ELEVATIONS FOR DESIGN OF HURRICANE PROTECTION LEVEES AND STRUCTURES

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY HURRICANE PROTECTION PROJECT

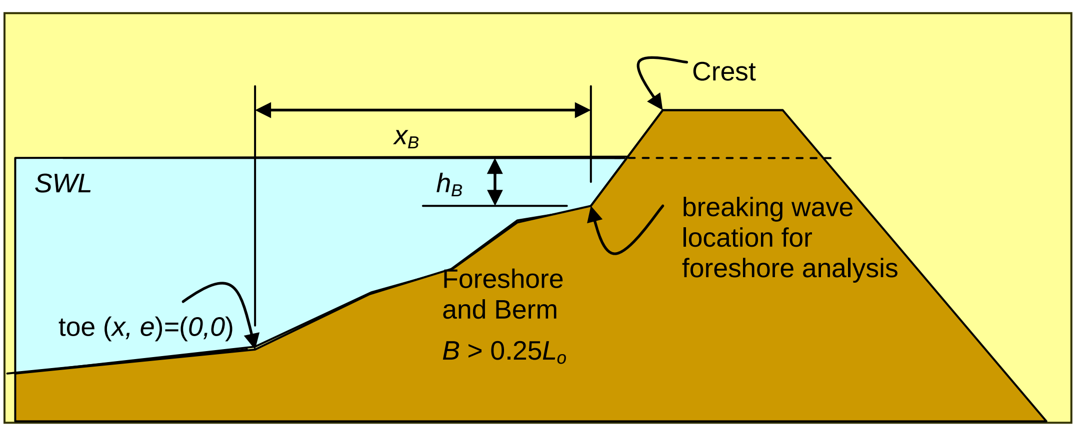
WEST BANK AND VICINITY, HURRICANE PROTECTION PROJECT

REPORT

Prepared by

US Army Corps of Engineers®

New Orleans District
09 October 2007



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Executive Summary

This report, Elevations for Design of Hurricane Protection Levees and Structures Lake Pontchartrain, Louisiana and Vicinity Hurricane Protection Project and West Bank and Vicinity, Hurricane Protection Project, provides a detailed documentation of the coastal and hydraulic engineering analysis performed to determine the 1% project design elevations for these two hurricane protection projects. The report has been prepared to provide levee and structure elevations so that the U.S. Army Corps of Engineers (USACE) can initiate detailed design and construction as described in the 4th Supplemental Appropriation, Public Law 109-234 of the One Hundred Ninth Congress:

> ….at least $495,300,000 shall be used consistent with the cost-sharing provisions under which the projects were originally constructed to raise levee heights where necessary and otherwise enhance the existing Lake Pontchartrain and Vicinity project and the existing West Bank and Vicinity project to provide the levels of protection necessary to achieve the certification required for participation in the National Flood Insurance Program under the base flood elevations current at the time of this construction….

The report presents 1% project design elevations sufficient to provide protection from a hurricane event that would produce a 1% exceedence surge elevation and associated waves. After construction is complete in 2011, the hurricane protection systems will meet the hydraulic requirements for levee certification, as documented in draft Engineering Technical Letter (ETL), Engineering and Design, Certification of Levee Systems, for the National Flood Insurance Program (NFIP), which is still in the developmental stage.

The elevations presented in this report should be considered initial elevations. Elevations are appropriate for design of some levee/floodwall segments which will not be affected by subsequent studies which might further modify the system 'footprint' enough to require reanalysis of the levee grades for that specific segment. More thorough engineering investigations will follow to determine final construction elevations on many segments of the system. Additional studies may be performed to evaluate alternatives. The designers may evaluate new alignments, change a levee to a floodwall, change levee cross sections, add breakwaters, incorporate armoring, and other measures that can change the parameters used to calculate the design elevations. Ongoing investigations include evaluation of incorporating the 40 Arpent and Maxent levees into the federal levee system, system analysis of the Mississippi River Gulf Outlet/ Gulf Intracoastal Waterway (MRGO/GIWW) gates, levee, and floodwalls, comparison of three sector gate alternatives for the Harvey and Algiers Canals, alternative alignment studies of the levee reach in the vicinity of Davis Pond, and other analysis.

Hydraulic design and analysis associated with upcoming investigations will be documented in engineering analysis reports and also in addenda to this 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.

New Processes and Procedures

For the coastal and hydraulic engineering analyses, new processes and procedures were formulated. A team of USACE, Federal Emergency Management Agency (FEMA), National Oceanic and Atmospheric Administration (NOAA), private sector, and academia developed a new process for estimating hurricane inundation probabilities, the Joint Probability Method with Optimal Sampling (JPM-OS). Results are being applied to USACE work under the 4th supplemental appropriation, Interagency Performance Evaluation Team (IPET) risk analysis, Louisiana Coastal Protection and Restoration project (LACPR), and FEMA Base Flood Elevations for production of digital flood maps for coastal Louisiana and Texas. The USACE 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. Additional information can be found in Chapter 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.

A team of USACE, academia, and Dutch experts developed a step-wise approach to determining design elevations based on a probabilistic analysis of wave overtopping rates. This analysis incorporates the uncertainties associated with the coastal parameters used to compute overtopping rates. A similar methodology has been developed using Goda formula to compute the wave forces with different confidence levels. The step wise approach is described in detail in Chapter 2. The step wise approach will be incorporated into Design Guidance prepared by the New Orleans District.

