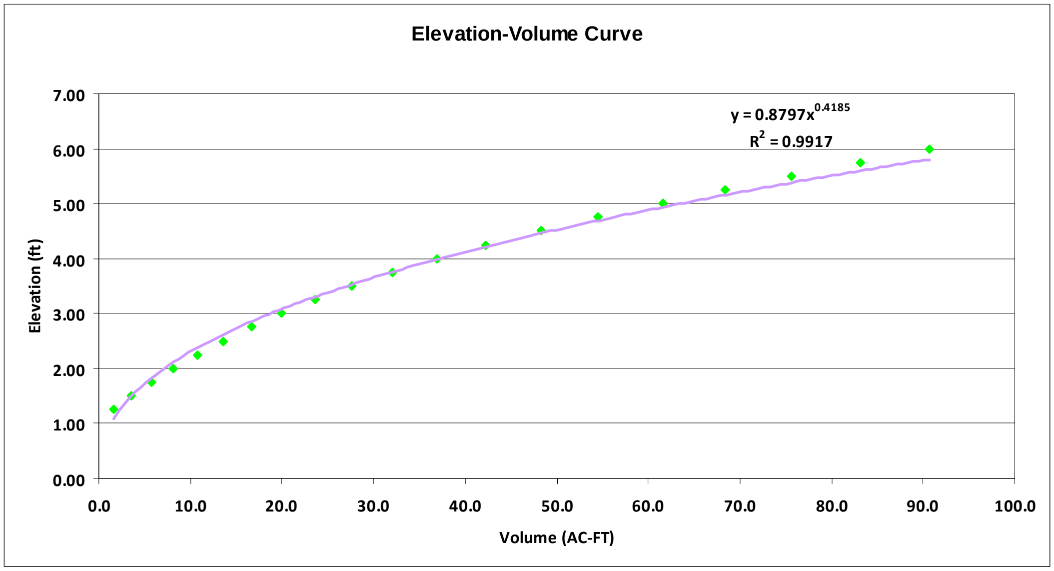

Figure 5 Elevation-Volume Curve (Elevation 1.0 ft = 0.0 ac-ft volume) (excluding area bounded by the EBI Floodwall

Armoring Discussion

When the original design of the tie-in was completed in 2009, the designer indicated "*As a conservative assumption, the transition must extend along the MRL 100 ft upstream and 100 ft downstream of the tie-in. The transition must also extend along the Caernarvon levee from the MRL levee to the 26 ft floodwall elevation.*"

The scour protection has been built according to the original design, with scour protection extending 100ft from tie-in (as pictured in Figure 2). Generalized HSDRRS design guidelines require 30' of scour protection at transitions. Summarized in the IPET report, observations of scour caused by Katrina at levee/floodwall transitions support this generalized guideline. Results of a Texas A&M physical modeling study titled "Levee Transition Study" further support the HSDRRS scour protection length guideline at transitions. The Texas A&M study shows that increased bottom velocities at transitions occur on the order of 30' from the transition. Given the findings in the Texas A&M study and IPET report, the 100' length of armoring at the tie-in is sufficient to reduce scour caused by overtopping at the transition during substantial overtopping. Given the overdesign of the concrete scour protection, it is expected to remain in-place for the overtopping rates expected during 1% and 0.2% events, for both existing and future conditions.

Conclusions and Recommendations

As constructed, with a portion of the tie-in at 19.96 to 20.3 ft NAVD88 2004.65, the LPV-149 tie-in does not meet HSDRRS wave overtopping design criteria for 1% existing conditions. However, the additional volume of overtopping for the 1% existing condition will not considerably impact interior stage levels, and can be handled by the interior drainage system with minimal increase in water level. There are other portions of the HSDRRS system, such as I-310 and I-10, where overtopping criteria are not met; the design for these areas includes scour protection and the additional volume entering the polder does not adversely impact interior water levels.

Scour protection exceeds generalized HSDRRS design recommendations, and is adequate to reduce scour for the 1% and 0.2% events for existing and future condition. The scour protection may need to be keyed in, so as not to incur scour under the concrete should the adjacent Mississippi River levee experience scour from overtopping.

At the 0.2% resiliency event for existing condition, significant increase in interior water levels will occur (> 2 ft) because of the large scale free-flow overtopping and the size of the storage area bounded by the HSDRRS system and the original levee. The increase in interior water level is even greater in the future for the resiliency event. This is different than other portions of the HSDRRS system, where the interior polder can handle the additional volume of water with minimal increase in water level.

For the future, at roughly 2030, the additional volume of overtopping will add at least 0.5 ft to interior water levels within the storage area bounded by the HSDRRS system and the original levee for the 1% annual exceedence exterior surge elevation and associated waves.

The Mississippi River levee section 82E-L meets existing condition requirements. However, over time, 82E-L will also become deficient, as the required future design elevation of 21.0 ft NAVD88, exceeds the existing levee elevation of 20.0 ft NAVD88. Analysis has shown that the 82E-L levee section will become deficient in 2056, near the end of the project life.

For future conditions, the tie-in could be raised to a minimum height of 24 ft, extended up the MRL, and armored to meet the HSDRRS overtopping criteria, ensure that surge will enter the Mississippi River instead of the St. Bernard Polder, and limit the increase in water levels for the resiliency event to around 1 ft. **The floodwall should extend up the MRL by a distance of 70 ft to ensure water passing over the MRL levee does not wrap around and flow into the St. Bernard polder. The existing scour pad extends upriver 100ft from the existing tie-in structure. If the tie-in is extended up-river 70 ft, the remaining 30ft of existing scour protection at the transition will be sufficient. Figure 6 presents a plan view of the proposed modification of the tie-in. This alternative would reduce overtopping rates to meet HSDRRS criteria, and reduce possibility of scour.**

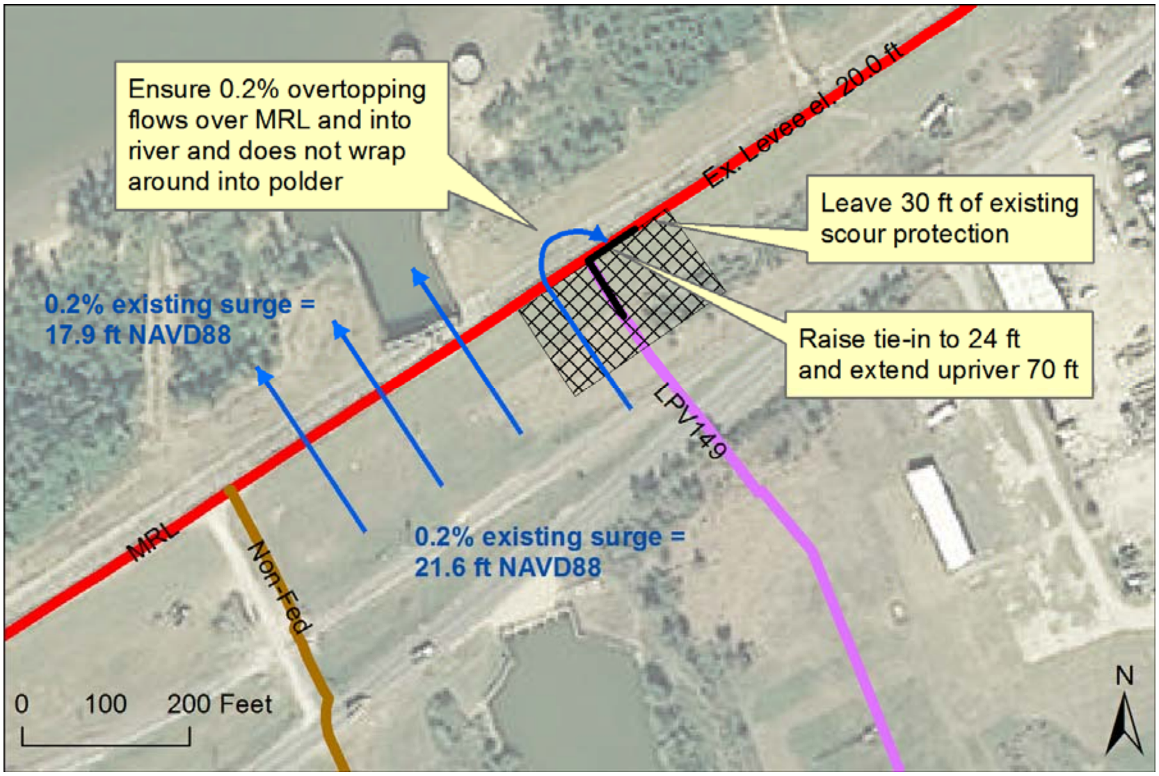

Figure 6 Modification of LPV-149 Tie-In

Another alternative to reduce the interior flooding would be to allow the overtopping, and add a pump to transfer the water to the St. Mary's pumpstation. The pump would be capable of transferring the 0.2% existing and future volumes entering the polder. Based on the overtopping rates and length of the tie-in, the peak flow entering the polder at the tie-in will be < 560 cfs with 90% confidence, and < 149 cfs with 50% confidence. The water could be pumped back over the floodwall at the floodgate, or transferred to the St. Mary's pumpstation.

SWAN model output at Caernarvon:

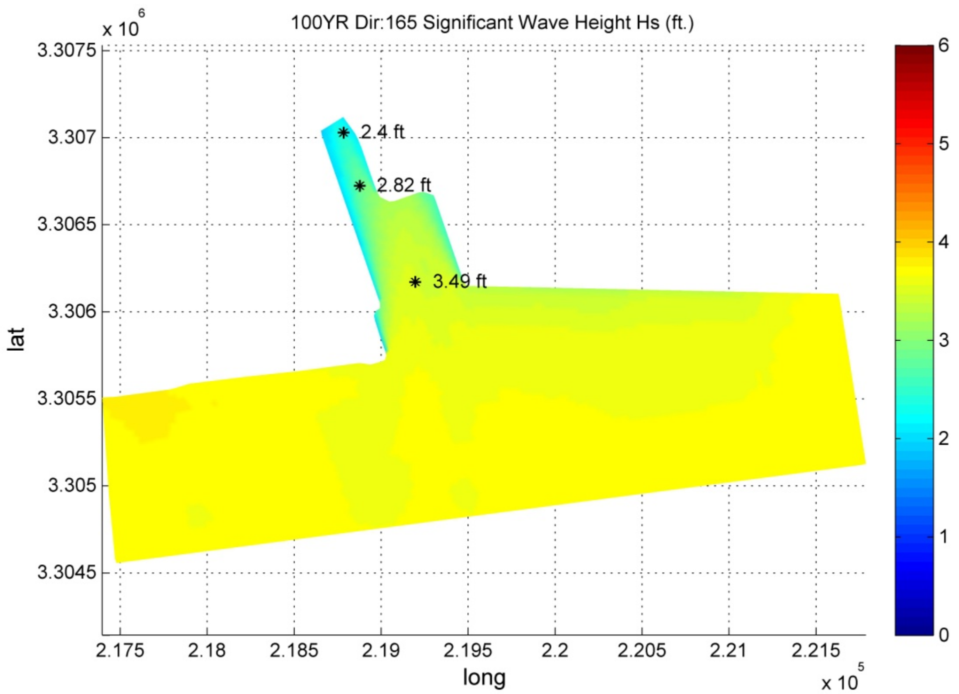

Figure 7 Existing Condition 1% Significant Wave Height (ft.)

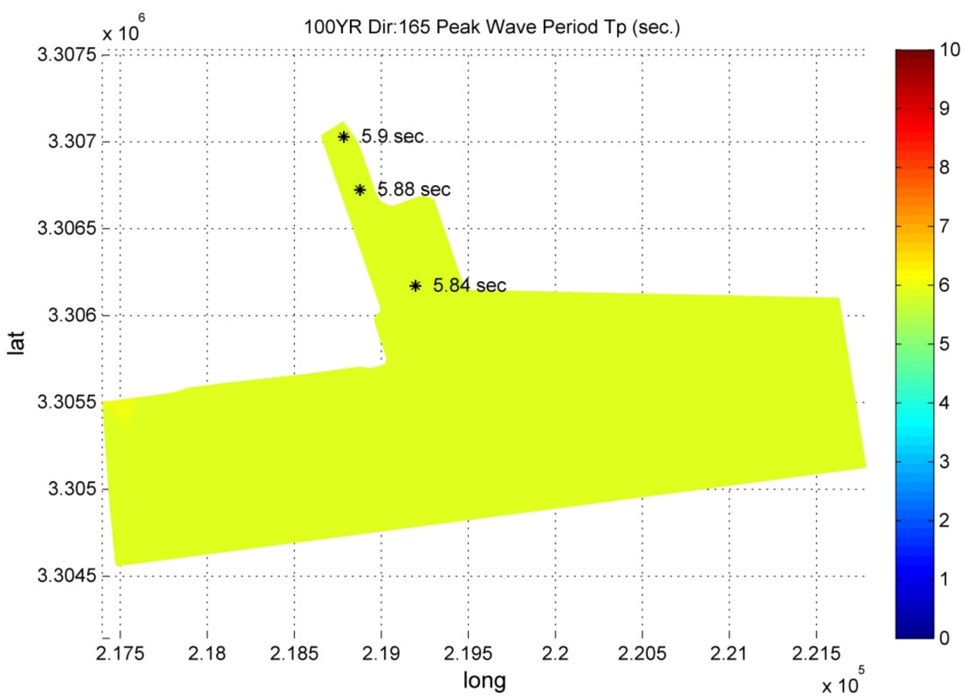

Figure 8 Existing Condition 1% Peak Wave Period (sec.)

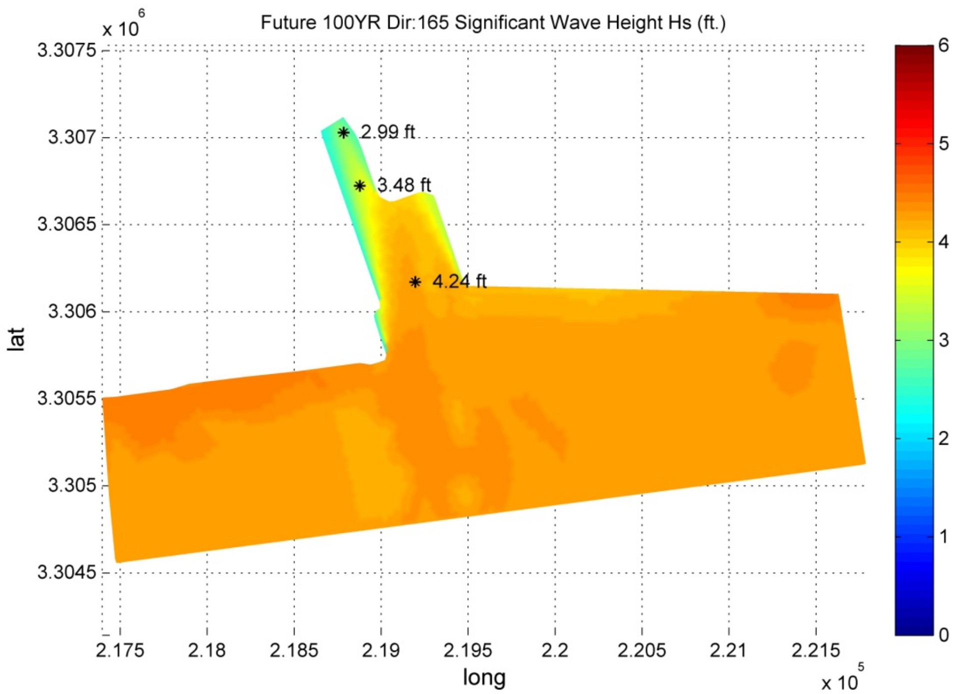

Figure 9   Future Condition 1% Significant Wave Height (ft.)

