1.0 HYDRAULICS

1.1 Design Philosophy for Preliminary Design of Hurricane Protection System

This chapter presents the hydraulic design approach to determine protection system design elevations sufficient to provide protection from a hurricane event that would produce a 1% exceedence surge elevation and associated waves. This surge elevation has a one-percent chance of being equaled or exceeded during any year. The protection system design elevations, referenced in this document as the 1% exceedence design elevations, have been developed for two authorized hurricane protection projects in the New Orleans area: Lake Pontchartrain, LA & Vicinity; and West Bank & Vicinity (see Figure 1.1).

An extensive USACE/FEMA internal review and ASCE external review has been conducted on the approach during the period March through August 2007. The review documents can be found in USACE/FEMA South East Louisiana Joint Surge Study Independent Technical Review (Draft report 15 August 2007) and ASCE One percent Review Team (OPRT), Report Number 1 (31 May 2007) and 2 (30 July 2007).

Initial design elevations for Lake Pontchartrain, LA & Vicinity; and West Bank & Vicinity projects can be found in the report, “Elevations for Design of Hurricane Protection Levees and Structures,” dated September 2007. Hydraulic design and analysis associated with upcoming investigations will be documented in engineering analysis reports and also in addenda to the report. All hydraulic analyses associated with the two protection systems can be found in one comprehensive document.

To assure continuity of design methodology and provide close quality management, final design elevations utilized throughout the New Orleans area will be reviewed by the New Orleans District Engineering Division Chief of Hydraulics and Hydrologic Branch.

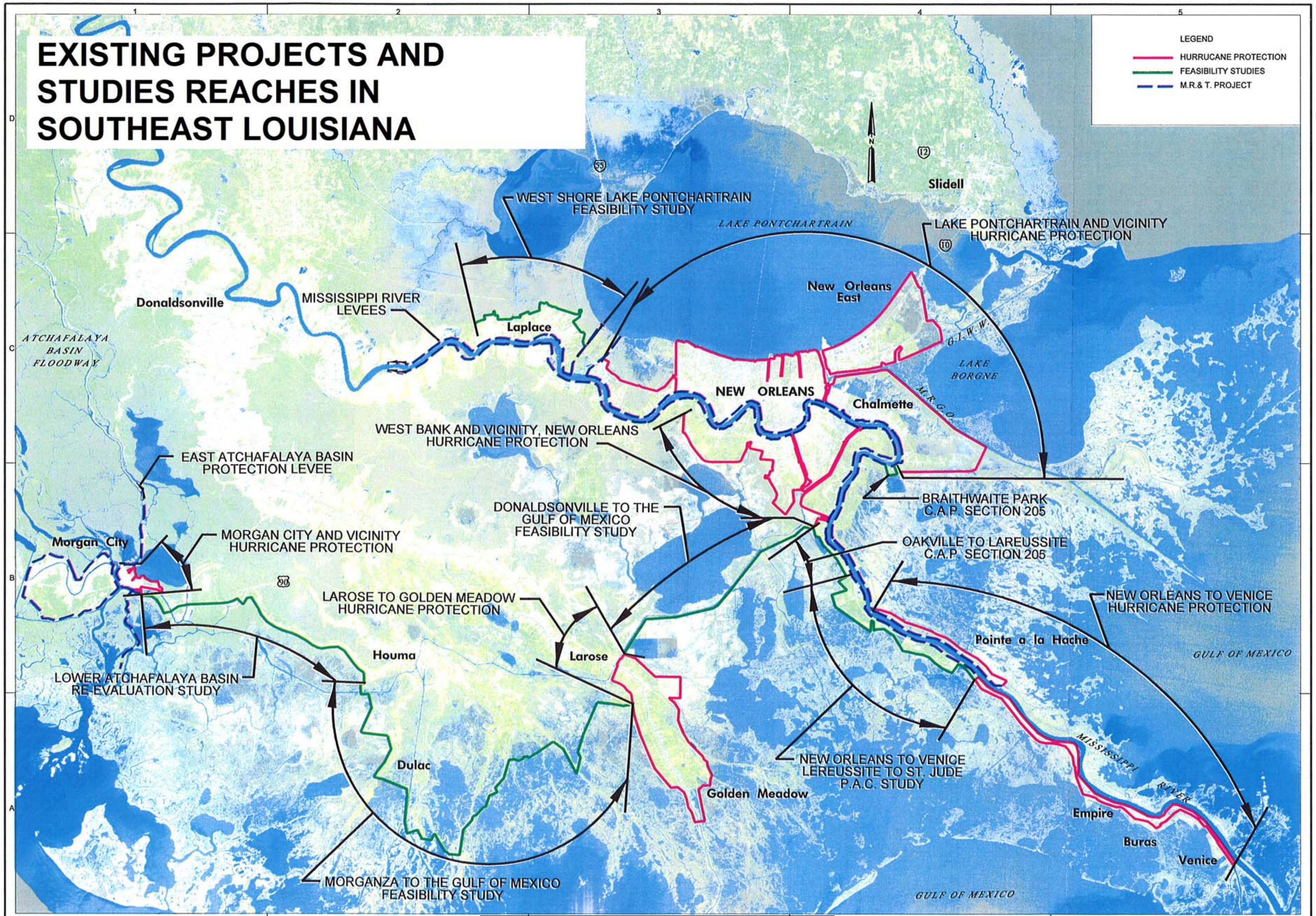


Figure 1.1 Map of existing projects and studies

1.2 Input Data and Methods for Design Approach

1.2.1 JPM-OS process

In 2006 and 2007, a team of Corps of Engineers, FEMA, NOAA, private sector, and academia developed a new process for estimating hurricane inundation probabilities, the Joint Probability Method with Optimal Sampling process (JPM-OS), see Resio (2007). This work was initiated for the Louisiana Coastal Protection and Restoration study (LACPR), but now is being applied to Corps work under the 4th supplemental appropriation, Interagency Performance Evaluation Team (IPET) risk analysis, and FEMA Base Flood Elevations for production of DFIRMs for coastal Louisiana and Texas. The Corps and FEMA work use the same model grids, the same model software, the same model input, such as wind fields, and the same method for estimating hurricane inundation probabilities. The JPM-OS process is shown in Figure 1.2. A more detailed description of the process and the modeling can be found in the White Paper, "Estimating Hurricane Inundation Probabilities" and documents prepared for FEMA for the coastal base flood elevation work.

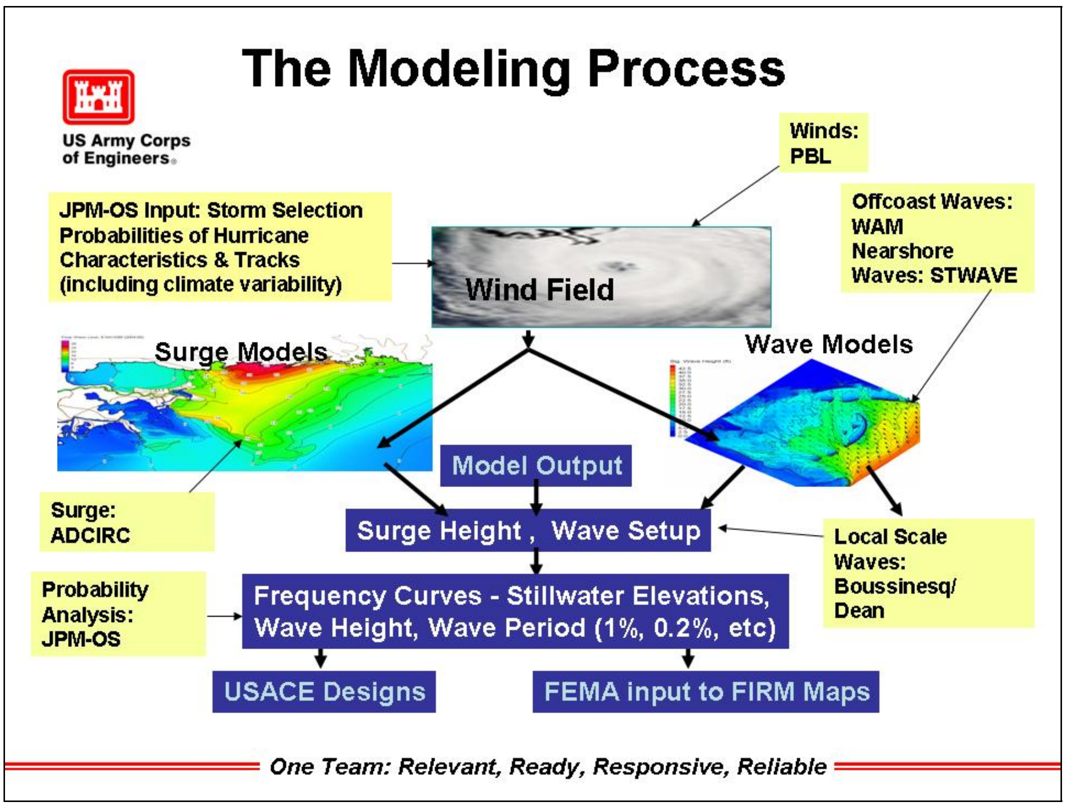


Figure 1.2 – The different components and their interaction in the JPM-OS Process

1.2.2 Modeling process

The following models are used in the JPM-OS process:

PBL – Planetary Boundary Layer Model. A marine planetary boundary layer model which links marine wind profiles to large scale pressure gradients and thermal properties has been developed by Oceanweather, Inc. Oceanweather, Inc is an internationally known company serving the international shipping, offshore industry and coastal engineering communities.