Criteria for wave overtopping thresholds were established in consultation with the American Society of Civil Engineers (ASCE) External Review Panel. The USACE Engineering Research and Development Center (ERDC) evaluated overtopping criteria and prepared a paper, “Evaluation of Permissible Wave Overtopping Criteria for Earthen Levees without Erosion Protection”, found in Appendix E.

An extensive USACE/FEMA internal review and ASCE external review has been conducted during the period March through August 2007. Consultation with ASCE external review members and USACE experts began much earlier in the design process. Comments have been incorporated into this report. 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).

IPET Findings and Application to the Design Elevations

As documented in the IPET report, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, Draft Final Report of the Interagency Performance Evaluation Task Force, Volume 1, Executive Summary and Overview, there were three overarching findings and recommendations:

1. The hurricane protection system in New Orleans did not perform as a system. IPET findings indicated it was important that all components have a common capability based on the character of the hazard they face.
2. Redundancy should be a component of the system.
3. Consideration should be given to the performance of the system if the design event or system requirements are exceeded.

A systems approach was used in the coastal and hydraulic engineering analyses. Surge and wave models were inclusive of the protection system area. Analyses included the evaluation of the effects of subsidence and sea level rise on surge elevations and waves. Construction of the hurricane protection system to the design elevations and cross sections in this report ensures that the components have a common capability based on the hazard.

Redundancy has been included in the system. The existing levee/floodwall system in the GIWW/IHNC and along the outfall canals will provide a useful measure of redundancy to the flood risk reduction system behind the primary line of protection such as the MRGO/GIWW gates, Seabrook gate, and the permanent outfall closures and pumps.   Sector gage alternatives for the Harvey and Algiers Canal will also have some levee/floodwalls along the interior drainage outlets that can provide a measure of redundancy.

Consideration has been given in the analyses to resiliency, the performance of the system if the design event or system requirements are exceeded. The USACE must be in a position to ensure that the 2011 system is resilient to severe hurricanes both now and into the future. Resiliency research facilitates the USACE to build better levees. Incorporation of resiliency into levee design will build trust in the community. Additional information on integrating resiliency into the hurricane protection system can be found in Chapter 6.

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1 Introduction

1.1 Background

The purpose of this report is to document the analysis performed by the New Orleans District 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% design elevations, have been developed for two authorized hurricane protection projects in the New Orleans area: Lake Pontchartrain, LA & Vicinity; and West Bank & Vicinity.

In September 2006, a preliminary analysis was performed by the New Orleans District to provide initial design elevations for ongoing design and evaluation of the protection system. This work was in advance of the completion of modeling and analysis performed jointly by the USACE/FEMA modeling team. The modeling work has advanced to sufficient completion for use in design. This report provides design elevations based on this advanced modeling effort.

This report presents the hydraulic design elevations for conceptual design of levees, floodwalls, breakwaters, seawalls and structures for the Lake Pontchartrain, LA & Vicinity; and West Bank & Vicinity projects. This chapter gives a description of the area (Section 1.2). Next, it discusses the intent of the design for the Hurricane Protection System (Section 1.3). This chapter closes with an outline of the report (Section 1.4).

An extensive USACE/FEMA internal review and ASCE external review was conducted during the period March through August 2007. Comments have been incorporated into this report. 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).

1.2 Description of Project Area

The Lake Pontchartrain, LA & Vicinity; and West Bank & Vicinity projects are shown in Figure 1 and Figure 2, respectively. The Lake Pontchartrain, LA & Vicinity project is designed to provide hurricane protection for residents between Lake Pontchartrain and the Mississippi River levee. The West Bank and Vicinity Project is designed to protect the urban area from Lake Cataouatche to Oakville, Louisiana along the west bank of the Mississippi. The majority of the parishes of Orleans, Jefferson, St. Bernard, St. Charles, and Plaquemines lie within the Hurricane Protection System.