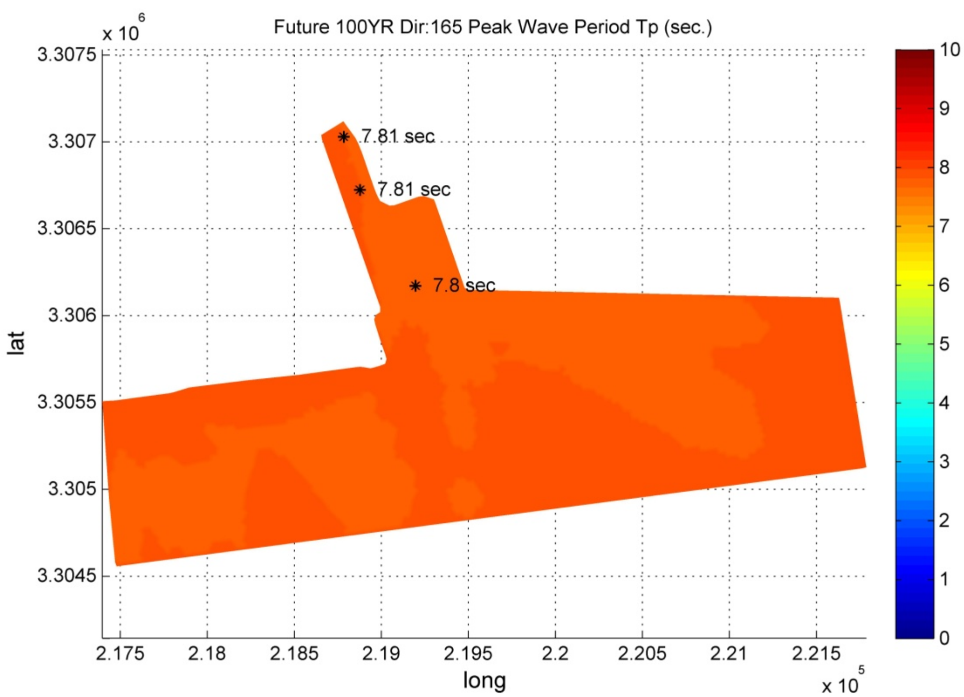

Figure 10   Future Condition 1% Peak Wave Period (sec.)

Appendix T

Overtopping Design Criteria Tables

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St Charles Parish
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
SC01-A1	St Charles Return Wall 17.5ft	Structure /Wall	Future	10.9	0.7	13.4	4.6	4.4	0.3	7.2	1.3	17.5	10.9	-	-	-	-	-	-	-	-	0.013	0.047
SC01-A2	St Charles Return Wall 17.0ft	Structure /Wall	Future	11.6	0.8	14.5	3.9	3.9	0.3	4.9	0.9	17.0	11.6	-	-	-	-	-	-	-	-	0.022	0.078
SC02-A	St Charles Parish Levee west of I-310	Levee	Existing	11.0	0.8	13.8	2.3	2.3	0.2	4.2	0.8	14.5	11.0	4	11.0	6	7.5	0.82	1	1	1	0.007	0.082
SC02-A	St Charles Parish Levee west of I-310	Levee	Future	12.5	0.8	15.3	3.1	3.1	0.2	4.8	0.8	16.5	12.5	4	12.5	8	7.8	0.67	1	1	1	0.006	0.065
SC02-B	St Charles Parish Levee east of I-310	Levee	Existing	10.5	0.8	13.5	1.6	1.6	0.2	3.2	0.6	14.0	10.5	3	-	-	-	1	1	-	1	0.004	0.041
SC02-B	St Charles Parish Levee east of I-310	Levee	Future	12.0	0.8	15.0	2.4	2.4	0.2	3.9	0.6	15.5	12.0	4	12.0	6	8.4	0.81	1	1	1	0.004	0.048
SC04	St Rose Canal Drainage Structure	Structure /Wall	Future	11.9	0.9	15.2	2.7	2.7	0.2	4.5	0.8	16.5	11.9	-	-	-	-	-	-	-	-	0.010	0.065
SC04-G	St Rose Canal Drainage Gate	Structure /Wall	Future	11.9	0.9	15.2	2.7	2.7	0.2	4.5	0.8	16.5	11.9	-	-	-	-	-	-	-	-	0.010	0.065
SC05-FW	Good Hope Floodwall	Structure /Wall	Future	12.5	0.8	15.4	3.1	3.1	0.2	4.7	0.8	17.0	12.5	-	-	-	-	-	-	-	-	0.020	0.078
SC05-G	Good Hope Gate	Structure /Wall	Future	12.5	0.8	15.4	3.1	3.1	0.2	4.7	0.8	17.0	12.5	-	-	-	-	-	-	-	-	0.020	0.078
SC06	Gulf South Pipeline T-Wall	Structure /Wall	Future	12.5	0.8	15.5	3.1	3.1	0.2	4.8	0.8	17.0	12.5	-	-	-	-	-	-	-	-	0.020	0.077
SC07	Cross Bayou Canal T-Wall	Structure /Wall	Future	12.5	0.8	15.4	3.1	3.1	0.2	4.7	0.8	17.0	12.5	-	-	-	-	-	-	-	-	0.019	0.078
SC08-FW1	Bayou Tepagnier Complex FP	Structure /Wall	Future	12.5	0.8	15.3	2.7	2.7	0.2	4.0	0.7	*18.5*	12.5	-	-	-	-	-	-	-	-	0.001	0.004
SC08-FW2	Bayou Tepagnier Complex T-walls	Structure /Wall	Future	12.5	0.8	15.3	2.7	2.7	0.2	4.0	0.7	16.5	12.5	-	-	-	-	-	-	-	-	0.001	0.004
SC09	Almedia Drainage Structure	Structure /Wall	Future	12.0	0.8	15.0	2.4	2.4	0.2	3.9	0.6	15.5	12.0	-	-	-	-	-	-	-	-	0.012	0.066
SC09-G	Almedia Drainage Gate	Structure /Wall	Future	12.0	0.8	15.0	2.4	2.4	0.2	3.9	0.6	15.5	12.0	-	-	-	-	-	-	-	-	0.012	0.066
SC10	Walker Drainage Structure	Structure /Wall	Future	11.9	0.8	14.9	2.5	2.5	0.2	3.8	0.6	15.5	11.9	-	-	-	-	-	-	-	-	0.015	0.071
SC10-G	Walker Drainage Gate	Structure /Wall	Future	11.9	0.8	14.9	2.5	2.5	0.2	3.8	0.6	15.5	11.9	-	-	-	-	-	-	-	-	0.015	0.071
SC11	Bonnet Carre Tie-in Floodwall	Structure /Wall	Future	12.5	0.8	15.3	2.7	2.7	0.2	4.0	0.7	*18.5*	12.5	-	-	-	-	-	-	-	-	0.001	0.004
SC12-FW1	I-310 Floodwall	Structure /Wall	Future	12.0	0.9	15.1	2.3	2.3	0.2	3.9	0.6	13.5	12.0	-	-	-	-	-	-	-	-	0.009	0.054
SC12-FW2	I-310 Floodwall	Structure /Wall	Future	12.0	0.9	15.1	2.3	2.3	0.2	3.9	0.6	15.5	12.0	-	-	-	-	-	-	-	-	0.009	0.054
SC13-FW	ICRR (CNRR) Gate Monolith	Structure /Wall	Future	11.8	0.8	14.8	2.4	2.4	0.2	4.0	0.7	15.5	11.8	-	-	-	-	-	-	-	-	0.009	0.050
SC13-G	ICRR (CNRR) Gate	Gate	Future	11.8	0.8	14.8	2.4	2.4	0.2	4.0	0.7	15.5	11.8	-	-	-	-	-	-	-	-	0.009	0.050
SC14	Airport Runway Levee	Levee	Existing	10.3	0.8	13.3	1.9	1.9	0.2	3.8	0.8	14.0	5.7	4	-	-	-	1	1	-	1	0.004	0.050
SC14	Airport Runway Levee	Levee	Future	11.8	0.8	14.8	2.7	2.7	0.2	4.5	0.8	15.5	7.2	4	11.8	8	7.7	0.7	1	1	1	0.004	0.051
SC15	Shell Pipeline Floodwall	Structure /Wall	Future	12.5	0.8	15.4	3.1	3.1	0.2	4.7	0.8	17.0	12.5	-	-	-	-	-	-	-	-	0.012	0.051
SC30	West Return Wall Transition	Structure /Wall	Future	11.8	0.8	14.7	2.9	2.9	0.2	5.0	0.9	16.0	11.8	-	-	-	-	-	-	-	-	0.011	0.049

Values in red italics include structural superiority.

Jefferson Parish
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
JL01	Lakefront Levee	Levee	Existing	9.0	0.6	11.2	7.5	4.1	0.4	7.7	1.5	15.5	10.3	4	10.0	15	4.5	0.6	1	1	1	0.001	0.016
JL01	Lakefront Levee	Levee	Future	10.5	0.6	12.7	8.3	4.7	0.4	8.3	1.5	17.5	11.8	4.25	10.0	15	4.5	0.6	1	1	1	0.002	0.024
JL02bw	Pump Station #1 (Bonnabel) with Breakwater at 14 ft	Structure /Wall	Future	10.3	0.7	12.7	8.3	7.1	0.3	8.1	1.6	14.0	17.8	-	-	-	-	-	-	-	-	*	*
JL02	Bonnabel PS #1 Fronting Protection	Structure /Wall	Future	10.3	0.7	12.7	*	2.5	0.3	8.1	1.6	*16.0*	*	-	-	-	-	-	-	-	-	0.001	0.003
JL03bw	Pump Station #2 (Suburban) with Breakwater at 13.2 ft	Structure /Wall	Future	10.4	0.7	12.7	8.2	7.0	0.3	8.1	1.6	13.2	17.5	-	-	-	-	-	-	-	-	*	*
JL03	Elmwood PS #2 Fronting Protection	Structure /Wall	Future	10.4	0.7	12.7	*	2.8	0.3	8.1	1.6	*16.0*	*	-	-	-	-	-	-	-	-	0.002	0.009
JL04bw	Pump Station #3 (Elmwood) with Breakwater at 10 ft	Structure /Wall	Future	10.5	0.6	12.7	8.4	7.6	0.4	8.1	1.6	10.0	19.0	-	-	-	-	-	-	-	-	*	*
JL04	Suburban PS #3 Fronting Protection	Structure /Wall	Future	10.5	0.6	12.7	*	4.2	0.4	8.1	1.6	*18.5*	*	-	-	-	-	-	-	-	-	0.004	0.016
JL05bw	Pump Station #4 (Duncan) with Breakwater at 14 ft	Structure /Wall	Future	10.5	0.7	12.8	8.2	7.1	0.3	8.1	1.6	14.0	17.8	-	-	-	-	-	-	-	-	*	*
JL05	Duncan PS #4 Fronting Protection	Structure /Wall	Future	10.5	0.7	12.8	*	2.5	0.3	8.1	1.6	*16.0*	*	-	-	-	-	-	-	-	-	0.001	0.004
JL06	Causeway Northbound & Southbound T-wall	Structure /Wall	Future	10.3	0.7	12.7	8.2	1.3	0.6	7.8	1.5	*15.0*	3.3	-	-	-	-	-	-	-	-	0.001	0.002
JL07	Wlliams Blvd Floodgate	Structure /Wall	Future	10.4	0.6	12.6	9.2	2.8	0.2	8.5	1.5	*16.5*	6.9	-	-	-	-	-	-	-	-	0.000	0.003
JL08	Bonnabel Boat Launch FG	Structure /Wall	Future	10.3	0.7	12.7	8.3	2.7	0.2	8.3	1.5	*16.5*	6.8	-	-	-	-	-	-	-	-	0.000	0.003
JL09	Return Wall	Structure /Wall	Future	10.3	0.7	13.1	8.2	4.9	0.4	8.3	1.6	17.5	12.3	-	-	-	-	-	-	-	-	0.028	0.086
JL10	US Coast Guard Station Levee	Levee	Existing	8.7	0.7	11.3	8.1	2.3	0.2	7.2	1.4	13.5	5.7	4	-	-	-	1	1	-	-	0.01	0.062
JL10	US Coast Guard Station Levee	Levee	Future	10.2	0.7	12.8	8.9	2.9	0.2	8.1	1.4	17.0	7.2	4	-	-	-	1	1	-	-	0.0081	0.042

Values in red italics include structural superiority.

Orleans Parish Metro Lakefront
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
NO01	New Orleans Lakefront Levee	Levee	Existing	8.7	0.7	11.3	8.1	5.1	0.5	7.2	1.4	16.0	12.8	5	4.0	Horiz	4.0	0.73	1	1	1	0.007	0.063
NO01	New Orleans Lakefront Levee	Levee	Future	10.2	0.7	12.8	8.9	5.7	0.5	7.6	1.4	19.0	14.3	5	4.0	Horiz	4.0	0.78	1	1	1	0.008	0.067
NO06-FW	New Orleans Marina Floodwall	Structure /Wall	Future	10.2	0.7	12.8	8.9	3.3	0.3	8.0	1.6	16.0	8.3	-	-	-	-	-	-	-	-	0.003	0.020
NO06-LT	New Orleans Marina Lv/FW Combo	Structure /Wall	Future	10.2	0.7	12.8	8.9	3.3	0.3	8.0	1.6	16.0	8.3	4	-	-	-	1	1	-	-	0.003	0.020
NO07-A	Bayou St. John Lakefront Floodwall	Structure /Wall	Future	10.1	0.8	13.1	7.8	4.4	0.4	7.4	1.5	16.0	11.0	-	-	-	-	-	-	-	-	0.026	0.093
NO07-B	Bayou St. John Bayou Floodwall	Structure /Wall	Future	10.1	0.8	13.1	7.8	3.0	0.3	4.0	0.8	16.0	7.5	-	-	-	-	-	-	-	-	0.002	0.011
NO07-BL	Bayou St. John Landward of Lakeshore Dr	Levee	Existing	8.6	0.8	11.6	7.0	3.0*	0.3	4.0*	0.8	15.0	6.0	3	-	-	-	1	1	-	-	0.007	0.084
NO07-BL	Bayou St. John Landward of Lakeshore Dr	Levee	Future	10.1	0.8	13.1	7.8	3.0*	0.3	4.0*	0.8	16.5	7.5	3	-	-	-	1	1	-	-	0.008	0.058
NO07-C	Bayou St. John Sector Gate	Structure /Wall	Future	10.1	0.8	13.1	7.8	3.0	0.3	4.0	0.8	16.0	7.5	-	-	-	-	-	-	-	-	0.002	0.011
NO08	Pontchartrain Beach Floodwall	Structure /Wall	Future	10.1	0.8	12.9	8.4	3.6	0.3	7.3	1.3	16.0	9.0	-	-	-	-	-	-	-	-	0.007	0.033
NO09	American Standard Floodwall	Structure /Wall	Future	10.1	0.8	12.8	8.0	4.4	0.6	7.1	1.4	16.5	11.0	-	-	-	-	-	-	-	-	0.028	0.096
NO10-LI	Topaz Street Lv/FW Combo	Structure /Wall	Future	10.2	0.7	12.8	8.1	2.9	0.3	8.1	1.7	18.0	7.3	3	-	-	-	1	1	-	-	0.003	0.020
NO10-LL	Topaz Street Lv/FW Combo	Structure /Wall	Future	10.2	0.7	12.8	8.9	2.9	0.3	8.1	1.7	16.0	7.3	3	-	-	-	1	1	-	-	0.003	0.020
NO11	London Ave Outfall Canal Closure	Structure /Wall	Future	10.1	0.8	12.9	8.4	2.2	0.2	3.4	0.7	18.0	7.5	-	-	-	-	-	-	-	-	0.000	0.001
NO12	Orleans Ave Outfall Canal Closure	Structure /Wall	Future	10.2	0.8	13.1	8.7	3.3	0.3	5.9	1.2	18.0	7.5	-	-	-	-	-	-	-	-	0.001	0.003
NO13	17th St Outfall Canal Closure	Structure /Wall	Future	10.2	0.7	12.8	8.9	7.2	0.7	6.9	1.4	18.0	22.2	-	-	-	-	-	-	-	-	0.004	0.020
NO14-G1	Floodgate nr Seabrook L-11 (L-13)	Structure /Wall	Future	10.1	0.8	13.1	7.1	2.5	0.2	7.9	1.4	16.0	6.3	-	-	-	-	-	-	-	-	0.000	0.002
NO14-G2	Ramp at Leroy Johnson Drive L-10	Levee	Existing	8.6	0.8	11.6	6.3	1.9	0.2	6.9	1.4	16.0	4.8	4	-	-	-	-	-	-	-	0.000	0.002
NO14-G2	Ramp at Leroy Johnson Drive L-10	Levee	Future	10.1	0.8	13.1	7.1	2.5	0.2	7.9	1.4	19.0	6.3	4	-	-	-	-	-	-	-	0.000	0.002
NO14-G3	Floodgate at Marconi	Structure /Wall	Future	10.2	0.8	13.1	8.7	2.5	0.2	7.9	1.4	16.0	-	-	-	-	-	-	-	-	-	0.000	0.001
NO14-L1A	West Roadway Gate	Structure /Wall	Future	10.2	0.8	13.1	8.9	2.5	0.2	7.9	1.4	16.0	7.3	-	-	-	-	-	-	-	-	0.000	0.002
NO14-L1	West Marina Gate	Structure /Wall	Future	10.2	0.8	13.1	8.9	2.5	0.2	7.9	1.4	16.0	8.3	-	-	-	-	-	-	-	-	0.000	0.002
NO14-L2	East Marina Gate	Structure /Wall	Future	10.2	0.8	13.1	8.9	2.5	0.2	7.9	1.4	16.0	8.3	-	-	-	-	-	-	-	-	0.000	0.002
NO14-L4A	Pontchartrain Blvd Gate	Structure /Wall	Future	10.2	0.8	13.1	8.9	2.5	0.2	7.9	1.4	16.0	6.3	-	-	-	-	-	-	-	-	0.000	0.002
NO14-G4	Floodgate at Lakeshore Drive just N of Lake Marina Av	Structure /Wall	Future	10.2	0.8	13.1	8.9	2.5	0.2	7.9	1.4	16.0	6.3	-	-	-	-	-	-	-	-	0.000	0.002
NO15-G2	Lakeshore Drive Floodgate W of London Av Canal	Structure /Wall	Future	10.1	0.8	12.9	8.4	1.6	0.1	8.7	1.4	18.5	4.0	-	-	-	-	-	-	-	-	0.000	0.001