ADCIRC – Advanced Circulation Model. The ADCIRC model is used for the surge modeling. ADCIRC was developed by the ADCIRC Development Group which includes representatives from the University of North Carolina, the University of Oklahoma, the University of Notre Dame, and the University of Texas. The New Orleans District (MVN) is a development partner with the ADCIRC Development Group. The ADCIRC Model is a state-of-the-art model that solves the generalized wave-continuity equation on linear triangular elements. For the coastal Louisiana modeling, the finite element grid contains approximately 2.1 million horizontal nodes and 4.2 million elements.

WAM - The global ocean WAve prediction Model called WAM is a third generation wave model developed by the Corps of Engineers Coastal and Hydraulics Laboratory (CHL). WAM is used for offshore waves and boundary conditions for the nearshore wave modeling. WAM predicts directional spectra as well as wave properties such as significant wave height, mean wave direction and frequency, swell wave height and mean direction, and wind stress fields corrected by including the wave induced stress and the drag coefficient at each grid point at chosen output times.

STWAVE – Steady State Spectral Wave Model. STWAVE is a nearshore wave model developed by CHL. For the JPM-OS effort, STWAVE is used to generate the nearshore wave heights and wave periods using boundary conditions from the WAM modeling. The WAM-to-STWAVE procedure is applied for each storm. For the analyses completed to date, the STWAVE model did not include frictional effects.

The JPM-OS modeling process is as follows (see also Figure 1.2). The PBL model is used to generate the wind fields required in the JPM-OS process. For each storm, the PBL model is used to construct 15-minute snapshots of wind and pressure fields for driving the surge and wave models. ADCIRC, WAM, and STWAVE model runs are performed on high speed computers at the Corps of Engineers Engineering Research and Development Center (ERDC) in Vicksburg, MS, the Lonestar computer at University of Texas, and similar computers. With all major rivers already “spun up”, the surge model ADCIRC is initiated assuming zero tide. The spectral deep water wave model WAM is run, in parallel with the initial ADCIRC run, to establish the directional wave spectra that serve as the boundary conditions for the near-coast wave model, STWAVE. The STWAVE model is used to produce the wave fields and estimated radiation stress fields. These stress fields, added to the PBL estimated wind stresses, are used in the ADCIRC model for the time period during which the radiation stress makes a significant contribution to the water levels.

Two conditions of the hurricane protection system have been modeled with ADCIRC/STWAVE for design purposes: 2007 condition and 2010 condition. The **2007 condition** considers the interim gates and closures at the three outfall canals and levees and floodwalls constructed to pre-Katrina authorized elevations. The **2010 condition** considers the permanent gates and closures at the three outfall canals, the gate on the GIWW/MRGO, and levees and floodwalls constructed to elevations at or greater than the preliminary 1% design elevations. For the 2010 runs, no gate is present at Seabrook.

For most Joint Probability Methods, several thousand events are evaluated. With the JPM-OS method, optimal sampling allows for a smaller number of events to be used. Based on optimized sampling, 152 hurricane events were modeled for the 2007 condition, and 56 hurricane events have been modeled for the 2010 condition. For the 2010 condition, the output from the 56 storms have been used with 96 storms from the 2007 condition to create a dataset of 152 storms required for the frequency analysis. A relationship has been determined from the two sets of conditions and applied to achieve a consistent dataset.

The 2007 results from ADCIRC and STWAVE have been used for Lake Pontchartrain Lakefront area and the West Bank. This area is not affected by the gates at GIWW/MRGO. The 2010 model results have been used for the analysis of the GIWW/MRGO gate were applied to the levee/floodwall sections starting from South Point to GIWW, the GIWW sections outside the gate and the St Bernard levee sections. In addition to that, the levee/floodwall sections of the GIWW and IHNC inside the gate with no Seabrook Gate have utilized the ADCIRC results.

1.2.3 Frequency Analysis

The output from the ADCIRC and STWAVE models used in the frequency analysis are the maximum surge elevation and maximum wave characteristics (significant wave height, peak period, and wave direction) at approximately 600 feet in front of the levee or floodwall. Typical parameters which are to be computed based on the surge level and the wave characteristics are the wave run-up and the overtopping rate. These parameters depend also on the levee geometry (i.e. levee height and levee slope). The determination of the wave overtopping will be discussed in Section 1.2.4.

An example of the model output at two locations within the hurricane protection system is shown in Figure 1.3. The wave characteristics along Lake Pontchartrain are typically wind-generated and depth-limited waves. There is a high correlation between the wave height and the wave period and between the surge level and wave height for this area. In contrast, the results at the MRGO are much more scattered. The relationship between the surge level and the wave height is less evident, and the wave period strongly varies as a function of the wave height. Long wave periods are observed for a few storm conditions. The long wave periods are related to swell waves from the ocean.

A probabilistic model is used to derive the surge elevation, wave height, and wave period frequency curves at specific points along the hurricane protection system using output from ADCIRC and STWAVE. This probabilistic model takes into account the joint probability of forward speed, size, central pressure, angle of approach and geographic distribution of the hurricanes. For more information, the reader is referred to Resio (2007).

Surge frequency curves are estimated from the ADCIRC output of the 152 storms for 2007 and 2010 conditions. There may be instances where there is no output from the 152 storms. In this case, estimates are to be made of the surge elevation for the missing output so that the frequency analysis continued to be based on 152 values. The resulting 1% surge levels are considered to be “best estimate” values. In addition to the best estimates, the probabilistic model also provides an error estimate of the 1% surge levels. Errors are generally in the order of 1 – 2 ft for the 1% surge levels.

The same methodology is also used to develop frequency curves for wave height and wave period. Examples of frequency curves can be found in Figure 1.4. The errors in the 1% wave height and wave period have been based on expert judgment (Smith, pers. comm.). The standard deviations of the 1% wave height and wave period are assumed to be 10% and 20% of the best estimate value, respectively.