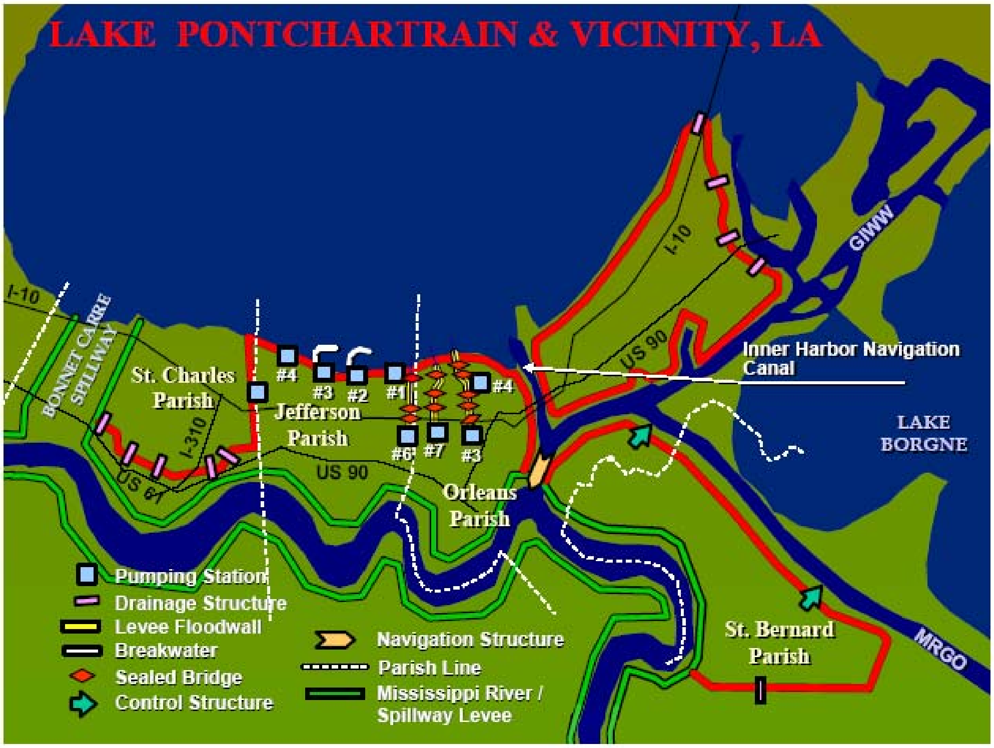

Figure 1 – Lake Pontchartrain, LA and Vicinity

Figure 2 – West Bank and Vicinity

1.3 Design Intent

The design intent for the Hurricane Protection System has several major components:

- Levee/structure design height
- Risk based analysis
- Levee/structure survivability
- Interior Structures/Pump Stations
- Subsidence
- Future Conditions
- Time Frame
- Monitoring and Maintenance

Levee/Structure Design Height
The protection system design elevations are sufficient to provide protection from a hurricane event that would produce a 1% exceedence surge elevation and associated waves. The design elevations presented in this report are determined using the 1% annual exceedence still water elevation, 1% annual exceedence wave height, and 1% annual exceedence wave period, and assume simultaneous occurrence of maxima of surge level and wave characteristics. These assumptions are conservative and are in line with a resilient design approach (IPET, 2007).

Design criteria for the levees and structures elevations also consider wave overtopping limits. Guidelines for establishing the overtopping rate threshold (i.e., the threshold associated with the onset of levee erosion and damage) for different types of embankments can be found in Engineering Manual (EM) 1110-2-1100 (Part VI), Table VI-5-6. These threshold values are consistent with those that are adopted by the Technical Advisory Committee on Flood Defence in the Netherlands (TAW 2002). After consultation with the ASCE External Review Panel, the following wave overtopping rates have been established for the New Orleans District hurricane protection systems:

- 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.

This report is neither the final nor complete guidance in the design of the hurricane protection system. More thorough investigations will follow to determine final construction elevations, and other studies and criteria will be applied to assure the safety and reliability of the total system.

The elevations resulting from this analysis and presented in this report are called initial design elevations with the clear understanding that the elevations used for design may vary from this report. This is because a number of general assumptions regarding the geometry of each reach may change. 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 documented in subsequent design reports.

Unless otherwise noted, all elevations presented in this report are in ft NAVD88 2004.65.

Risk Based Analysis
In the mid-1990s, USACE adopted a risk analysis approach for flood damage reduction project development. That policy, Engineering Regulation (ER) 1105-2-101, Risk Analysis for Flood Damage Reduction Studies, was updated in January 2006. Risk analysis explicitly, and analytically, incorporates consideration of uncertainty of parameters and functions used in the analysis to determine the undesirable consequences. Uncertainty is defined here as a measure of the imprecision of knowledge of variables and functions. Uncertainty may be represented by a specific probability distribution with associated parameters, or sometimes expressed simply as standard deviation.

Present guidance supplements freeboard by providing upper and lower bounds of required levee performance based on specified levels of assurance of protecting against the design flood. Levee and floodwall performance here is defined as providing assurance. As stated above, the design criteria are that the wave overtopping rate does not exceed 0.1 cfs/ft with 90% assurance. Furthermore, it does not exceed 0.01 cfs/ft with 50% assurance for grass-covered levees and 0.03 cfs/ft for floodwalls with appropriate protection on the back side. A probabilistic approach is used in calculating wave overtopping that incorporates uncertainty in the still water elevation and wave characteristics.

In April of 1997, two policy letters addressing levee certification determinations were issued. The first letter, *Guidance on Levee Certification for the National Flood Insurance Program*, dated April 10, 1997, was issued to ensure consistency throughout USACE with the application of the policy to levee certifications. This letter was updated and reissued with the policy letter, *Guidance on Levee Certification for the National Flood Insurance Program – FEMA Map Modernization Program Issues*, dated June 23, 2006. The emphasis in this updated letter and attachments describes USACE policy in the area of freeboard criteria by providing a performance target that is statistically based, reflecting stream profile variability and uncertainty.

Use of a risk based approach in the design of the hurricane protection system ensures that the design elevations meet certification requirements.

Levee Survivability – Resilience
IPET identified resilience as one of the "Overarching Lessons Learned" from Hurricane Katrina. Engineers are working to develop guidance to define resiliency and the level of resilience needed for levees and structures. Resiliency is herein briefly defined as the ability of the levee or structure to provide protection during events greater than the design event without total failure.

The minimum criteria for resilience must be that levees and structures do not catastrophically breach when design criteria are exceeded. Resilience also includes designing for possible changes in conditions, with the flexibility to adapt to future design conditions. For urban areas such as the New Orleans Metro area, 0.2% annual exceedence event is considered as an appropriate minimum level of evaluation for resiliency. Surge, wave heights, and overtopping rates for the 0.2% exceedence event are included in the report.

Additional research and modelling is needed to establish resiliency guidance. The impact of resilience criteria will be factored into the overall planning and design process.

Structures / Pump Stations

Pump stations throughout the New Orleans area have been constructed and are operated and maintained by local government agencies. There are no Federal pump stations in the hurricane and storm damage reduction system of greater New Orleans. Prior and present hurricane protection projects do not rely significantly on the ability to pump out water from rainfall and overtopping of levees and walls.