Orleans Parish Metro Lakefront 1% Overtopping Design Criteria																								
				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
NO15-G3	Ramp at Lakeshore Dr west of Elysian Fields	Levee	Existing	8.7	0.8	11.8	7.9	1.9	0.2	7.2	1.4	16.0	4.8	4	-	-	-	-	-	-	-	0.000	0.002
NO15-G3	Ramp at Lakeshore Dr west of Elysian Fields	Levee	Future	10.2	0.8	13.1	8.7	2.5	0.2	7.9	1.4	19.0	6.3	4	-	-	-	-	-	-	-	0.000	0.002
NO15-G4	Ramp at Lakeshore Dr east of Elysian Fields	Levee	Existing	8.6	0.8	11.4	7.2	1.0	0.1	6.9	1.4	16.0	2.5	4	-	-	-	-	-	-	-	0.000	0.001
NO15-G4	Ramp at Lakeshore Dr east of Elysian Fields	Levee	Future	10.1	0.8	12.9	8.0	1.6	0.1	8.7	1.4	19.0	4.0	4	-	-	-	-	-	-	-	0.000	0.001
NO15-G5	Ramp at Franklin Avenue	Levee	Existing	8.6	0.8	11.4	7.2	1.0	0.1	6.9	1.4	16.0	2.5	4	-	-	-	-	-	-	-	0.000	0.001
NO15-G5	Ramp at Franklin Avenue	Levee	Future	10.1	0.8	12.9	8.0	1.6	0.1	8.7	1.4	19.0	4.0	4	-	-	-	-	-	-	-	0.000	0.001
NO15-L9A (L-10)	West Floodgate at Pontchartrain Bch	Structure /Wall	Future	10.1	0.8	12.9	8.2	1.6	0.1	8.7	1.4	16.0	4.0	-	-	-	-	-	-	-	-	0.000	0.001
NO15-L9B (L-11)	Center Floodgate at Pontchartrain Bch	Structure /Wall	Future	10.1	0.8	12.9	8.2	1.6	0.1	8.7	1.4	16.0	4.0	-	-	-	-	-	-	-	-	0.000	0.001
NO15-L9C (L-12)	East Floodgate at Pontchartrain Bch	Structure /Wall	Future	10.1	0.8	12.9	8.2	1.6	0.1	8.7	1.4	16.0	4.0	-	-	-	-	-	-	-	-	0.000	0.001
NO15-L	Canal Boulevard Ramp Levee	Levee	Existing	8.7	0.7	11.3	7.8	2.5	0.5	7.2	1.4	16.0	6.3	4	-	-	-	1	1	-	-	0.001	0.011
NO15-L	Canal Boulevard Ramp Levee	Levee	Future	10.2	0.7	12.8	8.6	3.1	0.5	7.6	1.4	19.0	7.8	4	-	-	-	1	1	-	-	0.003	0.016
NO16	Lakeshore Drive Near Rail Street Floodgate	Structure /Wall	Future	10.1	0.8	13.1	7.8	5.6	0.5	7.3	1.4	*18.0*	14.0	-	-	-	-	-	-	-	-	0.002	0.030
NO17	Leroy Johnson Drive	Structure /Wall	Future	10.1	0.8	12.8	7.1	4.0	0.3	7.0	1.3	16.5	10.0	-	-	-	-	-	-	-	-	0.009	0.038
NO20-FW1	Floodwall Under Leon C. Simon Drive Near Seabrook (West)	Structure /Wall	Future	10.1	0.8	12.8	7.1	4.6	0.4	7.0	1.3	16.5	11.6	-	-	-	-	-	-	-	-	0.024	0.08
NO20-FW2	I-wall Tie-in to Seabrook Gate (West)	I-wall	Future	10.1	0.8	12.8	7.1	4.6	0.4	7.0	1.3	17.0	11.6	-	-	-	-	-	-	-	-	0.016	0.06
NO20-G1 (W-42)	Boat Launch Gate Near Seabrook (West)	Gate	Future	10.1	0.8	12.8	7.1	4.6	0.4	7.0	1.3	16.5	11.6	-	-	-	-	-	-	-	-	0.024	0.08
NO20-G2 (W-43)	Norfolk Southern Railroad Gate Near Seabrook (West)	Gate	Future	10.1	0.8	12.8	7.1	4.6	0.4	7.0	1.3	16.5	11.6	-	-	-	-	-	-	-	-	0.024	0.08

Values in red italics include structural superiority.

Orleans Parish East Lakefront
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
NE01bw	Citrus Lakefront Breakwater	Structure /Wall	Existing	8.6	0.7	11.0	6.6	4.6	0.5	6.7	1.3	9.0	11.6	-	-	-	-	-	-	-	-	*	*
NE01	Citrus Lakefront Levee/I-wall	Levee/I-wall	Existing	8.6	0.7	11.0	*	2.0	0.5	6.7	1.3	14.5	*	2.5	10.0**	-	5.0	1	1	0.9	0.55	0.001	0.009
NE01bw	Citrus Lakefront Breakwater	Structure /Wall	Future	10.1	0.7	12.5	7.4	5.2	0.3	7.1	1.4	13.5	13.1	-	-	-	-	-	-	-	-	*	*
NE01	Citrus Lakefront Levee/I-wall	Levee/I-wall	Future	10.1	0.7	12.5	*	1.6	0.3	7.1	1.4	14.5	*	2.5	10.0**	-	5.0	1	1	0.9	0.55	0.001	0.01
NE02	New Orleans East Lakefront Levee	Levee	Existing	8.9	0.7	11.5	6.4	4.0	0.4	6.6	1.3	16.5	10.0	3	5.0	Horiz.	5.0	0.6	1	0.9	0.55	0.007	0.056
NE02	New Orleans East Lakefront Levee	Levee	Future	10.4	0.7	13.0	7.2	4.6	0.4	7.1	1.3	20.5	11.5	3	5.0	Horiz.	5.0	0.71	1	0.9	0.55	0.001	0.053
NE03-FW	New Orleans Lakefront Airport East T-wall	Structure /Wall	Future	9.9	0.7	12.3	7.2	3.2	0.3	7.4	1.3	15.5	8.0	-	-	-	-	-	-	-	-	0.003	0.015
NE03-LI	New Orleans Lakefront Airport East Lv/FW Combo	Structure /Wall	Future	9.9	0.7	12.3	7.2	2.4	0.2	7.4	1.3	15.5	6.0	2.5	9.0**	-	6.4	1	1	0.9	0.55	0.006	0.035
NE04-FW	New Orleans Lakefront Airport West Floodwall	Structure /Wall	Future	10.0	0.7	12.6	7.5	3.2	0.3	7.5	1.4	15.5	8.0	-	-	-	-	-	-	-	-	0.004	0.019
NE04-G	Downman Road Gate	Structure /Wall	Future	10.0	0.7	12.6	7.5	3.2	0.3	7.5	1.4	15.5	8.0	-	-	-	-	-	-	-	-	0.004	0.019
NE05	Lincoln Beach Floodwall	Structure /Wall	Future	10.1	0.7	12.5	7.1	2.4	0.2	7.6	1.3	15.5	6.0	-	-	-	-	-	-	-	-	0.000	0.003
NE06	Collins Pipeline Crossing Floodwall	Structure /Wall	Future	10.4	0.7	13.0	7.0	3.8	0.3	7.1	1.3	*17.5*	9.5	-	-	-	-	-	-	-	-	0.003	0.012
NE07	Citrus Pump Station (OP #10)	Structure /Wall	Future	10.0	0.7	12.4	6.6	1.6	0.5	7.1	1.3	14.5	11.5	-	-	-	-	-	-	-	-	0.020	0.069
NE08	Jahncke Pump Station (OP #14)	Structure /Wall	Future	10.0	0.7	12.4	7.1	1.6	0.5	7.1	1.3	14.5	11.5	-	-	-	-	-	-	-	-	0.020	0.069
NE09	St Charles Pump Station (OP #16)	Structure /Wall	Future	9.9	0.7	12.3	7.2	4.0	0.3	7.3	1.3	15.5	10.0	-	-	-	-	-	-	-	-	0.017	0.060
NE30-FW	Transition Reach from NE01-NE02 T-walls	Structure /Wall	Future	10.1	0.7	13.0	6.6	4.8	0.4	7.1	1.3	14.5-17.5	12.0	4	11.0**	-	5.0	1	1	0.9	0.55	0.010	0.087
NE31	South Point Transition Reach from NE02 to NE17 at I-10	Levee	Existing	9.0	0.7	11.7	5.7	3.6	0.4	5.8	1.2	16.5	9.0	4	9.0	10	4.0	0.70	1	1	1	0.009	0.058
NE31	South Point Transition Reach from NE02 to NE17 at I-10	Levee	Future	10.5	0.7	13.2	6.5	4.2	0.4	6.3	1.2	18.0	10.5	4	10.5	10	4.0	0.71	1	1	1	0.007	0.053

** Toe of I-Wall, Top of Levee

Values in red italics include structural superiority.

GIWW Sections (outside MRGO gate)
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
NE10-A	NE17 at I-10 to NE13 at Hwy 11 Lv	Levee	Existing	9.5	0.8	12.5	4.3	3.6	0.4	5.8	1.2	17.0	9.0	4	-	-	-	1	1	1	1	0.009	0.062
NE10-A	NE17 at I-10 to NE13 at Hwy 11 Lv	Levee	Future	11.0	0.8	14.0	5.1	4.2	0.4	6.3	1.2	18.0	10.5	4	11.0	8	4.0	0.7	1	1	1	0.005	0.045
NE10-B	NE13 at Hwy 11 - South to NE10-C Lv	Levee	Existing	9.7	0.9	12.8	4.5	3.9	0.4	5.8	1.2	17.0	9.8	4	10.0	6	4.0	0.83	1	1	1	0.005	0.049
NE10-B	NE13 at Hwy 11 - South to NE10-C Lv	Levee	Future	11.2	0.9	14.3	5.3	4.5	0.4	6.2	1.2	18.0	11.3	4	11.0	8	4.0	0.69	1	1	1	0.007	0.070
NE10-C	NE10-B South - to NE14 at Hwy 90	Levee	Existing	10.6	0.8	13.5	4.7	4.2	0.4	5.7	1.1	17.0	10.6	4	11.0	8	4.0	0.67	1	1	1	0.003	0.035
NE10-C	NE10-B South - to NE14 at Hwy 90	Levee	Future	12.1	0.8	15.0	5.5	4.8	0.4	6.1	1.1	19.0	12.0	4	11.0	8	4.0	0.72	1	1	1	0.010	0.080
NE11-A	Hwy 90 to CSX RR Levee	Levee	Existing	14.3	0.9	17.5	5.7	5.5	0.6	6.1	1.2	22.0	13.8	4	15.0	12	10.0	0.60	1	1	1	0.007	0.072
NE11-A	Hwy 90 to CSX RR Levee	Levee	Future	15.8	0.9	19.0	6.5	6.1	0.6	6.4	1.2	23.5	15.3	5	15.0	12	10.0	0.74	1	1	1	0.008	0.075
NE11-B	CSX RR to GIWW Levee	Levee	Existing	16.2	1.0	19.7	7.5	6.4	0.6	6.8	1.4	24.0	16.0	5	15.0	12	10.0	0.71	1	1	1	0.010	0.100
NE11-B	CSX RR to GIWW Levee	Levee	Future	17.7	1.0	21.2	8.3	7.0	0.6	7.1	1.4	27.5	17.5	5	15.0	12	10.0	0.71	1	1	1	0.005	0.050
NE12-A	New Orleans East Back Lv From P. S. 15 East Along GIWW	Levee	Existing	17.4	1.0	20.9	7.5	7.0	0.7	6.8	1.4	27.0	17.5	4	17.4	10	8.5	0.64	1	1	1	0.008	0.083
NE12-A	New Orleans East Back Lv From P. S. 15 East Along GIWW	Levee	Future	18.9	1.0	22.4	8.3	7.6	0.7	7.1	1.4	29.5	19.0	4	18.9	10	10.0	0.71	1	1	1	0.009	0.082
NE12-B-L	New Orleans East Back Lv From Gate to Pump Station 15	Levee	Existing	18.4	1.0	22.1	7.5	7.4	0.7	6.8	1.4	27.5	18.5	5	18.4	10	10.5	0.77	1	1	1	0.008	0.084
NE12-B-L	New Orleans East Back Lv From Gate to Pump Station 15	Levee	Future	19.9	1.0	23.6	8.3	8.0	0.7	7.1	1.4	30.0	20.0	5	19.9	10	12.0	0.77	1	1	1	0.008	0.080
NE12-B-FW	Tie-ins Between NE12-B and IHNC T-walls	T-wall	Future	19.9	1.0	23.6	8.3	8.0	0.7	7.1	1.4	32.0	20.0	15	12.0**	-	8.0	1	1	1	1	0.001	0.017
NE13	Hwy 11 Floodgate	Structure /Wall	Future	11.0	0.8	14.4	5.4	4.4	0.4	6.2	1.2	18.0	11.0	-	-	-	-	-	-	-	-	0.010	0.039
NE14	Hwy 90 Floodgate	Structure /Wall	Future	12.5	0.9	15.7	6.0	5.0	0.4	6.1	1.1	22.0	12.5	-	-	-	-	-	-	-	-	0.004	0.017
NE15-G	CSX RR Floodgate	Gate	Future	17.3	1.0	20.7	8.3	6.8	0.6	7.1	1.4	27.5	17.0	-	-	-	-	-	-	-	-	0.026	0.087
NE15-FW	CSX RR Floodwall	Floodwall	Future	17.3	1.0	20.7	8.3	6.8	0.6	7.1	1.4	27.5	17.0	-	-	-	-	-	-	-	-	0.026	0.087
NE16	New Orleans East P.S. 15 T-walls	Structure /Wall	Future	18.9	1.0	22.4	8.3	7.6	0.7	7.1	1.4	30.5	19.0	-	-	-	-	-	-	-	-	0.028	0.088
NE17	I-10 Lv Ramp Flank	Levee	Existing	9.5	0.8	12.5	3.2	3.2	0.3	5.3	1.1	16.5	8.5	4	10.0	10	1.0	0.8	1	1	1	0.001	0.017
NE17	I-10 Lv Ramp Flank	Levee	Future	11.0	0.8	14.0	4.0	4.0	0.3	5.9	1.1	18.0	10.0	4	10.0	10	1.0	0.8	1	1	1	0.007	0.061
NE32	Transition Lv Between NE11-B & NE12-A	Levee	Existing	16.2	1.0	19.7	7.5	6.5	0.7	6.8	1.4	27.0	16.3	4	16.5	10	8.5	0.64	1	1	1	0.003	0.031
NE32	Transition Lv Between NE11-B & NE12-A	Levee	Future	17.7	1.0	21.2	8.3	7.1	0.7	7.1	1.4	29.5	17.8	4	16.5	10	8.5	0.71	1	1	1	0.003	0.033