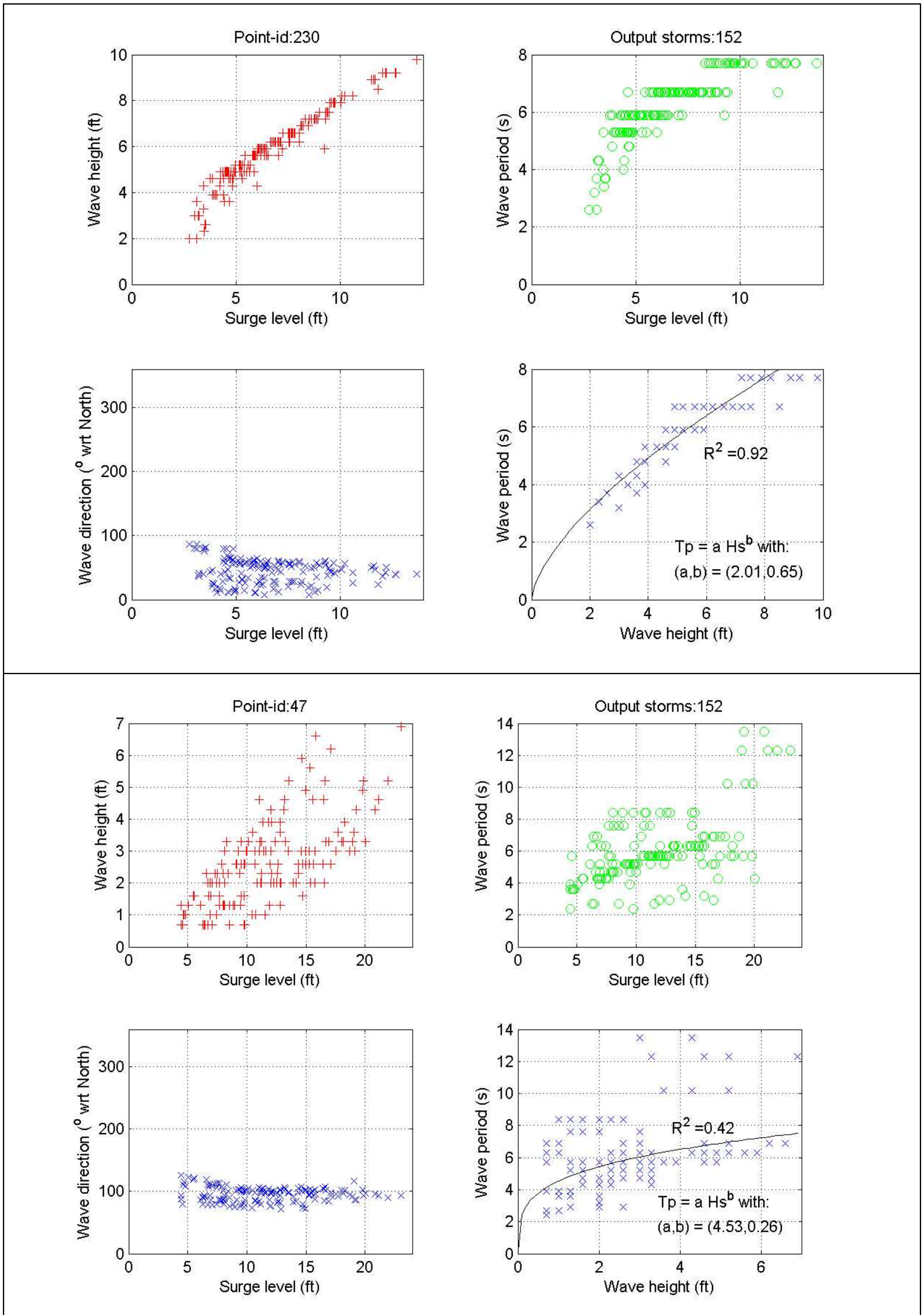
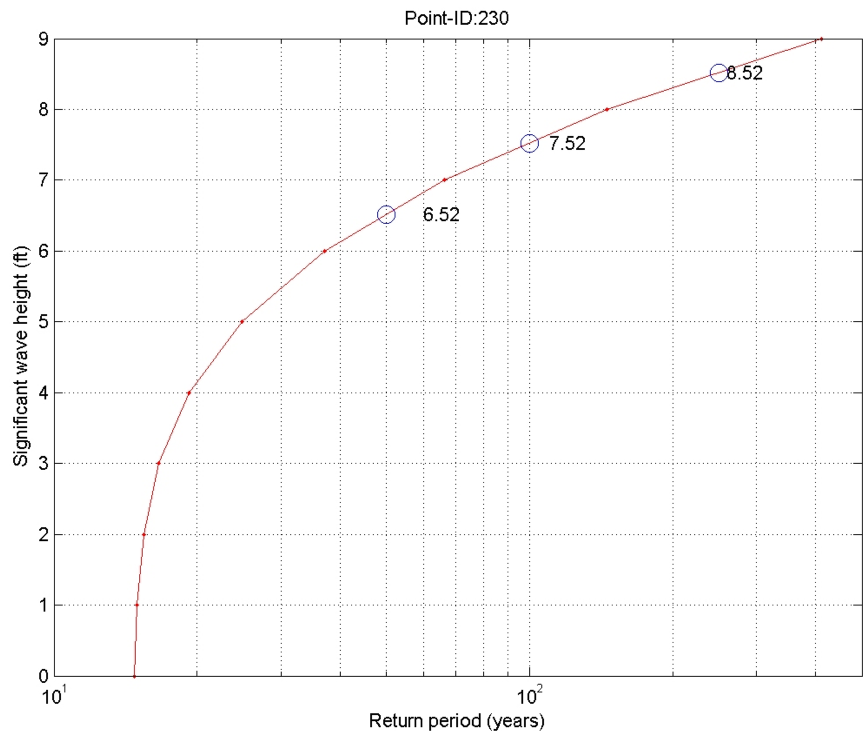


Figure 1.3 – Numerical results at Lake Pontchartrain (upper panel) and MRGO (lower panel) from ADCIRC and STWAVE.



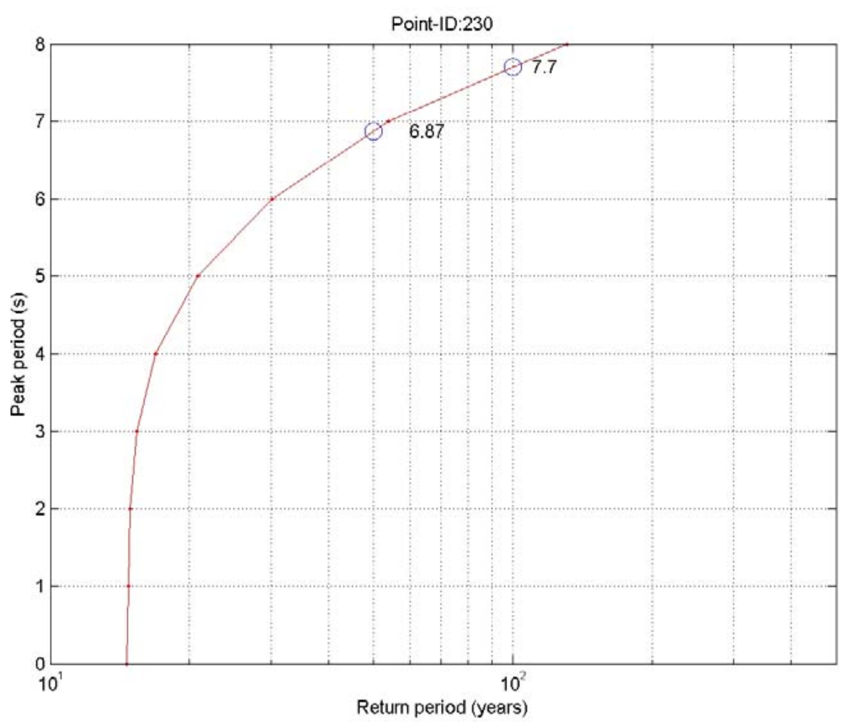


Figure 1.4 – Frequency curves of the wave height and wave period at Lake Pontchartrain (point 230) based on the STWAVE results and the JPM-OS method.

From the JPM-OS frequency analysis, 1% surge elevations, 1% wave heights, and 1% wave characteristics for existing conditions are applied in the wave run-up and overtopping calculations. These values do not consider any future changes due to factors such as subsidence and sea level rise. An additional analysis is performed representing conditions that may occur 50 years in the future and is discussed in Section 0. This future condition (year 2057) does consider changes in the surge levels and wave characteristics due to subsidence and sea level rise.

1.2.4 Wave Overtopping

Several methods are presently available for computing the wave overtopping rates. These methods can be divided into empirical methods (e.g. Van der Meer and Jansen, 1995 and Franco, 1999) and process-based methods (e.g. Lynett, 2002, 2004). Both methods are described briefly below:

> Empirical methods: Several empirical relationships are derived between the offshore hydraulic conditions (wave height, period and water level) and the levee geometry (levee height, slope) and the wave run-up and overtopping rate. These formulations are generally fitted against extensive sets of laboratory data. For levees, there are well-known relationships are formulated by Van der Meer and Jansen (1995) for wave run-up and overtopping. These relationships include the effect of berms, roughness, and wave incidence. These formulations have been incorporated in a software program (PC-Overslag) which is available on the internet at no cost (TAW, 2007)[1]. A second set of formulas developed by Franco&Franco (1999) were used to compute wave overtopping at a vertical wall. The equations were placed in an Excel spreadsheet.

> Process-based methods: In a process-based approach the run-up and overtopping rates are computed using the fundamental balance equations for mass and momentum of fluid motion. A Boussinesq model is presently the most appropriate model to compute these parameters within a reasonable time frame. The Boussinesq COULWAVE model from Texas AM was used for this report (e.g. Lynett, 2002, 2004).

Both methods have their advantages and disadvantages. The empirical methods are based on fitted curves through laboratory data, and their use is fairly straightforward. However, the disadvantage of the empirical methods is that these formulations cannot cope with very complex geometries. The basis of Boussinesq models is the governing equations of mass and momentum, and these models are able to handle more complex geometries. A drawback of these models is that they are still in an early stage of development, and the application is time-consuming. In addition, the Boussinesq model does not compute run-up and overtopping at vertical walls. As a design tool, the Boussinesq model lacks the capability to

[1] The reader is referred to the website: http://www.waterkeren.nl/download/pcoverslag.htm

execute in a production mode. Compound levee cross-sections cannot be modified iteratively in a straightforward and timely process.

It is concluded that both approaches give results within a factor of 2 - 3 if overtopping rates of 0.01 – 0.1 cfs/ft are considered. In terms of levee/flood wall heights, the differences in design elevations will be small (< 1ft).

1.2.5 Wave Forces

For floodwalls, pump station fronting protection, tie-in walls, and other vertical “hard” structures, the Goda formulation for computing wave forces is used (see e.g. USACE, 2001; part VI). A definition sketch is shown in Figure 1.5. Hydraulic inputs for these computations are the incoming wave height, wave period and the surge level. Moreover, the geometrical parameters of the structure (bottom elevation, top of wall, etc.) are inputs for this computation.