In urban and urbanizing areas, provision of a basic drainage system to collect and convey local runoff from rainfall is usually considered a non-Federal responsibility. Within the New Orleans area, however, there is a Federal project to improve interior drainage, the Southeast Louisiana Urban Flood Control Project.

Recognizing the damage that may result from a weakened or inoperable storm drainage system, the New Orleans District is working on several authorized features to reduce the consequences of interior flooding. They include:

- Completion of the Southeast Louisiana Urban Flood Control Project, a Federal project to improve interior drainage in New Orleans and surrounding communities.
- Design and construction of positive shut-off gates at pump stations to block backflow.
- Providing fronting protection at pump stations to improve resilience and survivability of pump stations through storm surge events.
- Storm proofing selected pump stations to improve discharge capabilities during storm events.

Subsidence

Planning for anticipated subsidence, both short-term and long-term, is included in the design of the hurricane and storm damage reduction system. During the design of individual reaches, geologists and geotechnical engineers will examine site-specific soil conditions and estimate long-term settlement and subsidence in the barriers. For levees over soft foundations, engineers typically recommend construction in several lifts. This allows the foundation soils to consolidate and gain in shear strength. When future lifts are constructed to higher elevations, the footprint of the levee system does not need to increase. Final construction lifts are typically constructed with a foot or more of added height in anticipation of long-term settlement. This added height assures that the levee crown elevation will be at or above the design elevation.

Future Conditions

Design elevations were calculated for both existing conditions and future (2057) conditions. Existing conditions represents conditions that will exist with the completion of 100-year system, scheduled for 2011. Future conditions include changes in still water levels and wave characteristics due to subsidence and sea level rise. Historical subsidence, projections of sea level rise, and previous studies were used to estimate future changes in still water level. Natural subsidence rates, including sea level rise, have been mapped by the New Orleans District for the Louisiana Coastal Area (LCA) study. A relative sea level rise of 1ft over 50 years was used for the purposes of this report. As noted in Section 2.6.2 of the report, the effect of increasing sea level rise on surge levels was further investigated and results in the 1.5 to 2.0 ft increase applied as future conditions. Moreover, the wave characteristics were also corrected for the increasing water depth.

The New Orleans District is also planning regular reassessment of design parameters in order to assure the effectiveness of the system in future years. Changes in sea level and land loss are some of the factors that need to be periodically revisited. The system should also undergo a reassessment after major events or significant changes in design and analysis methodologies. The need for a post-authorization change should be addressed after each reassessment. The intent is to conduct such reviews no less than once every 10 years.

Time Frame

It is the publicly stated goal of the New Orleans District to provide a complete system of hurricane and storm damage reduction barriers to provide a 1% annual exceedence event level of protection to the greater New Orleans area by the 2011 hurricane season. Some polders, which are already very nearly at the 1% annual exceedence level of protection, will get there sooner.

Within weeks of Hurricane Katrina, FEMA issued Advisory Base Flood Elevations (ABFE) in the greater New Orleans area as they reassess their flood insurance maps. Local municipalities are required to enforce the ABFE requirements as a condition of receiving federal aid and the state is requiring compliance by homeowners as a condition of the “Road Home” grant program. The USACE is working with FEMA to revise the inundation estimates to be used to establish new Base Flood Elevations for the region.

Monitoring and Maintenance

At a minimum, levees are inspected and maintained according to FEMA regulations contained in 44CFR65.10(d), Maintenance plans and criteria. (Attachment 733) This federal regulation requires formal and regular documentation attesting to the “stability, height and overall integrity of the levee and its associated structures and systems.”

Once initial construction is completed, the responsibility to operate, maintain, repair, replace and rehabilitate barriers is turned over to the local sponsor in most cases. Periodic inspections and annual reviews submitted to the USACE will assure proper performance. To ensure requirements are well understood, an Operations and Maintenance manual will be developed for each project and serve as the basis for future monitoring, inspection and reporting.

In addition, the USACE will conduct periodic surveys of levees as part of an improved quality assurance program. While there is no “guarantee” of funding for most federal programs, we fully expect funding will be available for the periodic monitoring and necessary maintenance. This is provided by Inspection of Completed Works, an annual line item in the federal budget.

1.4 Report Organization

A description of the design approach to determine the design elevations is given in Chapter 2. The design approach includes the use the 1% surge elevations and wave characteristics that have been derived using the recently developed probabilistic method (JPM-OS method). Furthermore, two design scenarios are defined in this chapter: existing conditions and future conditions. Both scenarios are applied during the design process. Chapters 3 and 4 present the resulting design elevations for each of the areas and locations under consideration: Lake Pontchartrain, LA and Vicinity and West Bank and Vicinity, respectively. Chapter 5 summarizes the derived design elevations in tables for the areas and locations. Chapter 6 presents conclusions and recommendations as well as issues that are not resolved yet in this report. For the convenience of the reader, generic procedures and methods are reported in the appendices.

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2 Design Approach

2.1 General

This chapter presents the design approach for the levee and structure design elevations and cross-sections. The outline of this chapter is as follows. Section 2.2 provides an overview of the modeling, frequency analysis, and methods used in the determination of the 1% design elevations. Section 2.3 presents the step-wise methodology for the determination of the 1% design elevations. Section 2.4 and Section 2.5 contain two examples (Jefferson Lakefront and MRGO levee) of this design approach. The design conditions are discussed in Section 2.6 and the design products of this report are summarized in Section 2.7. Section 2.8 contains concluding remarks.