** Toe of I-Wall, Top of Levee. Values in red italics include structural superiority.

IHNC and GIWW Sections (with MRGO/GIWW and Seabrook closures)
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
GI01	Levee Reach GI02 to IHNC	Levee	Existing	6.3	0.8	7.5	-	2.3	0.2	2.8	0.6	11.5	6.3	3	-	-	-	1	1	-	-	0.01	0.07
GI01	Levee Reach GI02 to IHNC	Levee	Future	6.6	0.8	8.9	-	2.3	0.2	2.8	0.6	11.5	6.6	3	-	-	-	1	1	-	-	0.01	0.07
GI02	Paris Road to Levee Reach GI02	Levee	Existing	6.3	0.8	7.5	-	2.3	0.2	2.8	0.6	11.5	6.3	3	-	-	-	1	1	-	-	0.01	0.07
GI02	Paris Road to Levee Reach GI02	Levee	Future	6.6	0.8	8.9	-	2.3	0.2	2.8	0.6	11.5	6.6	3	-	-	-	1	1	-	-	0.01	0.07
GI03	Michoud Canal to Michoud Slip	Levee	Existing	6.3	0.8	7.5	-	3.0	0.3	3.5	0.7	13.5	6.3	3	-	-	-	1	1	-	-	0.01	0.05
GI03	Michoud Canal to Michoud Slip	Levee	Future	6.6	0.8	8.9	-	3.0	0.3	3.5	0.7	13.5	6.6	3	-	-	-	1	1	-	-	0.01	0.05
GI03-W	Floodwall Under Paris Road Bridge	Structure /Wall	Future	6.6	0.8	8.9	-	3.0	0.3	3.5	0.7	11.5	5.0	-	-	-	-	-	-	-	-	0.03	0.09
GI04	Michoud Canal and Slip	Structure /Wall	Future	6.6	0.8	8.9	-	3.0	0.3	3.5	0.7	11.5	6.6	-	-	-	-	-	-	-	-	0.03	0.09
GI05	Amid Pump Station (Pump Station #20)	Structure /Wall	Future	6.6	0.8	8.9	-	2.3	0.2	2.8	0.6	10.5	6.6	-	-	-	-	-	-	-	-	0.01	0.10
GI06	Elaine Pump Station	Structure /Wall	Future	6.6	0.8	8.9	-	2.3	0.2	2.8	0.6	10.5	6.6	-	-	-	-	-	-	-	-	0.01	0.10
GI07	Grant Pump Station	Structure /Wall	Future	6.6	0.8	8.9	-	2.3	0.2	2.8	0.6	10.5	6.6	-	-	-	-	-	-	-	-	0.01	0.10
GI08	Bienvenue Floodgate	Structure /Wall	Future	6.6	0.8	8.9	-	3.0	0.3	3.5	0.7	*13.5*	20.0	-	-	-	-	-	-	-	-	0.03	0.09
IH01-WN	IHNC South of I-10 to N of Florida Av	Structure /Wall	Future	6.6	0.8	8.9	-	2.3	0.2	2.8	0.6	10.5	6.6	-	-	-	-	-	-	-	-	0.01	0.10
IH01-WS	S of Florida Av to IHNC Lock	Structure /Wall	Future	6.6	0.8	8.9	-	1.7	0.2	2.3	0.5	10.0	6.6	-	-	-	-	-	-	-	-	0.01	0.06
IH02-W	IHNC North of I-10	Structure /Wall	Future	6.6	0.8	8.9	-	1.7	0.2	2.3	0.5	10.0	6.6	-	-	-	-	-	-	-	-	0.01	0.06
IH03	IHNC Levee South From I-10	Levee	Existing	6.3	0.8	7.5	-	2.3	0.2	2.8	0.6	11.5	6.3	3	-	-	-	1	1	-	-	0.01	0.05
IH03	IHNC Levee South From I-10	Levee	Future	6.6	0.8	8.9	-	2.3	0.2	2.8	0.6	11.5	6.6	3	-	-	-	1	1	-	-	0.01	0.05
IH04-W	IHNC Lock to Pump Station (Pump Station #5)	Structure /Wall	Future	6.6	0.8	8.9	-	1.7	0.2	2.3	0.5	10.0	6.6	-	-	-	-	-	-	-	-	0.01	0.06
IH05-W	Dwyer Pump Station	Structure /Wall	Future	6.6	0.8	8.9	-	1.7	0.2	2.3	0.5	10.0	6.6	-	-	-	-	-	-	-	-	0.01	0.06
IH10	Orleans Pump Station #5 to Pump Station #19	Structure /Wall	Future	6.6	0.8	8.9	-	1.7	0.2	2.3	0.5	*12.0*	5.0	-	-	-	-	-	-	-	-	0.01	0.06

Values in red italics include structural superiority.

MRGO- GIWW and Seabrook Closures Sections 1% Overtopping Design Criteria																							
				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
MRGO-CS	MRGO Closure Floodwall Crenel/Merlon (South Barrier Wall)	Structure /Wall	Future	20.3	1.0	24.0	7.8	7.8	0.8	8.1	1.6	25.0/26.0	41.3	-	-	-	-	-	-	-	-	See note below	See note below
GIWW-FW	GIWW Tie-in T-walls to Levee (North T-Wall)	Structure /Wall	Future	20.3	1.0	24.0	7.8	7.8	0.8	8.1	1.6	26.0	17.3-20.3	-	-	-	-	-	-	-	-	See note below	See note below
BVN-FW	Bayou Bienvenue Braced Floodwall was Levee-A1 Crenel/Merlon (South Barrier Wall)	Structure /Wall	Future	20.3	1.0	24.0	7.8	7.8	0.8	8.1	1.6	25.0/26.0	35.3	-	-	-	-	-	-	-	-	See note below	See note below
MRGO-FW	Tie-in T-walls and at MRGO Levee (South T-Wall)	Structure /Wall	Future	20.3	1.0	24.0	7.8	7.8	0.8	8.1	1.6	26.0	16.3-20.3	-	-	-	-	-	-	-	-	See note below	See note below
Lake Borgne-FW	Lake Borgne Floodwall Crenel/Merlon (North Barrier Wall)	Structure /Wall	Future	20.3	1.0	24.0	7.8	7.8	0.8	8.1	1.6	25.0/26.0	41.3	-	-	-	-	-	-	-	-	See note below	See note below
GIWW-G	Navigable Floodgate at GIWW	Closure Gates	Future	20.3	1.0	24.0	7.8	7.8	0.8	8.1	1.6	26.0	37.8	-	-	-	-	-	-	-	-	See note below	See note below
GIWW-B	GIWW Concrete Swing Barge	Structure /Wall	Future	20.3	1.0	24.0	7.8	7.8	0.8	8.1	1.6	26.0	37.8	-	-	-	-	-	-	-	-	See note below	See note below
BVN-G	Navigable Floodgate at Bayou Bienvenue	Closure Gates	Future	20.3	1.0	24.0	7.8	7.8	0.8	8.1	1.6	26.0	28.3	-	-	-	-	-	-	-	-	See note below	See note below
GIWW-M	GIWW Monoliths	Structure /Wall	Future	20.3	1.0	24.0	7.8	7.8	0.8	8.1	1.6	26.0	50.3	-	-	-	-	-	-	-	-	See note below	See note below
SBRK-G	Closure Gate at Seabrook	Closure Gate	Future	10.8	0.8	13.9	8.0	6.0	0.5	6.1	1.1	16.0	15.0	-	-	-	-	-	-	-	-	0.208	0.411
SBRK-FW	Seabrook Closure Complex East and West Tie-in Walls	Structure /Wall	Future	10.8	0.8	13.9	8.0	3.2	0.3	6.4	1.2	16.0	8.0	-	-	-	-	-	-	-	-	0.004	0.015

Note: Wave Overtopping for the MRGO Surge Barrier is explained in detail in the report "Hydraulic Storm Surge & Wave Design, Inner Harbor Navigation Canal Hurricane Protection Project", AECOM, Inc., prepared for INCA/Gerwick JV, August 2009 and further modified in the IHNC System Analysis Report 2012.

Values in red italics include structural superiority.

St Bernard
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
SB11	MRGO Levee - IHNC Surge Barrier Tie-in 1.7 Miles to SB12	Levee/Floodwall Combo	Future	19.9	1.0	23.6	8.0	8.0	0.7	7.1	1.4	32.0	19.9	4	17.0**	20	4.0	1	1	1	1	0.0010	0.0170
SB12	MRGO Levee - SB11 0.9 Miles to SB13	Levee/Floodwall Combo	Future	18.8	1.1	22.6	7.5	7.5	0.7	7.1	1.4	30.0	18.8	4.5	17.0**	-	4.0	1	1	1	1	0.0004	0.0100
SB13	MRGO Levee -Bayou Bienvenue to Bayou Dupre	Levee/Floodwall Combo	Future	17.9	1.1	21.7	7.2	7.2	0.7	7.1	1.4	29.0	17.9	4.8	18.0**	-	4.0	1	1	1	1	0.0001	0.0030
SB15	MRGO Levee - Bayou Dupre to Hwy 46	Levee/Floodwall Combo	Future	17.1	1.2	21.4	6.8	6.8	0.6	7.2	1.4	28.0	17.1	4	17.0**	-	4.0	1	1	1	1	0.0001	0.0014
SB16	Caernarvon to Verret	Levee/Floodwall Combo	Future	18.8	1.0	22.4	6.0	6.0	0.5	7.0	1.3	32.0	18.8	4	17.5**	-	4.0	1	1	1	1	0.0062	0.0600
SB17	Caernarvon to Verret	Levee/Floodwall Combo	Future	19.4	1.2	23.5	6.0	6.0	0.5	7.0	1.3	32.0	17.3	4	17.0**	-	4.0	1	1	1	1	0.0094	0.0900
SB19-G	Bayou Dupre Control Structure	Structure /Wall	Future	17.3	1.0	21.0	7.5	7.5	0.6	8.3	1.6	*31.0*	29.8	-	-	-	-	-	-	-	-	0.0010	0.0040
SB19-FW	Bayou Dupre T-wall Tie-ins	Structure /Wall	Future	17.3	1.0	21.0	7.5	7.5	0.6	8.3	1.6	29.0	18.5	-	-	-	-	-	-	-	-	0.0210	0.0700
SB20	St. Mary Pump Station (Pump Station #8)	Structure /Wall	Future	18.5	1.0	21.8	6.0	6.0	0.5	7.0	1.3	32.0	19.5	-	-	-	-	-	-	-	-	0.0020	0.0090
SB21-TR	Transition Reach	Floodwall /Levee Combo	Future	19.5	1.2	23.6	4.3	4.3	0.4	7.2	1.3	26.0-32.0	19.5	4	17.5**	-	4.0	1	1	1	1	0.0100	0.0640
SB21-FW	Caernarvon Canal Floodwall (East of Canal)	T-wall	Future	19.5	1.2	23.6	4.3	4.3	0.4	7.2	1.3	26.0	19.5	-	-	-	-	-	-	-	-	0.0160	0.0770
SB21-G1	Caernarvon Canal Sector Gate	Gate	Future	19.5	1.2	23.6	4.3	4.3	0.4	7.2	1.3	26.0	19.5	-	-	-	-	-	-	-	-	0.0160	0.0770
SB21-G2	Caernarvon Canal Hwy 39 Gate and Railroad Gate	Gate	Future	19.5	1.2	23.6	4.3	4.3	0.4	7.2	1.3	26.0	17.9	-	-	-	-	-	-	-	-	0.0160	0.0770
SB21-MRL tie in	Tie-in to MRL	T-wall	Future	19.5	1.2	23.6	3.0	3.0	0.3	7.8	1.5	24.0	12.0	-	-	-	-	-	-	-	-	0.0109	0.0818
SB161-LT	Bayou Road to Hwy 46 Levee/Floodwall Combo	Levee/Floodwall Combo	Future	17.9	1.0	21.4	6.4	6.4	0.6	7.2	1.4	30.0	17.9	4	Varies**	-	4.0	1	1	1	1	0.0002	0.0050
SB161-G1	Caernarvon to Verret Hwy 46 Floodgate	Gate	Future	17.9	1.0	21.4	6.4	6.4	0.6	7.2	1.4	30.0	17.9	-	-	-	-	-	-	-	-	0.0052	0.0220
SB161-G2	Caernarvon to Verret Bayou Road Floodgate	Gate	Future	17.9	1.0	21.4	6.4	6.4	0.6	7.2	1.4	30.0	17.9	-	-	-	-	-	-	-	-	0.0052	0.0220

** Toe of Base Slab of the T-Wall, Top of Levee

Values in red italics include structural superiority.

Westbank Sections (Lake Cataouatche Reach)
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
WB01	US 90 to the Bayou Segnette State Park	Levee	Existing	6.5	0.7	9.0	2.1	2.1	0.2	5.5	1.1	11.5	6.5	4	3.5	14	1.0	0.92	1	1	1	0.004	0.029
WB01	US 90 to the Bayou Segnette State Park	Levee	Future	8.5	0.7	11.0	3.1	3.1	0.2	6.7	1.1	15.5	8.5	4	8.5	8	3.8	0.73	1	1	1	0.003	0.023
WB02	Lake Cataouatche Pump Station #1 and #2	Structure /Wall	Future	8.5	0.7	11.0	3.1	3.1	0.2	6.7	1.1	*15.5*	8.5	-	-	-	-	-	-	-	-	0.001	0.003
WB05	Bayou Segnette Pump Station #1 and #2	Structure /Wall	Future	8.5	0.7	11.1	2.4	2.4	0.1	5.6	0.9	*16.0*	8.5	-	-	-	-	-	-	-	-	0.000	0.000
WB31-FW1	Western Tie-in Monoliths at US90	Wall	Future	8.5	0.7	11.0	3.1	3.1	0.2	6.7	1.1	15.5	8.5	-	-	-	-	-	-	-	-	0.0006	0.0031
WB31-FW2	Railroad T-walls Union Pacific and BN&SF Railroads	Wall	Future	8.5	0.7	10.9	2.6	2.6	0.2	6.9	1.1	13.0	7.5	-	-	-	-	-	-	-	-	0.004	0.020
WB31-FW3	Bayou Verret Navigable Floodgates T-wall Section 1	T-wall	Future	8.5	0.7	11.0	3.1	3.1	0.2	6.7	1.1	15.5	8.5	-	-	-	-	-	-	-	-	0.000	0.0004
WB31-FW4	Bayou Verret Navigable Floodgates T-wall Section 2	T-wall	Future	8.5	0.7	11.0	3.1	3.1	0.2	6.7	1.1	15.5	8.5	-	-	-	-	-	-	-	-	0.000	0.0004
WB31-G1	BN&SF Railroad Swing Gate	Gate	Future	8.5	0.7	10.9	2.6	2.6	0.3	6.3	0.7	*15.0*	7.5	-	-	-	-	-	-	-	-	0.0002	0.001
WB31-G2	Union Pacific Railroad Swing Gate	Gate	Future	8.5	0.7	10.9	2.6	2.6	0.3	6.3	0.7	*15.0*	7.5	-	-	-	-	-	-	-	-	0.0002	0.001
WB31-G3	Bayou Verret Navigable Floodgate	Gate	Future	8.5	0.7	11.0	3.1	3.1	0.2	6.7	1.1	*16.0*	8.5	-	-	-	-	-	-	-	-	0.001	0.002
WB31-L East-West	Mississippi River to US 90 Levees	Levee	Existing	6.5	0.7	8.9	1.6	1.6	0.2	5.4	1.1	9.0	5.5	4	6.5	14	4.0	0.6	1	1	1	0.008	0.092
WB31-L East-West	Mississippi River to US 90 Levees	Levee	Future	8.5	0.7	10.9	2.6	2.6	0.2	6.9	1.1	13.0	7.5	4	8.5	14	4.5	0.6	1	1	1	0.008	0.057
WB31-L North-South	Mississippi River to US 90 Levees	Levee	Existing	6.5	0.7	8.9	1.6	1.5	0.2	2.5	0.5	9.0	5.5	3	-	-	-	-	1	1	1	0.006	0.075
WB31-L North-South	Mississippi River to US 90 Levees	Levee	Future	8.5	0.7	10.9	2.6	2.5	0.2	3.2	0.5	13.0	7.5	3	-	-	-	-	1	1	1	0.008	0.052
WB43	Bayou Segnette State Park Floodwall	Structure /Wall	Future	8.5	0.7	11.1	2.4	2.4	0.1	5.6	0.9	14.0	8.5	-	-	-	-	-	-	-	-	0.000	0.002

Values in red italics include structural superiority.