For submerged structures such as submerged breakwaters, ERDC has developed equations from measurements on a vertical wall in a straight flume physical model. There is the possibility of reflected waves in a confined basin, since his flumes tests did not consider wave amplification due to waves reflected from other vertical surfaces. Although refection would be possible under some conditions, the possibility of wave reflection was unlikely during a hurricane event when the seas were extremely disturbed. The reflected waves would need to be considered if forces during normal conditions are required.

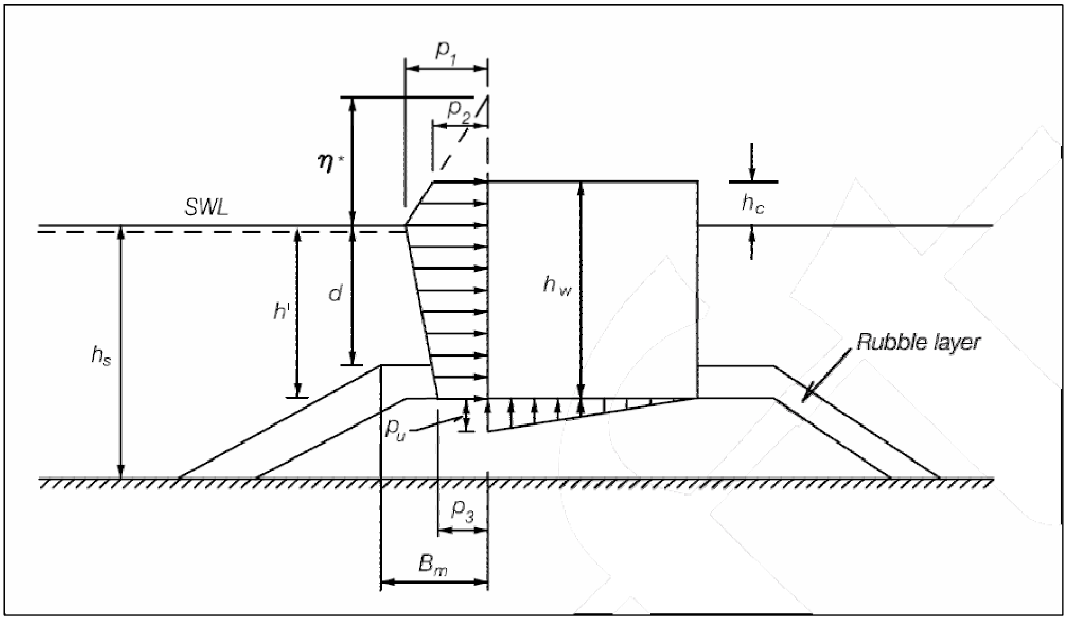


Figure 1.5 – Definition sketch of wave force calculations (source: Coastal Engineering Manual, 2001)

1.3 Step-wise Design Approach

The approach below gives a step-wise approach for determining design elevations and minimum cross sections of levees and design elevations for floodwalls. The step-wise approach is intended to be used for each section that is more or less uniform in terms of hydraulic boundary conditions (water levels, and wave characteristics) and geometry (levee, floodwall, structure). The hurricane protection reaches should be divided into segments with similar hydraulic boundary conditions, based on the JPM-OS frequency results for the water levels and wave characteristics.

Before giving an overview of the step-wise approach, several choices and assumptions in the design approach are discussed in detail. These items are:

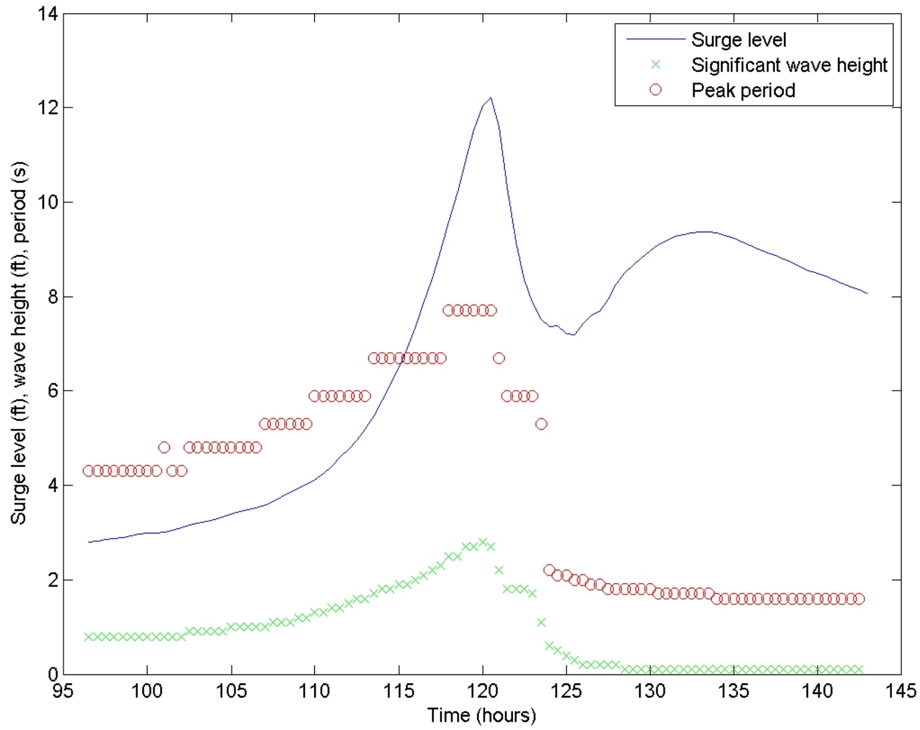
- Use of 1% values for surge levels and waves
- Simultaneous occurrence of maxima
- Breaker parameter
- Overtopping criteria
- Dealing with uncertainties

1.3.1 Use of 1% Values for Surge Elevations and Waves

The step-wise design approach below is probabilistic in the sense that it makes use of the derived 1% water elevations and 1% wave characteristics based on the JPM-OS method (see Resio et al., 2006). The procedure also includes an uncertainty analysis that accounts for uncertainties in the hydraulic parameters and the overtopping coefficients. However, the approach is not fully probabilistic because the correlation between the water elevation and the wave characteristics is not taken into account. This assumption is an important restriction of this approach. Because of this assumption the presented approach is conservative. The impact of this assumption may vary from location to location.

1.3.2 Simultaneous Occurrence of Maxima

Another assumption in the design approach is that the maximum water elevation and the maximum wave height occur simultaneously. Figure 1.6 shows time series of surge elevation and wave characteristics at two locations: Lake Pontchartrain and Lake Borgne. The plots show that the time lag between the peak of the surge elevation and the wave characteristics at both sites is small (< 1 hour). It should be noted that there are cases in which the time lag between surge and waves is a bit larger (say 1 – 2 hours). Although this assumption might be conservative for some locations, we feel that assuming a coincidence of maximum surge and maximum waves is reasonable for most of the levee and floodwall sections in our design approach.



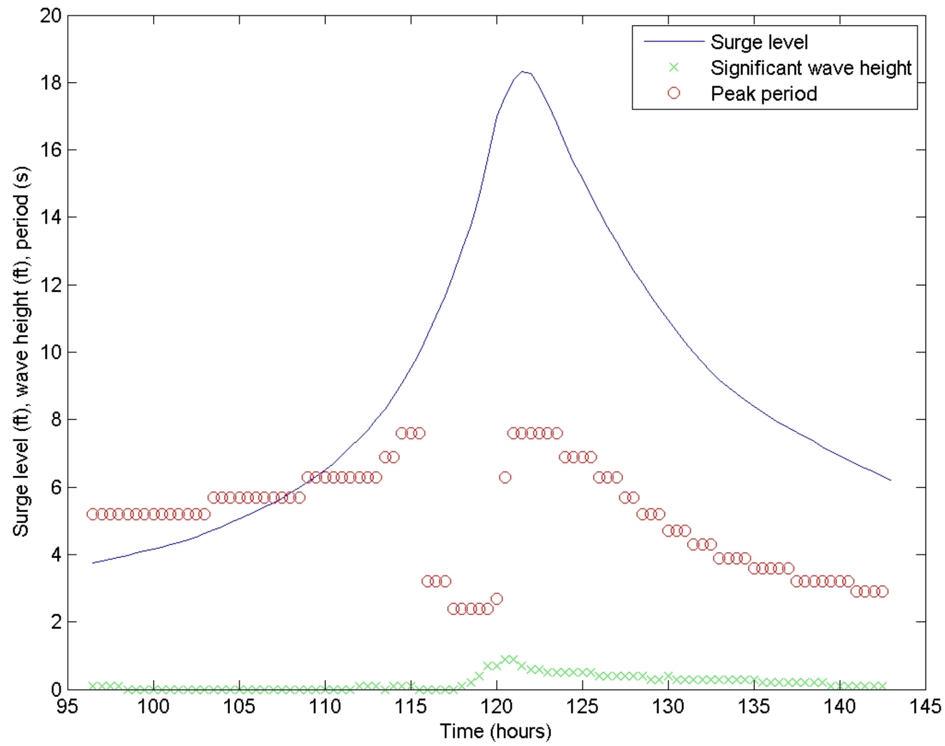


Figure 1.6 – Time histories of surge elevation and wave characteristics during storm 27 at Lake Pontchartrain (upper panel) and at Lake Borgne (lower panel).