2.2 Input Data and Methods for Design Approach

2.2.1 JPM-OS Process

In 2006 and 2007, a team of USACE, 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). 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 (Resio, 2007). This work was initiated for the Louisiana Coastal Protection and Restoration study (LACPR), but now is being applied to USACE 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 USACE 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 3.

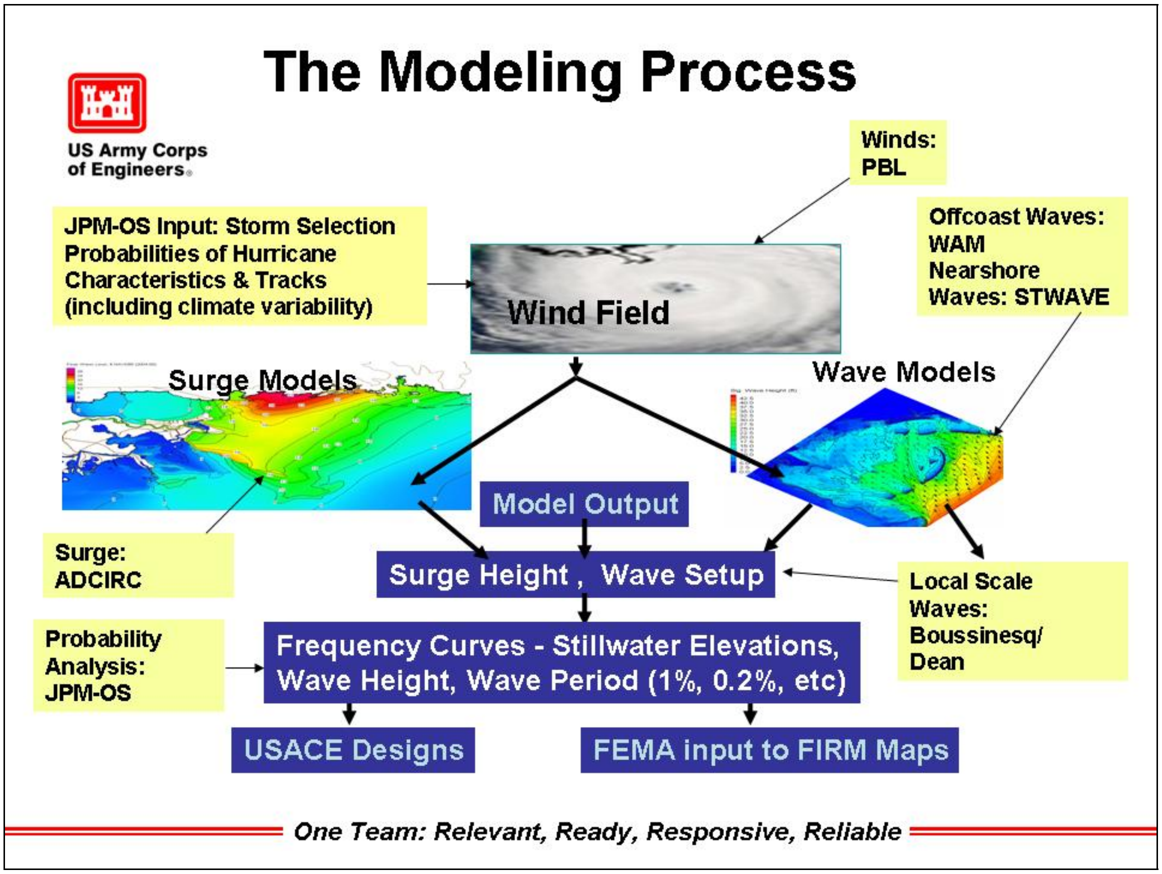


Figure 3 – The different components and their interaction in the JPM-OS Process

2.2.2 Modelling Process

The following models were 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 was 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 was 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 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 USACE Coastal and Hydraulics Laboratory (CHL) at ERDC in Vicksburg, MS. WAM was 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 was used to generate the nearshore wave heights and wave periods using boundary conditions from the WAM modeling. The WAM-to-STWAVE procedure was applied for each storm. For the design purposes, the STWAVE model did not include frictional effects. Additional discussion on the STWAVE model is contained in Chapter 6.

The JPM-OS modeling process is as follows (see also Figure 3). The PBL model was used to generate the wind fields required in the JPM-OS process. For each storm, the PBL model was used to construct 15-minute snapshots of wind and pressure fields for driving the surge and wave models. ADCIRC, WAM, and STWAVE model runs were performed on high speed computers at ERDC, the Lonestar computer at University of Texas, and similar computers. With all major rivers already “spun up”, the surge model ADCIRC was initiated assuming zero tide. The spectral deep water wave model WAM was 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 was used to produce the wave fields and estimated radiation stress fields. These stress fields, added to the PBL estimated wind stresses, were 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 were modeled with ADCIRC/STWAVE for design purposes: 2007 condition and 2010 condition. The **2007 condition** considered the interim gates and closures at the three outfall canals and levees and floodwalls constructed to pre-Katrina authorized elevations. The **2010 condition** considered 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 condition, no gate was 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 were modeled for the 2010 condition. For the 2010 condition, the output from the 56 storms was used with 96 storms from the 2007 condition to create a dataset of 152 storms required for the frequency analysis. A relationship was determined from the two sets of conditions and applied to achieve a consistent dataset.