Westbank Sections (Westwego to Harvey Reach)
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
WB07	New Westwego Pump Station #2	Pump Station	Future	8.5	0.7	11.1	2.4	2.4	0.1	5.6	0.9	*16.0*	8.5	-	-	-	-	-	-	-	-	0.000	0.000
WB08-B	New Westwego Pump Station #2 to Orleans Village Levee	Levee	Existing	6.5	0.7	9.1	1.4	1.4	0.1	4.3	0.9	10.5	5.5	3	-	-	-	1	1	-	-	0.001	0.010
WB08-B	New Westwego Pump Station #2 to Orleans Village Levee	Levee	Future	8.5	0.7	11.1	2.4	2.4	0.1	5.6	0.9	14.0	7.5	4	-	-	-	1	1	-	-	0.008	0.036
WB10-FW1	Westminster Pump Station Floodwalls	Pump Station	Future	8.5	0.7	11.1	2.4	2.4	0.1	5.6	0.9	*16.0*	8.5	-	-	-	-	-	-	-	-	0.000	0.000
WB10-FW2	Westminster Pump Station Tie-in Walls	Pump Station	Future	8.5	0.7	11.1	2.4	2.4	0.1	5.6	0.9	14.0	8.5	-	-	-	-	-	-	-	-	0.008	0.035
WB11	Ames to Mt. Kennedy Floodwall	Floodwall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	14.0	9.3	-	-	-	-	-	-	-	-	0.001	0.008
WB11-P1	Ames Pump Station	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.000	0.000
WB11-P2	Mt. Kennedy Pump Station	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.000	0.000
WB12	Estelle Pump Station #1 (Old Estelle)	Pump Station	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.000	0.000
WB32	Highway 45 to Highway 3134 Floodwall	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	14.0	9.3	-	-	-	-	-	-	-	-	0.001	0.008
WB41	Highway 3134 to Old Estelle Pump Stations Levee	Levee	Existing	7.3	0.9	10.4	1.3	1.3	0.1	3.7	0.7	10.5	7.3	3	-	-	-	1	1	-	-	0.003	0.034
WB41	Highway 3134 to Old Estelle Pump Stations Levee	Levee	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	14.0	9.3	4	-	-	-	1	1	-	-	0.010	0.061
WB42-FW1	Gulf South 1 Utility Crossing	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.010	0.063
WB42-FW2	Gulf South 2 Utility Crossing	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.010	0.063
WB42-FW3	Chevron Pipeline Crossing	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.010	0.063
WB42-FW4	Enterprise Pipeline Crossing	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.010	0.063
WB42-L	Orleans Village to Highway 45 Levee	Levee	Existing	7.3	0.9	10.4	1.3	1.3	0.1	3.7	0.7	10.5	7.3	3	-	-	-	1	1	-	-	0.003	0.035
WB42-L	Orleans Village to Highway 45 Levee	Levee	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	14.0	9.3	4	-	-	-	1	1	-	-	0.010	0.063
WB43-A	Bayou Segnette Complex to Company Canal Levee	Levee	**	3.0	0.5	**	**	1.4	0.1	2.5	0.5	5.0	5.0	4	-	-	-	1	1	-	-	0.004	0.049
WB43-B	Company Canal & Westwego Floodwall	Structure /Wall	**	3.0	0.5	**	**	1.4	0.1	2.5	0.5	4.0	4.4	-	-	-	-	-	-	-	-	0.017	0.091
WB43-C	Old Westwego Pump Station	Structure /Wall	**	3.0	0.5	**	**	1.4	0.1	2.5	0.5	5.0	5.0	-	-	-	-	-	-	-	-	0.007	0.04
WB43-D-CPLX	Bayou Segnette Complex	Complex	Future	8.5	0.7	11.1	2.4	2.4	0.1	5.6	0.9	16.0	8.5	-	-	-	-	-	-	-	-	0.000	0.000
WB43-D-FW	Bayou Segnette Complex Floodwall	Floodwall	Future	8.5	0.7	11.1	2.4	2.4	0.1	5.6	0.9	16.0	8.5	-	-	-	-	-	-	-	-	0.000	0.000
WB43-D-L	Bayou Segnette Complex Levee	Levee	Existing	6.5	0.7	9.1	1.4	1.4	0.1	4.3	0.9	10.5	5.5	3	-	-	-	1	1	-	-	0.001	0.010

Westbank Sections (Westwego to Harvey Reach) 1% Overtopping Design Criteria																							
				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
WB43-D-L	Bayou Segnette Complex Levee	Levee	Future	8.5	0.7	11.1	2.4	2.4	0.1	5.6	0.9	14.0	7.5	4	-	-	-	1	1	-	-	0.008	0.036
WB44-FW1	Old Estelle Pump Station to Robinson Point	Floodwall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	14.0	9.3	-	-	-	-	-	-	-	-	0.001	0.008
WB44-FW2	Inlet Floodwall	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.001	0.004

**The existing and future conditions are the same because the still water levels will be controlled by the actions taken at the Bayou Segnette Complex during storm events.

*The existing and future conditions are the same because the still water levels will be controlled by the actions taken at the WCC during storm events.

Values in red italics include structural superiority.

Westbank Sections (East of Harvey Canal Reach)
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
WB14-FW1	Robinson Point to Estelle Pump Station #2 West Floodwall	Floodwall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB14-FW2	Hero Pump Station to Algiers Canal Floodwall	Floodwall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB14-L	Estelle Pump Station #2 to Lapalco Sector Gate West Levee	Levee	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	3	-	-	-	1	1	-	-	0.0001	0.002
WB15-FW1	New Estelle Pump Station and Fronting Protection	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	*9.5*	5.3	-	-	-	-	-	-	-	-	0.000	0.0003
WB15-FW2	New Estelle Pump Station Tie-In Walls	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	*9.5*	5.3	-	-	-	-	-	-	-	-	0.000	0.0003
WB16-P	Cousins Pump Station #1, #2, and #3 (on Harvey Canal) Fronting Protection	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	*9.5*	5.3	-	-	-	-	-	-	-	-	0.000	0.0003
WB16-FW	Cousins Pump Station #1, #2, and #3 (on Harvey Canal) Floodwall	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB19	Transition Point to Hero Canal to Oakville	Levee	Existing	7.3	0.9	10.4	1.3	1.3	0.1	3.7	0.7	10.5	7.3	4	-	-	-	1	1	-	-	0.001	0.025
WB19	Transition Point to Hero Canal to Oakville	Levee	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	14.0	9.3	4	-	-	-	1	1	-	-	0.010	0.063
WB19-A1	Hero Canal Area West of Pump Station	Levee	Existing	7.3	0.9	10.4	1.3	1.3	0.1	3.7	0.7	10.5	7.3	4	-	-	-	1	1	-	-	0.001	0.025
WB19-A1	Hero Canal Area West of Pump Station	Levee	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	14.0	9.3	4	-	-	-	1	1	-	-	0.010	0.063
WB19-A2	Hero Canal Area East of Pump Station	Levee	Existing	7.3	0.9	10.4	1.3	1.3	0.1	3.7	0.7	10.5	7.3	4	-	-	-	1	1	-	-	0.001	0.025
WB19-A2	Hero Canal Area East of Pump Station	Levee	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	14.0	9.3	4	-	-	-	1	1	-	-	0.010	0.063
WB19-A-P	Fronting Protection for Pump Station near Sector Gate	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.001	0.008
WB19-AW-FW	Eastern Tie-in Floodwalls	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	14.0	9.3	-	-	-	-	-	-	-	-	0.001	0.008
WB19-AW-G1	Hwy 23 Northbound & Southbound T-walls	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	14.0	9.3	-	-	-	-	-	-	-	-	0.001	0.008
WB19-AW-G2	Eastern Tie-in Railroad Gate & Hwy Gate	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	14.0	9.3	-	-	-	-	-	-	-	-	0.001	0.008
WB19-FW	Hero Canal Sector Gate Floodwalls	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.0002	0.0018
WB19-G	Hero Canal Sector Gate	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.0002	0.0018
WB19-P	Oakville Pump Station Fronting Protection	Pump Station	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	15.5	9.3	-	-	-	-	-	-	-	-	0.000	0.001
WB23-P1	Belle Chase Pump Station #1	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	*9.5*	5.3	-	-	-	-	-	-	-	-	0.0000	0.0003
WB23-P2	Belle Chase Pump Station #2	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	*9.5*	5.3	-	-	-	-	-	-	-	-	0.0000	0.0003

Westbank Sections (East of Harvey Canal Reach)
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
WB23-P3	Whitney Barataria Pump Station	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	*9.5*	5.3	-	-	-	-	-	-	-	-	0.0000	0.0003
WB24	Planters Pump Station	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	*9.5*	5.3	-	-	-	-	-	-	-	-	0.000	0.0003
WB27	Hero Pump Station (on Harvey Canal)	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	*9.5*	5.3	-	-	-	-	-	-	-	-	0.000	0.0003
WB30-FW1	Algiers Canal West Floodwall near Belle Chase	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB30-FW2	Algiers Canal East Floodwall near Belle Chase	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB30-G1	Algiers Canal West Bank Floodgates	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB30-G2	Algiers Canal West Swing Gate near Belle Chase	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB30-G3	Algiers Canal West Gate at Belle Chase Tunnel	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB30-G4	Algiers Canal Railroad Gate West near Belle Chase	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB30-G5	Algiers Canal East Gate at Tunnel Road near Belle Chase	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB30-G6	Algiers Canal East Gate at Belle Chase Tunnel	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB30-G7	Algiers Canal East Gate near Belle Chase	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB30-G8	Algiers Canal Railroad Gate (east) near Belle Chase	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB30-L1	Algiers Canal West Bank Levee	Levee	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	3	-	-	-	1	1	-	-	0.0001	0.002
WB30-L2	Algiers Canal East Bank Levee	Levee	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	3	-	-	-	1	1	-	-	0.0001	0.002
WB30-W-P1	New Orleans S&WB Pump Stations #11 (also known as OP #11)	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	*9.5*	5.3	-	-	-	-	-	-	-	-	0.000	0.0003
WB30-W-P2	New Orleans S&WB Pump Stations #13	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	*9.5*	5.3	-	-	-	-	-	-	-	-	0.000	0.0003
WB40	Harvey Canal Floodwall	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002
WB40-L	Sector Gate at Lapalco Overpass on Harvey Canal	Structure /Wall	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	-	-	-	-	-	-	-	-	0.0004	0.002

*The existing and future conditions are the same because the still water levels will be controlled by the actions taken at the WCC during storm events.

Values in red italics include structural superiority.

West bank Reaches (Western Closure Complex)
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
WB90-CS	Control Structure at Estelle & Harvey Canals	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.0	0.0004
WB90-FW1	WCC 404c Floodwall	Structure /Wall	Future	9.3	0.9	12.4	2.3	2.3	0.1	4.9	0.7	*16.0*	9.3	-	-	-	-	-	-	-	-	0.0	0.0004
WB90-FW2	Closure Wall Between 404c Floodwall & WCC Sector Gate	Structure /Wall	Future	9.8	0.9	12.9	2.3	2.3	0.1	4.9	0.7	*16.0*	9.8	-	-	-	-	-	-	-	-	0.0001	0.001
WB90-FW3	WCC Discharge T-Wall (East)	Structure /Wall	Future	9.8	0.9	12.9	2.3	2.3	0.1	4.9	0.7	*16.0*	9.8	-	-	-	-	-	-	-	-	0.0001	0.001
WB90-G1	WCC Sector Gate	Structure /Wall	Future	9.8	0.9	12.9	2.3	2.3	0.1	4.9	0.7	*16.0*	9.8	-	-	-	-	-	-	-	-	0.0001	0.001
WB90 - G2	WCC Sluice Gate	Structure /Wall	Future	9.8	0.9	12.9	2.3	2.3	0.1	4.9	0.7	*16.0*	9.8	-	-	-	-	-	-	-	-	0.0001	0.001
WB90-P	WCC Pump Station	Structure /Wall	Future	9.8	0.9	12.9	2.3	2.3	0.1	4.9	0.7	*16.0*	9.8	-	-	-	-	-	-	-	-	0.0001	0.001
WB90-L1	WCC Intake Levee	Levee	*	5.3	0.4	*	*	1.5	0.2	2.5	0.5	8.5	5.3	3	-	-	-	-	1	1	-	0.001	0.013
WB90-L2	WCC Discharge Levee	Levee	Existing	7.8	0.9	10.9	1.3	1.3	0.1	3.7	0.7	11.0	7.8	3	-	-	-	-	1	1	-	0.003	0.035
WB90-L2	WCC Discharge Levee	Levee	Future	9.8	0.9	12.9	2.3	2.3	0.1	4.9	0.7	15.5	9.8	3	-	-	-	-	1	1	-	0.006	0.032

*The existing and future conditions are the same because the still water levels will be controlled by the actions taken at the WCC during storm events.

Values in red italics include structural superiority.