1.3.3 Breaker Parameter

In the design approach, overtopping rates are computed using empirical formulations. One input is the wave height at the toe of the structure. This value must be estimated from the wave results from the STWAVE modeling at 600ft before the protection levee or structure. Because the foreshore is generally very shallow (same order as the wave height), wave breaking plays an important role in that 600ft. Hence, it is not likely that the wave height at 600ft in front of the levee or structure will be equal to the wave height at the toe of the levee or structure, but will be lower.

To account for breaking in front of the levee or structure, the wave height from STWAVE is reduced using a breaker parameter. The breaker parameter is the ratio between the significant wave height and the local water depth. In the literature, the breaker parameter is often a constant or it is expressed as a function of bottom slope or incident wave. A typical range for this parameter is between 0.5 – 0.78 in engineering purposes. These values are generally obtained for situations with a mild sloping bed.

Laboratory experiments (Resio, pers. comm.) and Boussinesq runs (Lynett, pers. comm.) suggest that the breaker parameter of 0.4 is a realistic choice for a relatively long shallow foreshore as it is the case for the levees and structures within the project area. Based on recommendations from ERDC, this value has been used in the entire design approach to translate the significant wave heights based on STWAVE model results in the significant wave height at the toe of the levee or structure. The peak period from STWAVE has been used without modification.

1.3.4 Overtopping Criteria

A literature survey has been carried out to underpin the value for the overtopping criterion for levees that must be used in this design approach. The survey shows that various numbers have been proposed. Experimental validation of these numbers is very limited. Typical values according to the Dutch guidelines are (see also TAW, 2002):

- 0.001 cfs/linear ft (cfs/ft) for sandy soil with a poor grass cover;
- 0.01 cfs/ft for clayey soil with a reasonably good grass cover;
- 0.1 cfs/ft for a clay covering and a grass cover according to the requirements for the outer slope or for an armored inner slope.

The literature review suggests that a 0.1 cfs/ft is an appropriate range for maximum allowable overtopping rates based on Dutch and Japanese research.

However, it is difficult to assess the adequacy of applying criteria for the New Orleans area without a good understanding of the overall quality of the levees following many different periods of construction and the effects of stresses of past hurricanes. The actual field evidence supporting these criteria is limited. After consultation with the ASCE External Review Panel, the following wave overtopping rates have been established for the New Orleans District hurricane protection system:

- For the 1% exceedence still water, wave height and wave period, the maximum allowable average wave overtopping of 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 1% exceedence still water, wave height and wave period, the maximum allowable average wave overtopping of 0.1 cfs/ft at 90% level of assurance and 0.03 cfs/ft at 50% level of assurance for floodwalls with appropriate protection on the back side.

1.3.5 Dealing with Uncertainties

The hydraulic and geometrical parameters in the design approach are uncertain. Hence, the uncertainty in these parameters should be taken into account in the design process to come up with a robust design. This section proposes a method that accounts for uncertainties in water elevations and waves, and computes the overtopping rate with state-of-the-art formulations. The objective of this method is to include the uncertainties check if the overtopping criteria are still met with a certain percentage of assurance.

The parameters that are included in the uncertainty analysis are the 1% water elevation, wave height and wave period. Uncertainties in the geometric parameters are not included; it is assumed that the proposed heights and slopes in this design document are minimum values that will be constructed. To determine the overtopping rate, the probabilistic overtopping formulations from Van der Meer are applied (see textbox below) but also the Boussinesq results could be incorporated in the method. Besides the geometric parameters (levee height and slope), hydraulic input parameters for determination of the overtopping rate in Eq. 1 and 2 are the water elevation (ζ), the significant wave height (H_s) and the peak period (T_p).

In the design process, we use the best estimate 1% values for these parameters from the JPM-OS method (Resio, 2007); uncertainty in these values exists. Resio (2007) has provided a method to derive the standard deviation in the 1% surge elevation. 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 normally distributed.

Van der Meer overtopping formulations

The overtopping formulation from Van der Meer reads (see TAW 2002):

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b\xi_0 \exp\left(-4.75\frac{R_c}{H_{m0}}\frac{1}{\xi_0\gamma_b\gamma_f\gamma_\beta\gamma_v}\right)$$

$$with \quad \max imum: \quad \frac{q}{\sqrt{gH_{m0}^3}} = 0.2\exp\left(-2.6\frac{R_c}{H_{m0}}\frac{1}{\gamma_f\gamma_\beta}\right) \qquad (1)$$

With:

q : overtopping rate [cfs/ft]

g : gravitational acceleration [ft/s^2]

H_{m0} : wave height at toe of the structure [ft]

ξ_0: surf similarity parameter [-]

α : slope [-]

R_c : freeboard [ft]

γ : coefficient for presence of berm (b), friction (f), wave incidence (β), vertical wall (v)

The coefficients -4.75 and -2.6 in Eq. 1 are the mean values. The standard deviations of these coefficients are equal to 0.5 and 0.35, respectively and these errors are normally distributed (see TAW document).

Eq. 1 is valid for $\xi_0 < 5$ and slopes steeper than 1:8. For values of $\xi_0 > 7$ the following equation is proposed for the overtopping rate:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 10^{-0.92}\exp\left(-\frac{R_c}{\gamma_f\gamma_\beta H_{m0}(0.33+0.022\xi_0)}\right) \qquad (2)$$

The overtopping rates for the range $5 < \xi_0 < 7$ are obtained by linear interpolation of eq. 1 and 2 using the logarithmic value of the overtopping rates. For slopes between 1:8 and 1:15, the solution should be found by iteration. If the slope is less than 1:15, it should be considered as a berm or a foreshore depending on the length of the section compared to the deep water wave length. The coefficients -0.92 is the mean value. The standard deviation of this coefficient is equal to 0.24 and the error is normally distributed (see TAW 2002).

The Monte Carlo Analysis is executed as follows:

1. Draw a random number between 0 and 1 to set the exceedence probability p.
2. Compute the water elevation from a normal distribution using the mean 1% surge elevation and standard deviation as parameters and with an exceedence probability p.
3. Draw a random number between 0 and 1 to set the exceedence probability p.
4. Compute the wave height and wave period from a normal distribution using the mean 1% wave height/wave period and the associated standard deviation and with an exceedence probability p.
5. Repeat step 3 and 4 for the three overtopping coefficients independently.
6. Compute the overtopping rate for these hydraulic parameters and overtopping coefficients determined in step 2, 4 and 5
7. Repeat the step 1 – 5 a large number of times (N)
8. Compute the 50% and 90% confidence limit of the overtopping rate (i.e. q_{50} and q_{90})

The procedure is implemented in the numerical software package MATLAB.

The Jefferson Lakefront levee section along Lake Pontchartrain has been taken as a reference herein to show one result of this uncertainty analysis. Table 1.1 shows the typical input needed for the Monte Carlo Analysis. It shows the input parameters for the coefficients of the overtopping formulation, the 1% hydraulic design characteristics, and the levee characteristics. Furthermore, the levee characteristics are listed such as the design height and the slope. Several test runs show that N should be +/- 10,000 to reach statistically stationary results for the 50% and 90% confidence limit value of the overtopping rate (Figure 1.7).

Figure 1.8 shows the result of the Monte Carlo analysis; overtopping rate is shown as a function of the exceedence probability. The red lines indicate the 50% and 90% confidence limit value of the overtopping rate for levees. The 50% and 90%-value of the actual overtopping rate for this specific levee section are also depicted in the plot. The result shows that the 90%-value for overtopping is below 0.1 cfs/ft and the 50%-value is below 0.01 cfs/ft, and this section meets the design criteria.

The computation of the overtopping rate in the present MATLAB routine is limited in the sense that it can only take into account an average slope for the entire cross-section. If a wave berm exists, this effect is included in a berm factor. The berm factor is adjusted in a realistic range so that the mean overtopping rate is estimated correctly compared with the result from PC-Overslag.

Table 1.1 -- Input for Monte Carlo Analysis.