The 2007 results from ADCIRC and STWAVE were used for Lake Pontchartrain Lakefront area and the West Bank. This area is not affected by the gates at MRGO/GIWW. The 2010 model results used for the analysis of the MRGO/GIWW 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, the levee/floodwall sections of the GIWW and IHNC inside the gate with no Seabrook Gate were also designed with the ADCIRC results.

A special remark is made regarding the STWAVE results. As stated above, the STWAVE results in this design analysis do not consider friction. Sensitivity runs with the STWAVE model show that a run with and without friction can result in differences in wave heights of 3 ft or more for the same storm. Figure 4 through Figure 6 illustrate the differences in model output with and without friction. Figure 4 shows the location of several output points in the SWTAVE models. Figure 5 and Figure 6 show the wave heights for Storm 15 at point 10 from STWAVE with friction and STWAVE without friction.

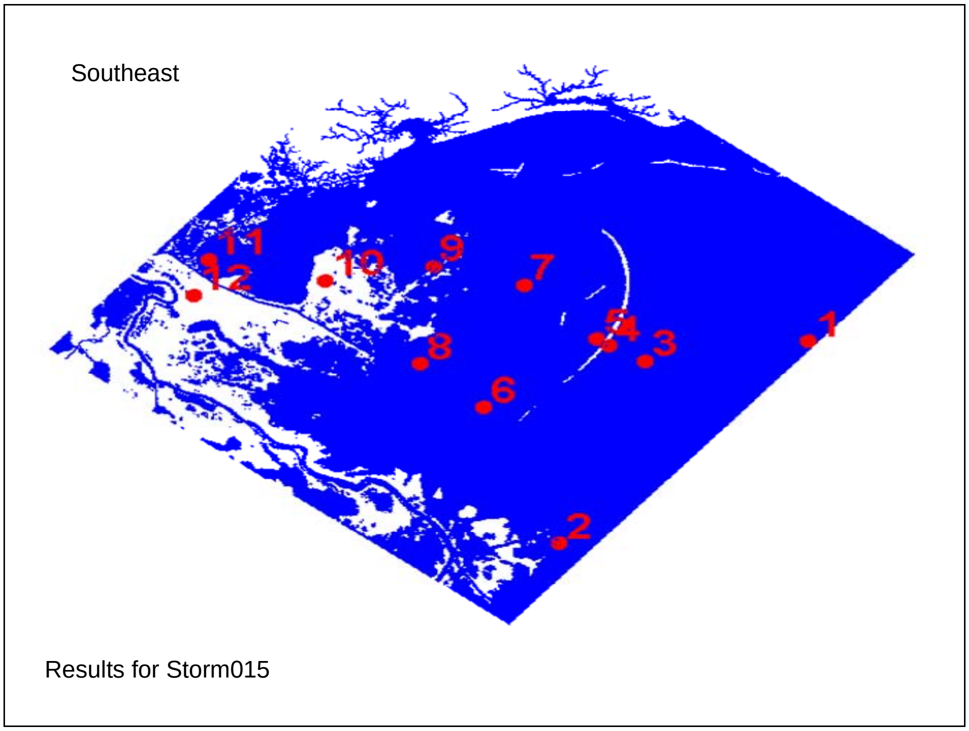


Figure 4 Locations of output points for STWAVE.