West Bank (RM 70W to RM 118W)
1% Hydraulic Boundary Conditions

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
70W-L	Plaquemines WB	Levee	Existing	15.4	1.0	18.2	2.5		0.3	4.0	0.8	21.0	10.0	3								0.007	0.055
70W-L	Plaquemines WB	Levee	Future	17.8	0.8	20.7	3.0		0.3	4.0	0.8	24.5	12.4	3								0.006	0.044
71W-L	Plaquemines WB	Levee	Existing	15.4	1.0	18.1	2.5		0.3	4.0	0.8	21.0	9.2	3								0.007	0.053
71W-L	Plaquemines WB	Levee	Future	17.8	0.8	20.8	3.0		0.3	4.0	0.8	24.5	11.7	3								0.006	0.048
72W-L	Plaquemines WB	Levee	Existing	15.3	1.0	18.1	2.5		0.3	4.0	0.8	21.0	7.9	3								0.006	0.052
72W-L	Plaquemines WB	Levee	Future	17.8	0.8	20.8	3.0		0.3	4.0	0.8	24.5	10.4	3								0.006	0.047
73W-L	Plaquemines WB	Levee	Existing	15.3	1.0	18.1	2.5		0.3	4.0	0.8	21.0	7.2	3								0.006	0.052
73W-L	Plaquemines WB	Levee	Future	17.9	0.8	20.8	3.0		0.3	4.0	0.8	24.5	9.8	3								0.006	0.049
74W-L	Plaquemines WB	Levee	Existing	15.2	1.0	18.1	2.5		0.3	4.0	0.8	20.5	8.5	3								0.006	0.048
74W-L	Plaquemines WB	Levee	Future	17.9	0.8	20.8	3.0		0.3	4.0	0.8	24.5	11.1	3								0.007	0.049
75W-L	Plaquemines WB	Levee	Existing	15.2	1.0	18.1	2.5		0.3	4.0	0.8	20.5	8.9	3								0.010	0.074
75W-L	Plaquemines WB	Levee	Future	17.8	0.8	20.9	3.0		0.3	4.0	0.8	24.5	11.6	3								0.006	0.048
76W-L	Plaquemines WB	Levee	Existing	15.1	1.0	18.0	2.5		0.3	4.0	0.8	20.5	6.5	3								0.009	0.074
76W-L	Plaquemines WB	Levee	Future	17.8	0.8	20.9	3.0		0.3	4.0	0.8	24.5	9.2	3								0.006	0.048
77W-L	Plaquemines WB	Levee	Existing	15.1	1.1	18.0	2.5		0.3	4.0	0.8	20.5	7.3	3								0.009	0.075
77W-L	Plaquemines WB	Levee	Future	17.8	0.8	20.9	3.0		0.3	4.0	0.8	24.5	10.0	3								0.007	0.047
78W-L	Plaquemines WB	Levee	Existing	15.1	1.1	18.0	2.5		0.3	4.0	0.8	20.5	6.6	3								0.009	0.072
78W-L	Plaquemines WB	Levee	Future	17.8	0.8	20.9	3.0		0.3	4.0	0.8	24.5	9.4	3								0.006	0.048
79W-L	Plaquemines WB	Levee	Existing	15.0	1.1	18.0	2.5		0.3	4.0	0.8	20.5	6.4	3								0.008	0.068
79W-L	Plaquemines WB	Levee	Future	17.8	0.8	20.9	3.0		0.3	4.0	0.8	24.5	9.2	3								0.006	0.047
80W-L	Plaquemines WB	Levee	Existing	15.0	1.1	17.9	1.5		0.2	2.5	0.5	20.0	6.4	3								0.000	0.001
80W-L	Plaquemines WB	Levee	Future	17.8	0.8	20.8	2.0		0.2	3.0	0.6	24.0	9.2	3								0.000	0.002
81W-L	Plaquemines WB	Levee	Existing	15.0	1.0	17.9	1.5		0.2	2.5	0.5	20.0	5.7	3								0.000	0.001
81W-L	Plaquemines WB	Levee	Future	17.8	0.8	20.8	2.0		0.2	3.0	0.6	24.0	8.5	3								0.000	0.002
82W-L	Orleans WB	Levee	Existing	15.0	1.0	17.9	2.3		0.2	3.8	0.8	20.0	5.5	3								0.006	0.055
82W-L	Orleans WB	Levee	Future	17.8	0.8	20.8	2.8		0.3	4.0	0.8	24.0	8.3	3								0.006	0.044
83W-L	Orleans WB	Levee	Existing	15.0	1.0	17.9	2.3		0.2	3.8	0.8	20.0	6.1	3								0.006	0.054
83W-L	Orleans WB	Levee	Future	17.7	0.8	20.7	2.8		0.3	4.0	0.8	24.0	8.9	3								0.006	0.042

West Bank (RM 70W to RM 118W) 1% Hydraulic Boundary Conditions																							
				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
84W-L	Orleans WB	Levee	Existing	15.0	1.0	17.9	2.3		0.2	3.8	0.8	20.0	7.7	3								0.006	0.055
84W-L	Orleans WB	Levee	Future	17.7	0.8	20.8	2.8		0.3	4.0	0.8	24.0	10.5	3								0.006	0.042
85W-L	Orleans WB	Levee	Existing	15.0	1.0	17.9	2.3		0.2	3.8	0.8	20.0	6.1	3								0.006	0.053
85W-L	Orleans WB	Levee	Future	17.7	0.8	20.8	2.8		0.3	4.0	0.8	24.0	8.9	3								0.006	0.043
86W-L	Orleans WB	Levee	Existing	15.0	1.0	17.9	2.3		0.2	3.8	0.8	20.0	5.9	3								0.006	0.057
86W-L	Orleans WB	Levee	Future	17.8	0.8	20.8	2.8		0.3	4.0	0.8	24.0	8.6	3								0.006	0.045
87W-L	Orleans WB	Levee	Existing	15.0	1.0	18.0	2.3		0.2	3.8	0.8	20.0	6.5	3								0.006	0.056
87W-L	Orleans WB	Levee	Future	17.8	0.8	20.9	2.8		0.3	4.0	0.8	24.0	9.3	3								0.006	0.046
88W-L	Orleans WB	Levee	Existing	15.1	1.0	18.0	2.3		0.2	3.8	0.8	20.0	7.4	3								0.007	0.061
88W-LF	Orleans WB	Structure/ Wall	Future	17.8	0.8	20.9	2.8		0.3	4.0	0.8	24.0	10.2	3								0.001	0.004
88W-L	Orleans WB	Levee	Future	17.8	0.8	20.9	2.8		0.3	4.0	0.8	24.0	10.2	3								0.007	0.049
89W-L	Orleans WB	Levee	Existing	15.1	1.0	18.1	2.3		0.2	3.8	0.8	20.0	6.0	3								0.007	0.063
89W-L	Orleans WB	Levee	Future	17.9	0.8	21.0	2.8		0.3	4.0	0.8	24.0	8.8	3								0.006	0.048
90W-L	Orleans WB	Levee	Existing	15.1	1.0	18.1	2.3		0.2	3.8	0.8	20.0	6.2	3								0.007	0.063
90W-L	Orleans WB	Levee	Future	17.9	0.9	21.1	2.8		0.3	4.0	0.8	24.0	9.0	3								0.007	0.051
91W-L	Orleans WB	Levee	Existing	15.1	1.0	18.1	2.3		0.2	3.8	0.8	20.0	5.7	3								0.007	0.063
91W-L	Orleans WB	Levee	Future	17.9	0.9	21.1	2.8		0.3	4.0	0.8	24.0	8.5	3								0.007	0.052
92W-L	Orleans WB	Levee	Existing	15.2	1.1	18.2	2.3		0.2	3.8	0.8	20.0	6.1	3								0.007	0.068
92W-L	Orleans WB	Levee	Future	17.9	0.9	21.2	2.8		0.3	4.0	0.8	24.0	8.8	3								0.007	0.053
93W-L	Orleans WB	Levee	Existing	15.2	1.1	18.2	2.3		0.2	3.8	0.8	20.0	5.0	3								0.007	0.071
93W-L	Orleans WB	Levee	Future	17.9	0.9	21.3	2.8		0.3	4.0	0.8	24.0	7.7	3								0.007	0.054
94W-L	Orleans WB	Levee	Existing	15.2	1.1	18.3	2.3		0.2	3.8	0.8	20.0	4.8	3								0.008	0.075
94W-L	Orleans WB	Levee	Future	18.0	0.9	21.3	2.8		0.3	4.0	0.8	24.0	7.5	3								0.008	0.055
95W-L	Orleans WB	Levee	Existing	15.3	1.1	18.4	2.3		0.2	3.8	0.8	20.0	3.4	3								0.009	0.076
95W-LF	Orleans WB	Structure/ Wall	Future	18.0	0.9	21.4	2.8		0.3	4.0	0.8	24.0	6.1	3								0.001	0.006
95W-L	Orleans WB	Levee	Future	18.0	0.9	21.4	2.8		0.3	4.0	0.8	24.0	6.1	3								0.008	0.058
96W-L	Jefferson WB	Levee	Existing	15.3	1.1	18.4	1.5		0.2	2.5	0.5	19.0	3.9	3								0.000	0.016
96W-LF	Jefferson WB	Structure/ Wall	Future	18.0	0.9	21.4	1.5		0.2	2.5	0.5	22.5	6.6	3								0.000	0.000

West Bank (RM 70W to RM 118W) 1% Hydraulic Boundary Conditions																							
				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
96W-L	Jefferson WB	Levee	Future	18.0	0.9	18.4	1.5		0.2	2.5	0.5	19.0	6.6	3								0.000	0.003
97W-L	Jefferson WB	Levee	Existing	15.3	1.0	21.4	1.5		0.2	2.5	0.5	22.5	7.1	3								0.000	0.017
97W-LF	Jefferson WB	Structure /Wall	Future	18.1	0.9	21.4	1.5		0.2	2.5	0.5	22.5	9.8	3								0.000	0.000
97W-L	Jefferson WB	Levee	Future	18.1	0.9	18.5	1.5		0.2	2.5	0.5	19.0	9.8	3								0.000	0.003
98W-L	Jefferson WB	Levee	Existing	15.4	1.0	21.5	1.5		0.2	2.5	0.5	22.5	5.8	3								0.001	0.018
98W-LF	Jefferson WB	Structure /Wall	Future	18.1	0.9	21.5	1.5		0.2	2.5	0.5	22.5	8.5	3								0.000	0.001
98W-L	Jefferson WB	Levee	Future	18.1	0.9	18.5	1.5		0.2	2.5	0.5	19.0	8.5	3								0.000	0.003
99W-L	Jefferson WB	Levee	Existing	15.4	1.1	21.5	1.5		0.2	2.5	0.5	22.5	6.0	3								0.000	0.019
99W-LF	Jefferson WB	Structure /Wall	Future	18.1	0.9	21.5	1.5		0.2	2.5	0.5	22.5	8.7	3								0.000	0.001
99W-L	Jefferson WB	Levee	Future	18.1	0.9	18.6	1.5		0.2	2.5	0.5	19.0	8.7	3								0.000	0.004
100W-L	Jefferson WB	Levee	Existing	15.5	1.1	21.6	1.5		0.2	3.5	0.7	22.5	4.3	3								0.004	0.051
100W-L	Jefferson WB	Levee	Future	18.2	0.9	21.6	2.0		0.2	3.5	0.7	22.5	6.9	3								0.006	0.055
101W-L	Jefferson WB	Levee	Existing	15.5	1.1	18.7	1.5		0.2	3.5	0.7	19.0	3.8	3								0.004	0.057
101W-L	Jefferson WB	Levee	Future	18.2	1.0	21.7	2.0		0.2	3.5	0.7	22.5	6.5	3								0.006	0.055
102W-L	Jefferson WB	Levee	Existing	15.5	1.1	18.7	1.5		0.2	3.5	0.7	19.0	0.9	3								0.004	0.054
102W-LF	Jefferson WB	Structure /Wall	Future	18.2	1.0	21.8	2.0		0.2	3.5	0.7	22.5	3.6	3								0.001	0.006
102W-L	Jefferson WB	Levee	Future	18.2	1.0	18.8	2.0		0.2	3.5	0.7	19.0	3.6	3								0.006	0.056
103W-L	Jefferson WB	Levee	Existing	15.5	1.1	21.8	1.5		0.2	3.5	0.7	22.5	4.1	3								0.004	0.057
103W-L	Jefferson WB	Levee	Future	18.2	1.0	21.8	2.0		0.2	3.5	0.7	22.5	6.8	3								0.006	0.058
104W-L	Jefferson WB	Levee	Existing	15.5	1.1	18.8	1.5		0.2	3.5	0.7	19.0	4.8	3								0.003	0.055
104W-L	Jefferson WB	Levee	Future	18.2	1.0	21.9	2.0		0.2	3.5	0.7	22.5	7.5	3								0.005	0.056
105W-L	Jefferson WB	Levee	Existing	15.5	1.1	18.8	1.5		0.2	3.5	0.7	19.0	2.5	3								0.004	0.060
105W-L	Jefferson WB	Levee	Future	18.2	1.0	21.9	2.0		0.2	3.5	0.7	22.5	5.2	3								0.006	0.062
106W-L	Jefferson WB	Levee	Existing	15.6	1.1	18.9	1.5		0.2	3.5	0.7	19.0	5.0	3								0.004	0.064
106W-LF	Jefferson WB	Structure /Wall	Future	18.3	1.0	22.0	2.0		0.2	3.5	0.7	22.5	7.7	3								0.001	0.008
106W-L	Jefferson WB	Levee	Future	18.3	1.0	19.0	2.0		0.2	3.5	0.7	19.0	7.7	3								0.007	0.070
106W-LF	Jefferson WB	Structure /Wall	Future	18.3	1.0	22.1	1.5		0.2	2.5	0.5	22.5	7.7	3								0.001	0.008

West Bank (RM 70W to RM 118W)
1% Hydraulic Boundary Conditions

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
			Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
106W-L	Jefferson WB	Levee	Future	18.3	1.0	22.1	2.0		0.2	3.5	0.7	22.5	7.7	3								0.007	0.070
107W-L	Jefferson WB	Levee	Existing	15.6	1.1	19.0	1.5		0.2	2.5	0.5	19.0	-0.1	3								0.001	0.030
107W-LF	Jefferson WB	Structure /Wall	Future	18.3	1.0	22.2	1.5		0.2	2.5	0.5	22.5	2.6	3								0.000	0.001
107W-L	Jefferson WB	Levee	Future	18.3	1.0	22.2	1.5		0.2	2.5	0.5	22.5	2.6	3								0.000	0.006
108W-L	Jefferson WB	Levee	Existing	15.7	1.1	19.1	1.5		0.2	2.5	0.5	19.0	0.0	3								0.001	0.032
108W-LF	Jefferson WB	Structure /Wall	Future	18.3	1.0	22.3	1.5		0.2	2.5	0.5	22.5	2.6	3								0.000	0.001
108W-L	Jefferson WB	Levee	Future	18.3	1.0	22.3	1.5		0.2	2.5	0.5	22.5	2.6	3								0.000	0.006
109W-L	Jefferson WB	Levee	Existing	15.7	1.1	19.2	2.0		0.2	3.5	0.7	20.0	5.2	3								0.006	0.068
109W-L	Jefferson WB	Levee	Future	18.4	1.1	22.3	2.5		0.3	4.0	0.8	24.0	7.9	3								0.007	0.059
110W-L	Jefferson WB	Levee	Existing	15.7	1.1	19.2	2.0		0.2	3.5	0.7	20.0	6.1	3								0.006	0.068
110W-LF	Jefferson WB	Structure /Wall	Future	18.4	1.1	22.4	2.5		0.3	4.0	0.8	24.0	8.8	3								0.000	0.004
110W-L	Jefferson WB	Levee	Future	18.4	1.1	22.4	2.5		0.3	4.0	0.8	24.0	8.8	3								0.007	0.060
111W-L	Jefferson WB	Levee	Existing	15.7	1.1	19.3	2.0		0.2	3.5	0.7	20.0	2.8	3								0.006	0.070
111W-L	Jefferson WB	Levee	Future	18.4	1.1	22.5	2.5		0.3	4.0	0.8	24.0	5.5	3								0.007	0.061
112W-L	Jefferson WB	Levee	Existing	15.7	1.1	19.3	2.0		0.2	3.5	0.7	20.0	1.2	3								0.007	0.076
112W-L	Jefferson WB	Levee	Future	18.5	1.1	22.6	2.5		0.3	4.0	0.8	24.0	3.9	3								0.007	0.069
113W-L	Jefferson WB	Levee	Existing	15.8	1.1	19.4	2.0		0.2	3.5	0.7	20.0	4.3	3								0.007	0.081
113W-L	Jefferson WB	Levee	Future	18.5	1.1	22.7	2.5		0.3	4.0	0.8	24.0	7.0	3								0.008	0.070
114W-L	Jefferson WB	Levee	Existing	15.8	1.2	19.5	2.0		0.2	3.5	0.7	20.0	5.3	3								0.007	0.084
114W-L	Jefferson WB	Levee	Future	18.5	1.1	22.8	2.5		0.3	4.0	0.8	24.0	8.1	3								0.008	0.072
115W-L	St. Charles WB	Levee	Existing	15.9	1.2	19.6	2.0		0.2	3.5	0.7	20.0	4.9	3								0.007	0.097
115W-L	St. Charles WB	Levee	Future	18.6	1.2	22.9	2.5		0.3	4.0	0.8	24.0	7.7	3								0.009	0.078
116W-L	St. Charles WB	Levee	Existing	15.9	1.2	19.7	1.5		0.2	2.5	0.5	20.0	3.7	3								0.000	0.010
116W-L	St. Charles WB	Levee	Future	18.7	1.2	23.0	1.5		0.2	2.5	0.5	24.0	6.5	3								0.000	0.001
117W-L	St. Charles WB	Levee	Existing	16.0	1.2	19.7	1.5		0.2	2.5	0.5	20.0	3.4	3								0.000	0.010
117W-L	St. Charles WB	Levee	Future	18.7	1.2	23.1	1.5		0.2	2.5	0.5	24.0	6.1	3								0.000	0.001
118W-L	St. Charles WB	Levee	Existing	16.0	1.2	19.8	1.5		0.2	2.5	0.5	20.0	3.4	3								0.000	0.011

West Bank (RM 70W to RM 118W)
1% Hydraulic Boundary Conditions

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
118W-L	St. Charles WB	Levee	Future	18.8	1.2	23.2	1.5		0.2	2.5	0.5	24.0	6.1	3								0.000	0.001