Parameter	Mean	Standard Deviation	Unit	Remarks
Coefficient overtopping formula in Eq. 1	-4.75	0.5	-	Mean and standard deviation follow from TAW manual (TAW, 2002)
Coefficient overtopping formula in Eq. 1	-2.6	0.35	-	See above
Coefficient overtopping formula in Eq. 2	-0.92	0.24	-	See above
1% water elevation	9.0	0.6	ft	Values follow from JPM-OS analysis (see Resio, 2007)
1% wave height	3.6	0.4	ft	Mean value from JPM-OS analysis, standard deviation 10% of mean value based on expert judgment
1% wave period	7.7	1.54	s	Mean value from JPM-OS analysis, standard deviation 20% of mean value based on expert judgment
Levee height	16.5	-	ft	
Slope	1V:4H	-	-	
Berm factor	0.6	-	-	
Number of runs	10,000	-	-	

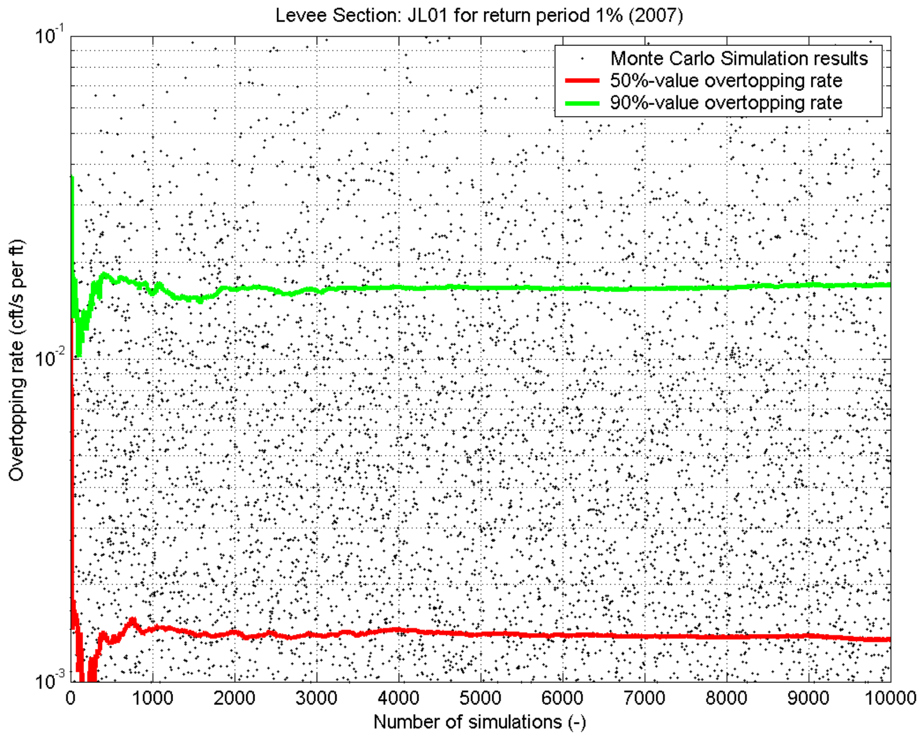


Figure 1.7 – The 50% and 90% confidence limit value of the overtopping rate as a function of the number of simulations during the Monte Carlo Analysis. The dots represent the actual results from the Monte Carlo Simulation, whereas the red and green lines represent the moving value over the number of simulations.

Notice that the uncertainty analysis described above is also implemented to compute the wave forces with different confidence levels. It makes use of exactly the same procedure, but computes the wave forces based on the Goda formulation. A Monte Carlo Simulation is performed with the water level, wave height and wave period, and the associated uncertainty, to compute the 50% and 90% assurance wave forces. Dependency between the errors in the wave height and wave period is maintained, whereas the error in the surge level and the wave characteristics are to be treated independently.

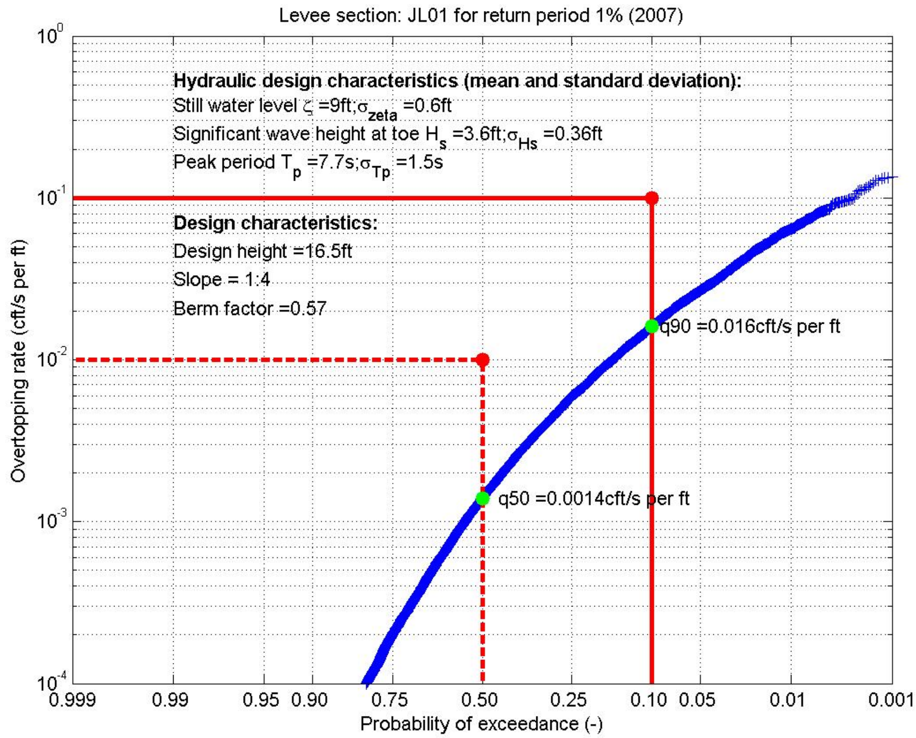


Figure 1.8 – Result of Monte Carlo Analysis for Jefferson Lakefront levee (existing conditions).

1.3.6 Step-Wise Approach

The proposed step-wise approach for design is as follows:

<u>Step 1: Water elevation</u>

1.1 Examine the 1% surge elevation from the surge frequency plots at all output points along the reach under consideration. The 1% surge elevations are the results based on the 152 storm combinations and using the probabilistic tool (JPM-OS method).

1.2 Determine the maximum 1% surge elevation for a design reach and use this number for the entire reach. The maximum is chosen to meet the design criterion at the most critical point in the section.

<u>Step 2: Wave characteristics</u>

2.1 Examine the 1% significant wave height and peak period from the frequency plots at all output points along the reach. The 1% wave heights and peak periods are the results based on the 152 storm combinations and using the probabilistic tool based on the JPM-OS method.

2.2 Determine the maximum 1% significant wave height and peak period for the reach and use these numbers for the entire reach. The maximum wave height and wave period are chosen to meet the design criterion at the most critical point in the section under consideration.

2.3 Determine if the foreshore in front of the structure is shallow. The foreshore is shallow if the ratio between the significant wave height (H_s) and the water depth (h) is small ($H_s/h > 1/3$) and if the foreshore length (L) is longer than one deep water wave length L0 (thus: $L > L_o$ with $L_o = gT_p^2/(2\pi)$). If so, the wave height at the toe of the structure should be reduced according to H_{smax} = 0.4 h. This reduction should only be applied if an empirical method is applied for determining the overtopping rate (e.g. PC-Overslag). The breaking effect is automatically included in the Boussinesq runs.

<u>Step 3: Overtopping rate</u>

3.1 Apply PC-Overslag with Van der Meer formulations (see also CEM) to determine the overtopping rates. If a wall is present, the empirical formulation of Franco&Franco (1999) will be applied. For specific complicated cross-sections, the Boussinesq lookup tables may be applied as well to compute the overtopping rate.

3.2 Determine the overtopping rate based on the 1% (average) values for the surge elevation, the significant wave height and the peak period. Use the reduced wave height in case of a shallow foreshore in the empirical approach only (e.g. PC-Overslag).

<u>Step 4: Dealing with uncertainties</u>

4.1 Apply a Monte Carlo Simulation to compute the chance of exceedence of the overtopping rate given the design elevation and slope from step 3. This method takes into account the uncertainties in the 1% water elevation, the 1% wave height and the 1% wave period. The approach is explained in detail in the next section.

4.2 Check if the overtopping rate will not exceed the design thresholds for overtopping. If yes, the design process is finished from a hydraulic point of view. If not adapt the levee or floodwall height or slope in such a way that this criterion is reached.

<u>Step 5: Resiliency</u>

For the design analysis, the overtopping rate for the 0.2% exceedence event is evaluated and both the 50% and 90% confidence limits of the overtopping rates are computed given the 1% designs. This information will be used in the entire design process to evaluate the resilience and check if armoring or other measures are necessary. This approach is still under review, and no final decisions have been made as to the use of the 0.2% event information.

1.4 Design Conditions

Two design conditions are considered in this report: existing conditions and future conditions. Both conditions are discussed below.

1.4.1 Existing Conditions

Design elevations for this scenario are considered to reflect conditions that are likely to exist in the year 2007 or year 2010. It is assumed that all levee and floodwall repairs have been made, and the interim or permanent closures and pumping stations at 17th St., Orleans Avenue and London Avenue outfall Canals are in place. The gates on the MRGO/GIWW are in place.