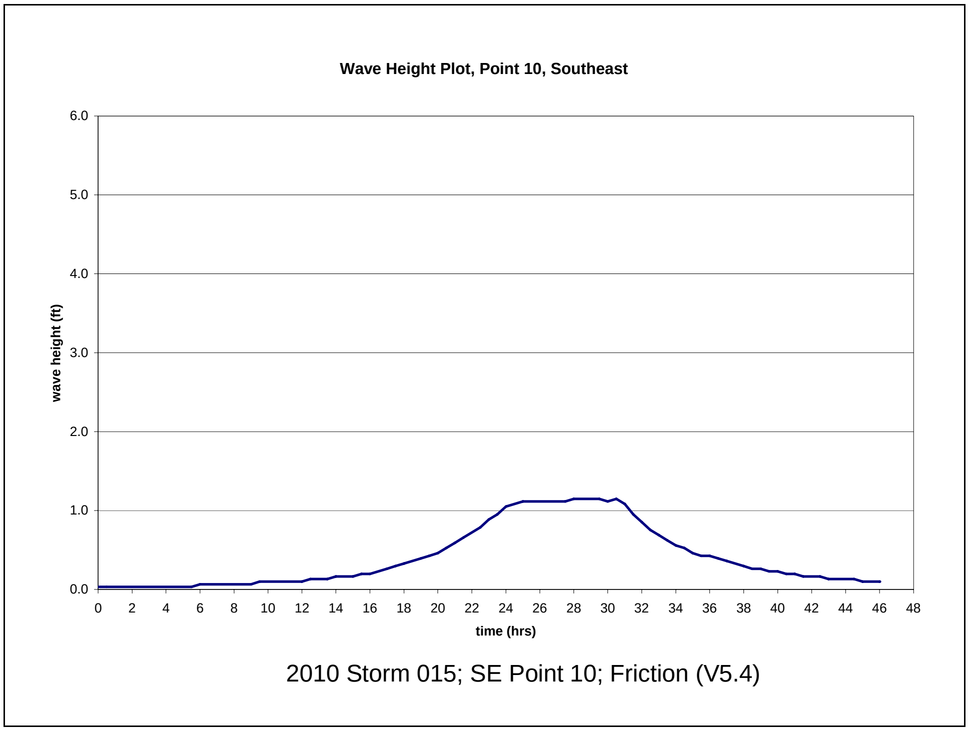


Figure 5 STWAVE model with friction

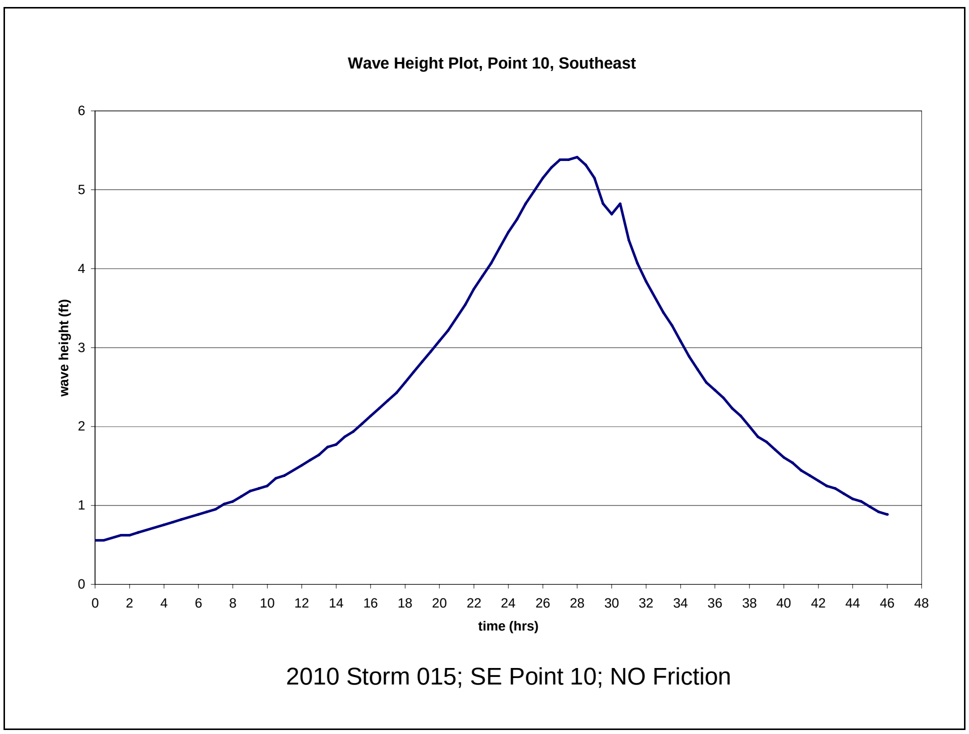


Figure 6 STWAVE model without friction

ERDC has run the Katrina wind fields in the Lake Pontchartrain STWAVE model with friction, to determine the effect of friction on wave climate in the lake and in the marshes of St. Charles Parish. The results show only small changes in the waves in Lake Pontchartrain and differences on the order of 1 to 2 ft in the marshes of St.Charles Parish. Furthermore, preliminary results from the LACPR work indicate that the magnitude of the difference in design elevations as a result of the lower wave height can be as much as 4 to 6 ft when extensive marsh vegetation exists in front of the levee system (e.g. Caernarvon to Verret levee).

ERDC experts have indicated that how the landscape interacts with the waves is an area where research is needed. Until there is good wave data in for coastal Louisiana, models that use friction will overestimate the effects of vegetation on wetlands. Another aspect is that not known is what the wetlands will be in the future. At present, there is no authorization to maintain coastal features. Further, use of science where there is no agreement among the experts and there is so much scientific uncertainty does not make sense for detailed designs.

Based on these considerations, the wave results without friction have been applied in this design study. Use of the STWAVE results without friction for the 1% design elevations results in a conservative design. Evaluation of waves can become part of a continued evaluation. Additional information regarding future research is given in Chapter 6.

2.2.3 Frequency Analysis

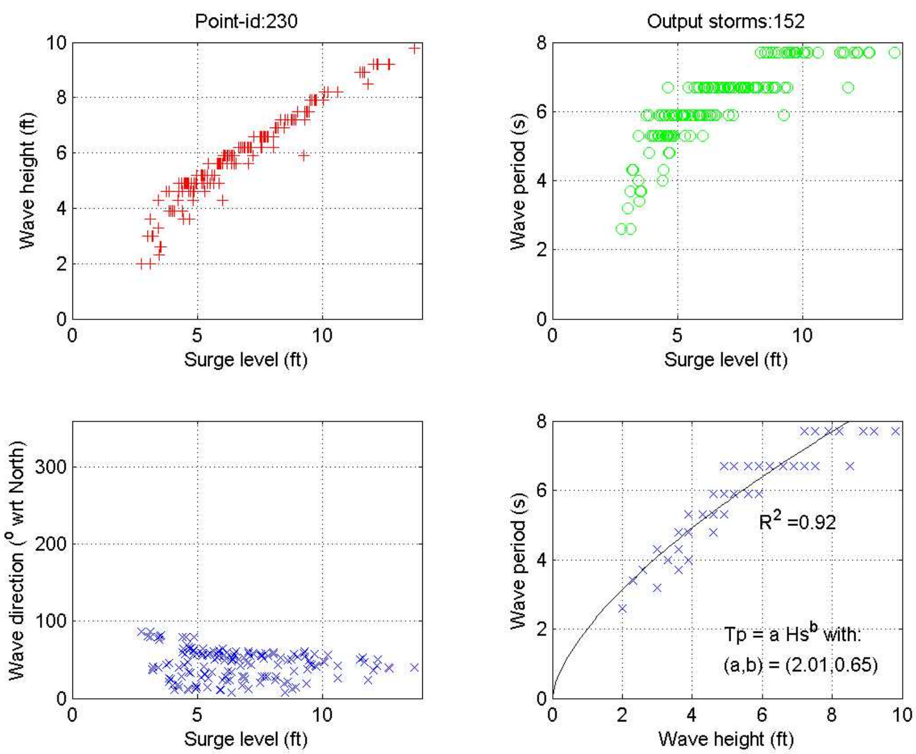
The output from the ADCIRC and STWAVE models used in the frequency analysis are the maximum surge elevation and maximum wave characteristics (significant wave height, peak period, and wave direction) at approximately 600 ft in front of the levee or floodwall. Typical parameters which are to be computed 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 2.2.4.