New Orleans to Venice
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
		Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
11W-L	11	Levee	Existing	12.4	0.8			5.5	0.6	5.2	1.0	18.5	0	4								0.006	0.063
11W-L	11	Levee	Future	13.3	0.8			6.3	0.6	5.5	1.1	20.0	0	4								0.007	0.068
12W-L	12	Levee	Existing	12.7	0.9			5.5	0.6	5.2	1.0	18.5	0	4								0.01	0.093
12W-L	12	Levee	Future	13.6	0.9			6.3	0.6	5.5	1.1	20.5	0	4								0.007	0.064
13W-L	13	Levee	Existing	12.9	0.9			5.5	0.6	5.2	1.0	19.0	0	4								0.007	0.078
13W-L	13	Levee	Future	13.8	0.9			6.3	0.6	5.5	1.1	20.5	0	4								0.009	0.084
14W-L	14	Levee	Existing	13.1	0.9			6.0	0.6	5.5	1.1	19.5	0	4								0.01	0.093
14W-L	14	Levee	Future	14.1	0.9			6.8	0.7	5.8	1.2	21.5	0	4								0.007	0.068
15W-L	15	Levee	Existing	13.4	0.9			6.0	0.6	5.5	1.1	20.0	0	4								0.008	0.079
15W-L	15	Levee	Future	14.3	0.9			6.8	0.7	5.8	1.2	21.5	0	4								0.009	0.084
16W-L	16	Levee	Existing	13.6	0.9			6.0	0.6	5.5	1.1	20.5	0	4								0.006	0.066
16W-L	16	Levee	Future	14.5	0.9			6.8	0.7	5.8	1.2	22.0	0	4								0.008	0.072
17W-L	17	Levee	Existing	13.9	0.9			6.0	0.6	5.5	1.1	20.5	0	4								0.009	0.089
17W-L	17	Levee	Future	14.7	0.9			6.8	0.7	5.8	1.2	22.5	0	4								0.006	0.065
18W-L	18	Levee	Existing	14.1	0.9			6.0	0.6	5.5	1.1	21.0	0	4								0.008	0.075
18W-L	18	Levee	Future	15.0	0.9			6.8	0.7	5.8	1.2	22.5	0	4								0.009	0.081
19W-L	19	Levee	Existing	14.3	1.0			6.0	0.6	5.5	1.1	21.0	0	4								0.009	0.093
19W-L	19	Levee	Future	15.1	1.0			6.8	0.7	5.8	1.2	23.0	0	4								0.006	0.068
20W-L	20	Levee	Existing	14.5	1.0			6.0	0.6	5.5	1.1	21.5	0	4								0.007	0.076
20W-L	20	Levee	Future	15.3	1.0			6.8	0.7	5.8	1.2	23.0	0	4								0.008	0.08
21W-L	21	Levee	Existing	14.7	1.0			6.0	0.6	5.5	1.1	21.5	0	4								0.009	0.099
21W-L	21	Levee	Future	15.6	1.0			6.8	0.7	5.8	1.2	23.5	0	4								0.006	0.073
22W-L	22	Levee	Existing	14.8	1.1			6.0	0.6	5.5	1.1	22.0	0	4								0.006	0.072
22W-L	22	Levee	Future	15.7	1.1			6.8	0.7	5.8	1.2	23.5	0	4								0.007	0.078
23W-L	23	Levee	Existing	14.9	1.1			6.0	0.6	5.5	1.1	22.0	0	4								0.007	0.087
23W-L	23	Levee	Future	15.8	1.1			6.8	0.7	5.8	1.2	23.5	0	4								0.009	0.098
24W-L	24	Levee	Existing	15.0	1.1			6.0	0.6	5.5	1.1	22.0	0	4								0.008	0.098
24W-L	24	Levee	Future	16.0	1.1			6.8	0.7	5.8	1.2	24.0	0	4								0.007	0.079

New Orleans to Venice
1% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
25W-L	25	Levee	Existing	15.1	1.1			6.0	0.6	5.5	1.1	22.5	0	4								0.006	0.073
25W-L	25	Levee	Future	16.1	1.1			6.8	0.7	5.8	1.2	24.0	0	4								0.008	0.088
26W-L	26	Levee	Existing	15.3	1.1			6.5	0.7	5.7	1.1	23.0	0	4								0.006	0.077
26W-L	26	Levee	Future	16.2	1.1			7.3	0.7	6.0	1.2	24.5	0	4								0.008	0.089
27W-L	27	Levee	Existing	15.4	1.1			6.5	0.7	5.7	1.1	23.0	0	4								0.007	0.08
27W-L	27	Levee	Future	16.3	1.1			7.3	0.7	6.0	1.2	24.5	0	4								0.009	0.092
28W-L	28	Levee	Existing	15.5	1.1			6.5	0.7	5.7	1.1	23.0	0	4								0.009	0.091
28W-L	28	Levee	Future	16.5	1.1			7.3	0.7	6.0	1.2	25.0	0	4								0.007	0.075
29W-L	29	Levee	Existing	15.6	1.1			6.5	0.7	5.7	1.1	23.0	0	4								0.009	0.099
29W-L	29	Levee	Future	16.6	1.1			7.3	0.7	6.0	1.2	25.0	0	4								0.008	0.082
30W-L	30	Levee	Existing	15.6	1.1			6.5	0.7	5.7	1.1	23.0	0	4								0.009	0.097
30W-L	30	Levee	Future	16.6	1.1			7.3	0.7	6.0	1.2	25.0	0	4								0.008	0.082
31W-L	31	Levee	Existing	15.6	1.1			6.5	0.7	5.7	1.1	23.0	0	4								0.009	0.098
31W-L	31	Levee	Future	16.6	1.1			7.3	0.7	6.0	1.2	25.0	0	4								0.008	0.085
32W-L	32	Levee	Existing	15.5	1.1			6.5	0.7	5.7	1.1	23.0	0	4								0.009	0.091
32W-L	32	Levee	Future	16.5	1.1			7.3	0.7	6.0	1.2	25.0	0	4								0.008	0.08
33W-L	33	Levee	Existing	15.4	1.0			6.0	0.6	5.5	1.1	23.0	0	4								0.005	0.063
33W-L	33	Levee	Future	16.5	1.0			6.8	0.7	5.8	1.2	25.0	0	4								0.005	0.057
34W-L	34	Levee	Existing	15.5	1.0			6.0	0.6	5.5	1.1	23.0	0	4								0.005	0.061
34W-L	34	Levee	Future	16.5	1.0			6.8	0.7	5.8	1.2	25.0	0	4								0.005	0.057
35W-L	35	Levee	Existing	15.5	1.0			6.0	0.6	5.5	1.1	23.0	0	4								0.005	0.069
35W-L	35	Levee	Future	16.6	1.0			6.8	0.7	5.8	1.2	25.0	0	4								0.005	0.063
36W-L	36	Levee	Existing	15.6	1.1			6.0	0.6	5.5	1.1	23.0	0	4								0.006	0.074
36W-L	36	Levee	Future	16.7	1.0			6.8	0.7	5.8	1.2	25.0	0	4								0.006	0.068
37W-L	37	Levee	Existing	15.7	1.1			6.0	0.6	5.5	1.1	23.0	0	4								0.006	0.076
37W-L	37	Levee	Future	16.7	1.1			6.8	0.7	5.8	1.2	25.0	0	4								0.006	0.073
38W-L	38	Levee	Existing	15.7	1.1			6.0	0.6	5.5	1.1	23.0	0	4								0.007	0.077
38W-L	38	Levee	Future	16.8	1.1			6.8	0.7	5.8	1.2	25.0	0	4								0.007	0.073

New Orleans to Venice 1% Overtopping Design Criteria																							
				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
39W-L	39	Levee	Existing	15.7	1.1			6.0	0.6	5.5	1.1	23.0	0	4								0.007	0.083
39W-L	39	Levee	Future	16.9	1.1			6.8	0.7	5.8	1.2	25.0	0	4								0.007	0.084
40W-L	40	Levee	Existing	15.7	1.1			6.0	0.6	5.5	1.1	23.0	0	4								0.006	0.084
40W-L	40	Levee	Future	16.9	1.1			6.8	0.7	5.8	1.2	25.0	0	4								0.007	0.085
41W-L	41	Levee	Existing	15.7	1.1			5.5	0.6	5.2	1.0	23.0	0	4								0.003	0.046
41W-L	41	Levee	Future	16.9	1.1			6.3	0.6	5.5	1.1	25.0	0	4								0.004	0.053
42W-L	42	Levee	Existing	15.7	1.1			5.5	0.6	5.2	1.0	23.0	0	4								0.003	0.044
42W-L	42	Levee	Future	16.9	1.1			6.3	0.6	5.5	1.1	25.0	0	4								0.004	0.052
43W-L	43	Levee	Existing	15.6	1.0			5.0	0.5	5.0	1.0	23.0	0	4								0.001	0.024
43W-L	43	Levee	Future	16.9	1.1			5.8	0.6	5.4	1.1	25.0	0	4								0.002	0.037
44W-L	44	Levee	Existing	15.6	1.1			5.0	0.5	5.0	1.0	23.0	0	4								0.001	0.022
44W-L	44	Levee	Future	16.9	1.1			5.8	0.6	5.4	1.1	25.0	0	4								0.002	0.036
45W-L	45	Levee	Existing	15.5	1.0			3.5	0.4	4.5	0.9	23.5	0	3								0.008	0.06
45W-L	45	Levee	Future	16.9	1.0			3.8	0.4	4.5	0.9	25.5	0	3								0.007	0.056
46W-L	46	Levee	Existing	15.5	1.0			3.5	0.4	4.5	0.9	23.5	0	3								0.008	0.06
46W-L	46	Levee	Future	16.9	1.0			3.8	0.4	4.5	0.9	25.5	0	3								0.007	0.056
47W-L	47	Levee	Existing	15.5	1.0			3.5	0.4	4.5	0.9	23.5	0	3								0.008	0.06
47W-L	47	Levee	Future	16.9	1.0			3.8	0.4	4.5	0.9	25.5	0	3								0.007	0.058
48W-L	48	Levee	Existing	15.5	1.0			3.5	0.4	4.5	0.9	23.5	0	3								0.008	0.061
48W-L	48	Levee	Future	16.9	1.0			3.8	0.4	4.5	0.9	25.5	0	3								0.007	0.058
49W-L	49	Levee	Existing	15.5	1.0			3.5	0.4	4.5	0.9	23.5	0	3								0.008	0.059
49W-L	49	Levee	Future	17.0	1.0			3.8	0.4	4.5	0.9	25.5	0	3								0.008	0.058
50W-L	50	Levee	Existing	15.5	1.0			3.5	0.4	4.5	0.9	23.5	0	3								0.008	0.06
50W-L	50	Levee	Future	17.0	1.0			3.8	0.4	4.5	0.9	25.5	0	3								0.008	0.06
51W-L	51	Levee	Existing	15.5	1.0			3.5	0.4	4.5	0.9	23.5	0	3								0.008	0.058
51W-L	51	Levee	Future	17.0	1.0			3.8	0.4	4.5	0.9	25.5	0	3								0.008	0.06
52W-L	52	Levee	Existing	15.5	1.0			3.5	0.4	4.5	0.9	23.5	0	3								0.008	0.058
52W-L	52	Levee	Future	17.1	0.9			3.8	0.4	4.5	0.9	25.5	0	3								0.009	0.062

New Orleans to Venice 1% Overtopping Design Criteria																							
				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
			Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
53W-L	53	Levee	Existing	15.5	0.9			3.5	0.4	4.5	0.9	23.5	0	3								0.008	0.061
53W-L	53	Levee	Future	17.1	0.9			3.8	0.4	4.5	0.9	25.5	0	3								0.009	0.066
54W-L	54	Levee	Existing	15.6	1.0			3.5	0.4	4.5	0.9	23.5	0	3								0.008	0.058
54W-L	54	Levee	Future	17.2	0.9			3.8	0.4	4.5	0.9	25.5	0	3								0.009	0.065
55W-L	55	Levee	Existing	15.6	0.9			3.5	0.4	4.5	0.9	23.5	0	3								0.009	0.06
55W-L	55	Levee	Future	17.3	0.9			3.8	0.4	4.5	0.9	25.5	0	3								0.01	0.068
56W-L	56	Levee	Existing	15.7	1.0			3.5	0.4	4.5	0.9	23.5	0	3								0.01	0.063
56W-L	56	Levee	Future	17.4	1.0			3.8	0.4	4.5	0.9	25.5	0	3								0.01	0.075
57W-L	57	Levee	Existing	15.7	1.0			3.5	0.4	4.5	0.9	23.5	0	3								0.009	0.062
57W-L	57	Levee	Future	17.4	1.0			3.8	0.4	4.5	0.9	25.5	0	3								0.01	0.078
58W-L	58	Levee	Existing	15.7	0.9			3.5	0.4	4.5	0.9	23.5	0	3								0.009	0.063
58W-L	58	Levee	Future	17.4	0.9			3.8	0.4	4.5	0.9	25.5	0	3								0.01	0.076
59W-L	59	Levee	Existing	15.7	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.009	0.067
59W-L	59	Levee	Future	17.5	0.9			3.3	0.3	4.0	0.8	24.5	0	3								0.007	0.055
60W-L	60	Levee	Existing	15.7	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.009	0.068
60W-L	60	Levee	Future	17.5	0.9			3.3	0.3	4.0	0.8	24.5	0	3								0.007	0.057
61W-L	61	Levee	Existing	15.7	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.009	0.064
61W-L	61	Levee	Future	17.5	0.9			3.3	0.3	4.0	0.8	24.5	0	3								0.007	0.054
62W-L	62	Levee	Existing	15.6	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.008	0.063
62W-L	62	Levee	Future	17.5	0.9			3.3	0.3	4.0	0.8	24.5	0	3								0.007	0.056
63W-L	63	Levee	Existing	15.6	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.009	0.062
63W-L	63	Levee	Future	17.5	0.9			3.3	0.3	4.0	0.8	24.5	0	3								0.007	0.058
64W-L	64	Levee	Existing	15.6	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.008	0.061
64W-L	64	Levee	Future	17.6	0.9			3.3	0.3	4.0	0.8	24.5	0	3								0.007	0.058
65W-L	65	Levee	Existing	15.5	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.008	0.059
65W-L	65	Levee	Future	17.6	0.9			3.3	0.3	4.0	0.8	24.5	0	3								0.007	0.059
66W-L	66	Levee	Existing	15.5	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.008	0.056
66W-L	66	Levee	Future	17.6	0.9			3.3	0.3	4.0	0.8	24.5	0	3								0.008	0.06

New Orleans to Venice 1% Overtopping Design Criteria																							
				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
67W-L	67	Levee	Existing	15.5	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.007	0.058
67W-L	67	Levee	Future	17.7	0.9			3.3	0.3	4.0	0.8	24.5	0	3								0.008	0.066
68W-L	68	Levee	Existing	15.5	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.007	0.06
68W-L	68	Levee	Future	17.8	0.9			3.3	0.3	4.0	0.8	24.5	0	3								0.009	0.072
69W-L	69	Levee	Existing	15.5	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.007	0.059
69W-L	69	Levee	Future	17.8	0.9			3.3	0.3	4.0	0.8	24.5	0	3								0.009	0.075
70W-L	70	Levee	Existing	15.4	0.9			3.0	0.3	4.0	0.8	22.0	0	3								0.007	0.056
70W-L	70	Levee	Future	17.8	1.0			3.3	0.3	4.0	0.8	24.5	0	3								0.01	0.076