For most of the analysis, the existing surge elevations are based on the ADCIRC results of the 152 storm conditions for the 2007 case in conjunction with the JPM-OS method. The existing wave conditions are derived based on the STWAVE results, and are derived in a similar way. Model results from the 2010 condition were used for the analysis of the area that is affected by the MRGO/GIWW gate.

1.4.2 Future Conditions

Design elevations for this scenario are considered to reflect conditions that are likely to exist in the year 2057. Changes in surge elevations will occur in the future due to subsidence and sea level rise. Historical subsidence, projections of sea level rise, and previous studies were used to estimate future changes in surge elevations. Natural subsidence rates, including sea level rise, have been mapped by MVN for the LCA effort. Figure 1.9 shows the combined natural subsidence/eustatic sea level rise for the hurricane protection project area. The values presented in Figure 1.9 are geologic rates and do not consider any factors such as pumped drainage, which can influence regional subsidence. A relative sea level rise of 1ft over 50 years was used in the design analysis to represent future conditions in the entire area.

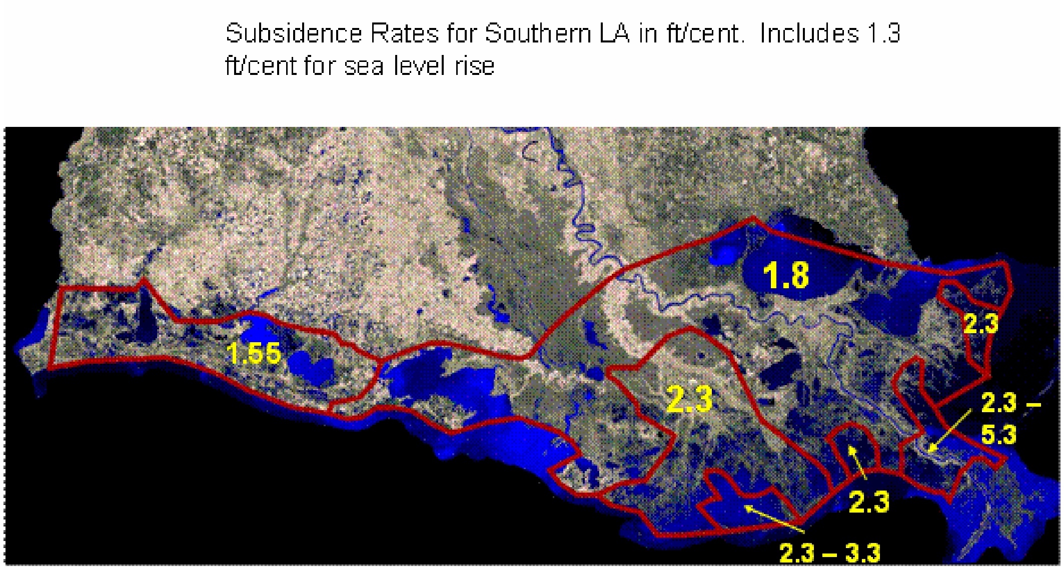


Figure 1.9 Estimated relative sea level rise during 100 year (subsidence + sea level rise)

Several ADCIRC and STWAVE model runs were performed to investigate the effect of the increasing sea level rise on surge levels and wave characteristics. These results show that:

- The surge levels increase more than proportional to increasing sea level rise (factor 1.5 to 2). A factor 1.5 implies that 1 ft sea level rise results in 1.5 ft increase of the surge level etc.
- The wave heights increase due to sea level rise. The relative effect on the wave heights is about 0.3 to 0.6 which means that 1 ft surge level results in 0.3 to 0.6 ft increment of wave height.
- The effects are not uniform in the entire area but depend on the local water depth, and geometry of the area of interest.

Based on these, the future conditions are summarized below (Table 1.2):

Table 1.2 - Future conditions for surge level and wave characteristics

Future conditions	Surge level h_{surge}		Significant wave height H_s		Peak period T_p
	$\Delta h_{surge}/\Delta h_{sealevel}$ (-)	Δh_{surge} (ft)	$\Delta H/\Delta h_{surge}$ (-)	ΔH (ft)	ΔT_p (s)
Lake Pontchartrain, New Orleans East, IHNC and GIWW, St Bernard	1.5	+1.5ft	0.5	+0.75ft	Increase by assuming unchanged wave steepness (H/T^2)
Caernarvon, West Bank	2.0	+2ft	0.5	+1ft	Increase by unchanged wave steepness (H/T^2)

Because the future condition surge elevations are derived from the surge elevations for existing conditions, uncertainty in the data and methodologies has been included. No additional value was added to address uncertainty in the increment representing subsidence, land loss, and sea level rise. The future condition surge elevation was used in wave computations, wave loads on walls and other "hard" structures, and to determine design elevations.

1.5 Design Elevations and Loads

In the design analysis, two types of flood protection are considered: soft structures (levees) and hard structures (floodwalls and other structures like pumping stations).

Levees. The design elevations are computed for both the present and the future conditions. The design elevations presented in this report only consider (relative) sea level rise for future conditions, but do not consider settlement or other structural adjustments. The design elevation recommended for levee construction at this time is the existing elevation. The levees are expected to be adapted several times during its lifetime due to settlement and changes in the hydraulic conditions should be taken into account as well.

Floodwalls and Other Structures. The recommended design elevation for floodwalls and other “hard” structures is the future conditions elevation. The recommended design elevation for floodwalls and other “hard” structures should be no less than the future condition design elevation of adjacent levees. Floodwalls and other “hard” structures will require extensive reconstruction in the future; incorporating future changes into the design of these structures now is a prudent design consideration.

The design elevations of floodwalls sometimes do include structural superiority. Structural superiority is incorporated in the design elevation for those structures that would be very difficult to rebuild, if damaged, because of disruption in services. Examples are major highway and railroad gates that require detours, pumping station fronting protection that requires reductions to pumping capacity, sector gated structures, etc. These structures are to be constructed to the 2057 levels plus 2 ft. for structural superiority. Floodwalls that can be rebuilt in areas with little or no disruption of services are to be constructed to the 2057 level.

The wave forces have been computed for the floodwalls and submerged breakwaters. These forces are evaluated for future conditions (2057). Wave forces are evaluated for two confidence levels (50% and 90%) to present the uncertainty in these numbers. At this moment, there has not been made a final decision at MVN which of these results will be used in the structural design.

1.6 Armoring

1.6.1 Introduction

Damage sustained to the levee system during Hurricane Katrina occurred primarily: (1) at transitions between earthen levees and vertical floodwall structures, (2) on the protected-side slopes of earthen levees, and (3) near the protected side base of vertical floodwalls. In May 2006, US Army Engineer Research and Development Center (ERDC), Vicksburg, MS completed an evaluation of armoring for the US Army Engineer District, New Orleans (MVN) and for Task Force Guardian (TFG). The purpose of this evaluation was to overview levee and floodwall failure modes, characterize the hydrodynamic forces that protection systems must withstand, establish initial performance criteria for protection systems, and provide an initial assessment of available armoring and protection systems.

There are four major topics relating to armoring for which guidance is required – protected side fortification of levees to minimize the effects of overtopping, frontside protection of levees from wave attack, protected side protection of walls and levee/wall transition areas, and the use of engineering solutions such as breakwaters and soil modification to modify or reduce overtopping effects.

Scour protection details and guidance used for TFG have been included in the Structrural section of this document; it is included as reference only. Proper engineering must be accomplished to ensure the best solution. There are many factors that must be considered, such as scour materials, overtopping hydraulics, and the effects of water that has overtopped on interior drainage and infrastructure.

Different materials are available for armoring. They include: Riprap; Gabions or other wire baskets filled with stone; Rock-filled wire or geogrid mattresses; Articulated concrete mattresses of interlocking blocks or blocks connected by cables; Cast-in-place, concrete-filled geosynthetic mattresses or tubes; Soil stabilizing devices designed to retain the soil within the structure such as geocells; Mattresses designed to hold vegetation in place such as "Turf Reinforcement Mats" (TRMs); and paving with asphalt or concrete. Soil reinforcement and the use of best construction materials and techniques may improve the levee's ability to withstand erosion.

1.6.2 Levee Armoring

Two essential items are needed in order to design armoring. First, it is essential to know the anticipated extreme loading for which armoring is required, and, second, it is essential to know the limits of applicability of various armoring protection systems and the upper limits of the extreme loading for which protection is desired. When both of these are known, the engineer will select the appropriate armoring that has a resistance equal or greater than the anticipated extreme loading.

The current design philosophy entails limiting the overtopping of protections that occur in the 1% event to a quantity that can be carried by typical turf covering. The more critical design condition is to provide armoring for overtopping of protections that occur in the 0.2% event. The hydraulic engineer will provide the design overtopping rates for this event. It is important to note that overflow of the system, i.e., free flow at the still water level, is not allowed for the 1% or 0.2% events. Armoring will be designed to protect from wave and over splash only.

The use of existing guidelines for stone as an armoring material clearly demonstrates the problem of lack of testing and lack of guidance on hydraulic issues related to overtopping; one such problem is the thickness of the stone vs the depth of wave runup or overtopping. For stone to withstand the magnitude of the velocities experienced during Hurricane Katrina computed by IPET on the MRGO levee, the thickness calculated using traditional methods contained in EM 1110-2-1601 is considerably larger than the depth of water. Will the overtopping continue to flow on top of the rock or be absorbed within the rock thickness? How are the velocities altered?

Revetment is presently being tested at ERDC as a potential armoring material along the MRGO levee. Anchoring the revetment is a critical issue. ERDC tests show the possibility of the revetment at the toe of the floodside slope to roll up; at the toe of the backside slope, the revetment was lifted each time a wave of water reached it.

In addition to armoring protection for all forms of overtopping, armoring protection may be needed for wave attack. Overtopping protection is for the crest and the back, or protected, side of the levee, and wave protection is for the floodside of the levee. The floodside protection for wave attack is much better documented than is the protection for overtopping. Armor stone size and riprap gradations can be obtained from the interactive version of the Coastal Engineering Manual.

ERDC found that few (if any) armoring or slope protection products have been tested at large scale for effectiveness when subjected to wave overtopping. The periodic nature of wave overtopping makes a difference between wave overtopping and steady flow overtopping. As each wave overtops, it has a forward velocity across the levee crest that likely exceeds the crest velocity of surge overtopping. Thus, unprotected soil on the levee crest that is stable for surge overtopping may erode if waves overtop. However, this flow condition is unsteady and peak velocities are sustained for only a brief time. In addition, the unsteady discharge over the crest results in a limited overtopping volume. Consequently, any erosion on the backside slope due to wave overtopping is intermittent, and probably does not progress at rates as high as what can occur for steady surge overtopping.

Without a doubt, turf is the most economical revetment material in terms of installation and maintenance. However, there are situations where turf is not strong enough to resist the erosive forces due to design conditions. The more preferable alternative is to use turf reinforcement since it has distinct advantages in terms of cost, weight, ease of installation and maintenance over other systems of armoring. When the potential erosion forces are deemed to be greater than the resistance capacity of reinforced turf, other systems such as rip-rap, articulated mats, interlocking blocks, gabions, concrete paving, etc. will be required.

However, before designing armoring for wave attack it is important to recognize how well the turf on the New Orleans Lakefront levees (LPV project) withstood wave attack. Waves of 2.5 to 3 meters were measured on the south shore of the lake in the vicinity of the new Coast Guard station just west of the 17th Street Canal. To the east, the levee is protected by the Orleans seawall but to the west in Jefferson Parish there is little protection for the levee. Along the entire Lakefront levee, there was no reported wave erosion.

The Dutch have published a technical report on the erosion resistance of grass as levee (dike) covering (TAW, 1997). In the Netherlands, waves against the outer banks of sea and lake dikes can reach heights of more than 1.5 meters. The Dutch found that very good grass mats, on a bank of slope 1:3 to 1:4 and on erosion-resistant undersoil, can withstand waves up to 1.0 meters with no serious damage after more than one day. The damage free period for waves of slightly more than 1.0 meters was shorter, but still long enough to cope with the Dutch storm flood. The underlayer was found to be important; it should always consist of adequate erosion-resistant clay, which must be at least 1 to 1.5 meters thick. Grass mats above the still water level were found to resist waves higher than grass mats in the wave breaking zone.

1.6.2.1 Turf Design

Both the Dutch and the Danes have done extensive testing of existing turf on dikes. The resistance to erosion increases with the density of root mass. The critical parameter is the dry root mass per unit area. They have also determined the best practices to increase the root mass of the turf. All of the mechanisms that are expounded by the Dutch and the Danes appear counter-intuitive at first but upon reflection make perfect sense. For example, non-fertilized turf has better erosion resistance than fertilized turf. This is because the amount of roots is the most important factor. Fertilization will produce lush greenery, but the greenery does not contribute to erosion resistance. It merely shears off in any high energy environment. Fertilization allows the roots to uptake lots of nutrients without having to extend the root mass in search of nutrients. For the same reason soils with low nutrient content produce better erosion resistant turf, since the roots have to grow and search for nutrients. A large variety of species will produce a better turf since there will be competition among the plants. The Danes categorize a turf in terms of the number of species per 25 square meters. A good dike turf will have over 20 species per 25 square meters.

Land use will influence the quality of the turf. Grazing of livestock (equivalent to our frequent mowing) does not produce the same root mass as haying. Allowing the grass to grow tall before cutting encourages deeper roots to support the taller grass. Of course the grass should be removed (as is done in making hay) for two reasons, one so that the cut grass does not suffocate the grass plants and two so that the cut grass does not compost and produce nutrients in the upper layer and thus impeding root growth.

The geotechnical lab at ERDC produced a scope of work and a cost estimate to investigate the strength of the turf on the hurricane levees in the New Orleans District. The scope included parameterizing the depth and density of the roots for various levee turfs. When this investigation gets funded, it will help District engineers to understand the limits of turf protection. This investigation will also have help to answer questions MVN-ED-H engineers have about the testing of reinforced turf mats at the Colorado State steep gradient flume facility.

In the past very little attention has been given to the production of quality turf. It is essential that the Corps begin to look at turf as the important revetment material that it is and start to implement a program along with the local sponsors to produce the best quality turf and turf management practices.

1.6.2.2 Turf Reinforcement

Turf reinforcement has four distinct advantages over any other system of levee armoring. Foremost, the turf reinforcement does not contribute any significant weight that will induce settlement or stability issues. The cost is much less than rock, or any other heavy material. Turf reinforcement can be more quickly installed than any other system. Turf reinforcement is easily maintained, it just needs to be mowed the same as turf. Riprap and gabions will eventually have trees and shrubs growing in them and properly removing them is a serious negative consideration.

For the reasons listed above turf reinforcement mats (TRM) should be given serious consideration in the effort to armor the hurricane protection levees. The only question is to determine the limits of the applicability of TRM protection. Only vigorous research can provide this much needed answer.

1.6.3 Walls and Levee Transitions

Floodwalls that may be overtopped by rising water should be designed with erosion protection on the protected side capable of resisting the force of the free-falling water jet. Equations are available to compute the location where the free-falling water jets hits the ground on the backside of the wall. This location is dependent on the height of the wall and the surge height above the wall. ERDC found that these equations may under estimate the distance. The protection coverage must extend away from the wall beyond this location to account for the hydraulic jump that will form when the flow changes from supercritical to subcritical as well as uncertainty in the computation. Where overtopping is from waves only, the unsteady discharge will be a function of wave height, wave period, and surge elevation relative to the wall. Erosion of unprotected soil will occur as the waves cascade over the wall, but the unsteadiness of the process, coupled with the variation of impact point due to irregular waves, makes scour estimation difficult, if not impossible.

For transition areas, as indicated in the ERDC report, simple analytical methods for estimating the increased flow velocities that occur at transitions are lacking, and most likely either physical modeling or sophisticated numerical simulations will be required to establish flow velocities due to surge overtopping in the vicinity of levee/floodwall transitions. However, some insight into the overtopping problem can be gleaned by looking at results obtained from two-dimensional inviscid jet theory. Based on discharge contours, the flow velocity along the outer edge of the jet is about 1.64 times the flow velocity through the middle of the gap. Therefore, it is easy to see that the region immediately adjacent to the vertical wall experiences the largest flow velocity. The addition of waves propagating on top of the overtopping surge compounds the complexity of the flow situation, and no simple procedures are available to address this case. Laboratory testing will be the best tool for examining the stability of armoring alternatives subjected to water and wave overtopping at levee transitions.

1.6.4 On-going Studies

ERDC Coastal and Hydraulics Laboratory has completed field study of the effects of the 2005 hurricanes on the hurricane and storm damage reduction system. Their findings are summarized in the report, “Protection Alternatives for Levees and Floodwalls in Southeast Louisiana: Phase One Evaluation.” Although the document is still a draft, Chapter 4, “Protection for Overtopped Floodwalls,” is included as an appendix to these guidelines for information only.

Phase Two of the study, which is to provide physical modeling and recommendations for design of overtopping and scour protection, has not been completed. That information will be incorporated into these guidelines as soon as it is available.

Task Force Hope has commissioned an Armoring Team to provide guidance on the use of existing technologies for armoring and to more rigorously investigate armoring design and methods for future use. Engineering Division Hydraulics Branch has also chartered a team to investigate ways to provide resiliency for levees and walls that are overtopped by events exceeding design conditions. This effort includes plans to perform a field test of a levee subjected to overtopping forces. Input from these two teams will guide future design work and design guidance.