New Orleans to Venice
2% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
11W-L	11	Levee	Existing	9.7	0.8			4.8	0.5	4.9	1.0	14.5	0	4								0.006	0.066
11W-L	11	Levee	Future	10.5	0.8			5.5	0.6	5.3	1.1	16.0	0	4								0.008	0.072
12W-L	12	Levee	Existing	9.9	0.8			4.8	0.5	4.9	1.0	15.0	0	4								0.005	0.056
12W-L	12	Levee	Future	10.7	0.8			5.5	0.6	5.3	1.1	16.5	0	4								0.006	0.062
13W-L	13	Levee	Existing	10.1	0.8			4.8	0.5	4.9	1.0	15.0	0	4								0.007	0.079
13W-L	13	Levee	Future	11.0	0.8			5.5	0.6	5.3	1.1	16.5	0	4								0.009	0.083
14W-L	14	Levee	Existing	10.3	0.8			4.8	0.5	4.9	1.0	15.5	0	4								0.005	0.06
14W-L	14	Levee	Future	11.2	0.8			5.5	0.6	5.3	1.1	17.0	0	4								0.007	0.068
15W-L	15	Levee	Existing	10.5	0.8			4.8	0.5	4.9	1.0	15.5	0	4								0.007	0.08
15W-L	15	Levee	Future	11.4	0.8			5.5	0.6	5.3	1.1	17.0	0	4								0.009	0.086
16W-L	16	Levee	Existing	10.7	0.8			5.0	0.5	5.0	1.0	16.0	0	4								0.006	0.067
16W-L	16	Levee	Future	11.5	0.8			5.8	0.6	5.4	1.1	17.5	0	4								0.008	0.073
17W-L	17	Levee	Existing	10.9	0.8			5.0	0.5	5.0	1.0	16.0	0	4								0.008	0.088
17W-L	17	Levee	Future	11.7	0.8			5.8	0.6	5.4	1.1	18.0	0	4								0.006	0.059
18W-L	18	Levee	Existing	11.1	0.8			5.0	0.5	5.0	1.0	16.5	0	4								0.007	0.07
18W-L	18	Levee	Future	11.9	0.8			5.8	0.6	5.4	1.1	18.0	0	4								0.008	0.075
19W-L	19	Levee	Existing	11.2	0.9			5.0	0.5	5.0	1.0	16.5	0	4								0.008	0.085
19W-L	19	Levee	Future	12.0	0.9			5.8	0.6	5.4	1.1	18.0	0	4								0.009	0.09
20W-L	20	Levee	Existing	11.4	0.9			5.0	0.5	5.0	1.0	17.0	0	4								0.005	0.064
20W-L	20	Levee	Future	12.2	0.9			5.8	0.6	5.4	1.1	18.5	0	4								0.007	0.069
21W-L	21	Levee	Existing	11.5	0.9			5.0	0.5	5.0	1.0	17.0	0	4								0.007	0.083
21W-L	21	Levee	Future	12.4	0.9			5.8	0.6	5.4	1.1	18.5	0	4								0.008	0.089
22W-L	22	Levee	Existing	11.6	1.0			5.0	0.5	5.0	1.0	17.0	0	4								0.008	0.092
22W-L	22	Levee	Future	12.5	1.0			5.8	0.6	5.4	1.1	19.0	0	4								0.006	0.063
23W-L	23	Levee	Existing	11.7	1.0			5.0	0.5	5.0	1.0	17.5	0	4								0.005	0.068
23W-L	23	Levee	Future	12.6	1.0			5.8	0.6	5.4	1.1	19.0	0	4								0.007	0.076
24W-L	24	Levee	Existing	11.8	1.0			5.0	0.5	5.0	1.0	17.5	0	4								0.006	0.079
24W-L	24	Levee	Future	12.7	1.0			5.8	0.6	5.4	1.1	19.0	0	4								0.008	0.09

New Orleans to Venice 2% Overtopping Design Criteria																							
				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
25W-L	25	Levee	Existing	11.9	1.0			5.0	0.5	5.0	1.0	17.5	0	4								0.007	0.087
25W-L	25	Levee	Future	12.8	1.0			5.8	0.6	5.4	1.1	19.0	0	4								0.009	0.1
26W-L	26	Levee	Existing	12.0	1.0			5.5	0.6	5.2	1.0	18.0	0	4								0.006	0.077
26W-L	26	Levee	Future	12.9	1.0			6.3	0.6	5.5	1.1	19.5	0	4								0.007	0.082
27W-L	27	Levee	Existing	12.1	1.0			5.5	0.6	5.2	1.0	18.0	0	4								0.007	0.08
27W-L	27	Levee	Future	13.0	1.0			6.3	0.6	5.5	1.1	19.5	0	4								0.008	0.084
28W-L	28	Levee	Existing	12.1	1.0			5.5	0.6	5.2	1.0	18.0	0	4								0.008	0.085
28W-L	28	Levee	Future	13.0	1.0			6.3	0.6	5.5	1.1	19.5	0	4								0.009	0.091
29W-L	29	Levee	Existing	12.1	1.0			5.5	0.6	5.2	1.0	18.0	0	4								0.007	0.089
29W-L	29	Levee	Future	13.1	1.0			6.3	0.6	5.5	1.1	19.5	0	4								0.009	0.096
30W-L	30	Levee	Existing	12.1	1.0			5.5	0.6	5.2	1.0	18.0	0	4								0.007	0.084
30W-L	30	Levee	Future	13.1	1.0			6.3	0.6	5.5	1.1	19.5	0	4								0.009	0.097
31W-L	31	Levee	Existing	12.1	1.0			5.5	0.6	5.2	1.0	18.0	0	4								0.007	0.084
31W-L	31	Levee	Future	13.1	1.0			6.3	0.6	5.5	1.1	19.5	0	4								0.009	0.093
32W-L	32	Levee	Existing	12.0	1.0			5.5	0.6	5.2	1.0	18.0	0	4								0.006	0.074
32W-L	32	Levee	Future	13.0	1.0			6.3	0.6	5.5	1.1	19.5	0	4								0.008	0.087
33W-L	33	Levee	Existing	11.9	1.0			5.5	0.6	5.2	1.0	18.0	0	4								0.006	0.066
33W-L	33	Levee	Future	12.9	1.0			6.3	0.6	5.5	1.1	19.5	0	4								0.007	0.075
34W-L	34	Levee	Existing	11.9	0.9			5.0	0.5	5.0	1.0	18.0	0	4								0.003	0.048
34W-L	34	Levee	Future	12.9	0.9			5.8	0.6	5.4	1.1	19.5	0	4								0.006	0.07
35W-L	35	Levee	Existing	12.0	1.0			5.0	0.5	5.0	1.0	18.0	0	4								0.004	0.056
35W-L	35	Levee	Future	13.0	0.9			5.8	0.6	5.4	1.1	19.5	0	4								0.007	0.074
36W-L	36	Levee	Existing	12.0	1.0			5.0	0.5	5.0	1.0	18.0	0	4								0.004	0.058
36W-L	36	Levee	Future	13.0	1.0			5.8	0.6	5.4	1.1	19.5	0	4								0.007	0.077
37W-L	37	Levee	Existing	12.0	1.0			5.0	0.5	5.0	1.0	18.0	0	4								0.004	0.058
37W-L	37	Levee	Future	13.1	1.0			5.8	0.6	5.4	1.1	19.5	0	4								0.008	0.082
38W-L	38	Levee	Existing	12.1	1.0			5.0	0.5	5.0	1.0	18.0	0	4								0.005	0.06
38W-L	38	Levee	Future	13.2	1.0			5.8	0.6	5.4	1.1	19.5	0	4								0.009	0.095

New Orleans to Venice
2% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
39W-L	39	Levee	Existing	12.1	1.0			5.0	0.5	5.0	1.0	18.0	0	4								0.005	0.068
39W-L	39	Levee	Future	13.2	1.0			5.8	0.6	5.4	1.1	19.5	0	4								0.009	0.1
40W-L	40	Levee	Existing	12.1	1.0			5.0	0.5	5.0	1.0	18.0	0	4								0.005	0.072
40W-L	40	Levee	Future	13.3	1.0			5.8	0.6	5.4	1.1	20.0	0	4								0.006	0.072
41W-L	41	Levee	Existing	12.2	1.0			4.5	0.5	4.7	0.9	18.0	0	4								0.003	0.042
41W-L	41	Levee	Future	13.4	1.0			5.3	0.5	5.1	1.0	20.0	0	4								0.003	0.048
42W-L	42	Levee	Existing	12.2	1.0			4.5	0.5	4.7	0.9	18.0	0	4								0.003	0.043
42W-L	42	Levee	Future	13.4	1.0			5.3	0.5	5.1	1.0	20.0	0	4								0.004	0.051
43W-L	43	Levee	Existing	12.2	1.0			4.0	0.4	4.5	0.9	18.0	0	4								0.001	0.023
43W-L	43	Levee	Future	13.5	1.0			4.8	0.5	4.9	1.0	20.0	0	4								0.002	0.033
44W-L	44	Levee	Existing	12.3	1.0			4.0	0.4	4.5	0.9	18.0	0	4								0.001	0.023
44W-L	44	Levee	Future	13.6	1.0			4.8	0.5	4.9	1.0	20.0	0	4								0.002	0.036
45W-L	45	Levee	Existing	12.3	1.0			2.8	0.3	4.3	0.9	18.5	0	3								0.008	0.054
45W-L	45	Levee	Future	13.6	1.0			3.0	0.3	4.3	0.9	20.5	0	3								0.007	0.051
46W-L	46	Levee	Existing	12.3	1.0			2.8	0.3	4.3	0.9	18.5	0	3								0.008	0.056
46W-L	46	Levee	Future	13.7	1.0			3.0	0.3	4.3	0.9	20.5	0	3								0.007	0.055
47W-L	47	Levee	Existing	12.3	1.0			2.8	0.3	4.3	0.9	18.5	0	3								0.008	0.055
47W-L	47	Levee	Future	13.7	1.0			3.0	0.3	4.3	0.9	20.5	0	3								0.008	0.054
48W-L	48	Levee	Existing	12.3	0.9			2.8	0.3	4.3	0.9	18.5	0	3								0.008	0.06
48W-L	48	Levee	Future	13.8	1.0			3.0	0.3	4.3	0.9	20.5	0	3								0.008	0.057
49W-L	49	Levee	Existing	12.4	0.9			2.8	0.3	4.3	0.9	18.5	0	3								0.01	0.062
49W-L	49	Levee	Future	13.9	0.9			3.0	0.3	4.3	0.9	20.5	0	3								0.008	0.059
50W-L	50	Levee	Existing	12.4	0.9			2.8	0.3	4.3	0.9	18.5	0	3								0.01	0.063
50W-L	50	Levee	Future	13.9	0.9			3.0	0.3	4.3	0.9	20.5	0	3								0.009	0.063
51W-L	51	Levee	Existing	12.5	0.9			2.8	0.3	4.3	0.9	19.0	0	3								0.006	0.042
51W-L	51	Levee	Future	14.0	0.9			3.0	0.3	4.3	0.9	21.0	0	3								0.006	0.046
52W-L	52	Levee	Existing	12.5	0.9			2.8	0.3	4.3	0.9	19.0	0	3								0.006	0.042
52W-L	52	Levee	Future	14.1	0.9			3.0	0.3	4.3	0.9	21.0	0	3								0.006	0.045

New Orleans to Venice
2% Overtopping Design Criteria

				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
53W-L	53	Levee	Existing	12.6	0.9			2.8	0.3	4.3	0.9	19.0	0	3								0.007	0.047
53W-L	53	Levee	Future	14.2	0.9			3.0	0.3	4.3	0.9	21.0	0	3								0.007	0.049
54W-L	54	Levee	Existing	12.7	0.9			2.8	0.3	4.3	0.9	19.0	0	3								0.007	0.046
54W-L	54	Levee	Future	14.3	0.9			3.0	0.3	4.3	0.9	21.0	0	3								0.008	0.053
55W-L	55	Levee	Existing	12.7	0.9			2.8	0.3	4.3	0.9	19.0	0	3								0.007	0.047
55W-L	55	Levee	Future	14.3	0.9			3.0	0.3	4.3	0.9	21.0	0	3								0.009	0.055
56W-L	56	Levee	Existing	12.7	0.9			2.8	0.3	4.3	0.9	19.0	0	3								0.008	0.051
56W-L	56	Levee	Future	14.4	0.9			3.0	0.3	4.3	0.9	21.0	0	3								0.009	0.06
57W-L	57	Levee	Existing	12.7	0.9			2.8	0.3	4.3	0.9	19.0	0	3								0.008	0.05
57W-L	57	Levee	Future	14.4	0.9			3.0	0.3	4.3	0.9	21.0	0	3								0.009	0.062
58W-L	58	Levee	Existing	12.8	0.9			2.8	0.3	4.3	0.9	19.0	0	3								0.008	0.051
58W-L	58	Levee	Future	14.5	0.9			3.0	0.3	4.3	0.9	21.0	0	3								0.01	0.065
59W-L	59	Levee	Existing	12.8	0.9			2.3	0.2	3.8	0.8	17.5	0	3								0.009	0.063
59W-L	59	Levee	Future	14.5	0.9			2.5	0.3	3.8	0.8	20.0	0	3								0.006	0.047
60W-L	60	Levee	Existing	12.8	0.8			2.3	0.2	3.8	0.8	17.5	0	3								0.009	0.064
60W-L	60	Levee	Future	14.5	0.8			2.5	0.3	3.8	0.8	20.0	0	3								0.006	0.048
61W-L	61	Levee	Existing	12.7	0.8			2.3	0.2	3.8	0.8	17.5	0	3								0.008	0.057
61W-L	61	Levee	Future	14.5	0.8			2.5	0.3	3.8	0.8	20.0	0	3								0.006	0.046
62W-L	62	Levee	Existing	12.7	0.8			2.3	0.2	3.8	0.8	17.5	0	3								0.008	0.059
62W-L	62	Levee	Future	14.5	0.8			2.5	0.3	3.8	0.8	20.0	0	3								0.006	0.045
63W-L	63	Levee	Existing	12.7	0.8			2.3	0.2	3.8	0.8	17.5	0	3								0.008	0.054
63W-L	63	Levee	Future	14.5	0.8			2.5	0.3	3.8	0.8	20.0	0	3								0.006	0.045
64W-L	64	Levee	Existing	12.7	0.8			2.3	0.2	3.8	0.8	17.5	0	3								0.008	0.053
64W-L	64	Levee	Future	14.6	0.8			2.5	0.3	3.8	0.8	20.0	0	3								0.007	0.046
65W-L	65	Levee	Existing	12.7	0.8			2.3	0.2	3.8	0.8	17.5	0	3								0.008	0.054
65W-L	65	Levee	Future	14.6	0.8			2.5	0.3	3.8	0.8	20.0	0	3								0.007	0.05
66W-L	66	Levee	Existing	12.6	0.8			2.3	0.2	3.8	0.8	17.5	0	3								0.007	0.05
66W-L	66	Levee	Future	14.6	0.8			2.5	0.3	3.8	0.8	20.0	0	3								0.006	0.05

New Orleans to Venice 2% Overtopping Design Criteria																								
				1% Surge level (ft)		0.2% Surge level (ft)	1% Significant wave height (ft)			1% Peak Period (s) Tp		Structure El	Depth for Design	Levee Slope	Upper Inflection Pt	Berm Slope	Lower Inflection Pt		Friction			Overtopping	
Segment	Name	Type	Condition	Mean	Std	Mean	STWAVE wave ht	Modified Design wave ht	STD	Mean	Std	Height (ft)	(ft)	tan α	El (ft)	tan α	El (ft)	Berm Factor	Upper	Berm	Lower	q50 (cft/s per ft)	q90 (cft/s per ft)
67W-L	67	Levee	Existing	12.7	0.8			2.3	0.2	3.8	0.8	17.5	0	3								0.007	0.054
67W-L	67	Levee	Future	14.7	0.8			2.5	0.3	3.8	0.8	20.0	0	3								0.007	0.054
68W-L	68	Levee	Existing	12.7	0.8			2.3	0.2	3.8	0.8	17.5	0	3								0.007	0.055
68W-L	68	Levee	Future	14.7	0.8			2.5	0.3	3.8	0.8	20.0	0	3								0.008	0.058
69W-L	69	Levee	Existing	12.6	0.8			2.3	0.2	3.8	0.8	17.5	0	3								0.006	0.05
69W-L	69	Levee	Future	14.7	0.9			2.5	0.3	3.8	0.8	20.0	0	3								0.007	0.06
70W-L	70	Levee	Existing	12.5	0.9			2.3	0.2	3.8	0.8	17.5	0	3								0.006	0.049
70W-L	70	Levee	Future	14.7	0.9			2.5	0.3	3.8	0.8	20.0	0	3								0.008	0.06