An example of the model output at two locations within the hurricane protection system is shown in Figure 7. 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 was 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 were estimated from the ADCIRC output of the 152 storms for 2007 and 2010 conditions. Along the West Bank, there were instances where there was no output from the 152 storms. In this case, estimates were 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. The errors were generally in the order of 1 – 2 ft for the 1% surge levels.

The same methodology was also used to develop frequency curves for wave height and wave period. Examples of frequency curves can be found in Figure 8. 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.



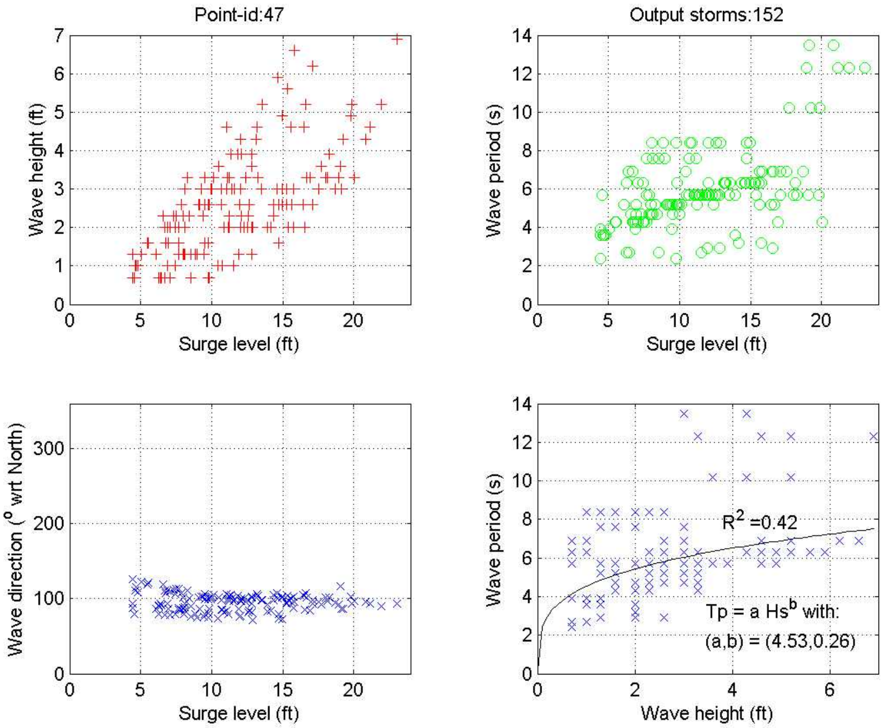
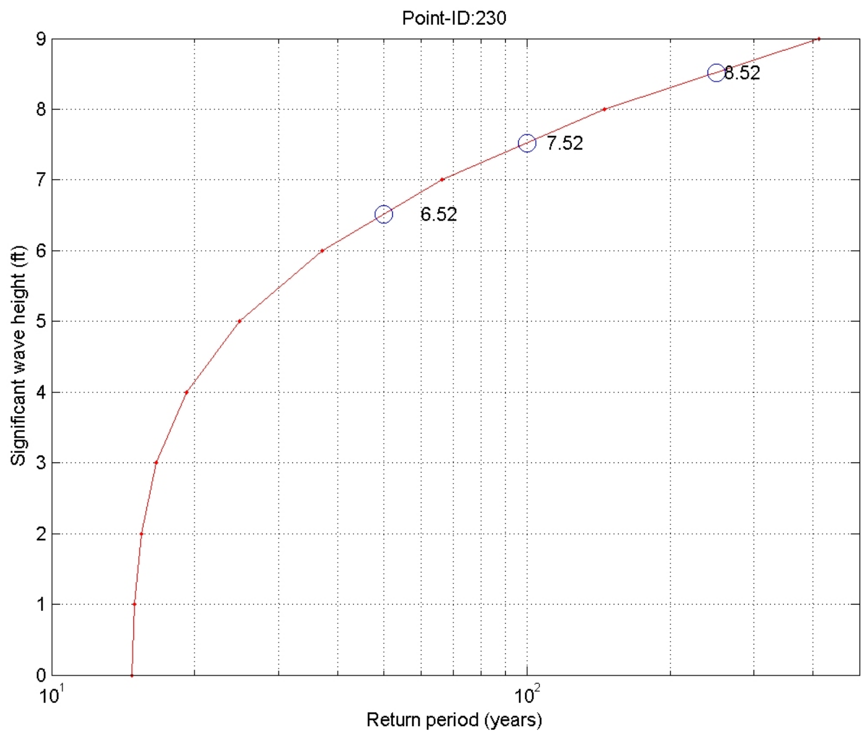


Figure 7 – Numerical results at Lake Pontchartrain (upper panel) and MRGO (lower panel) from ADCIRC and STWAVE.



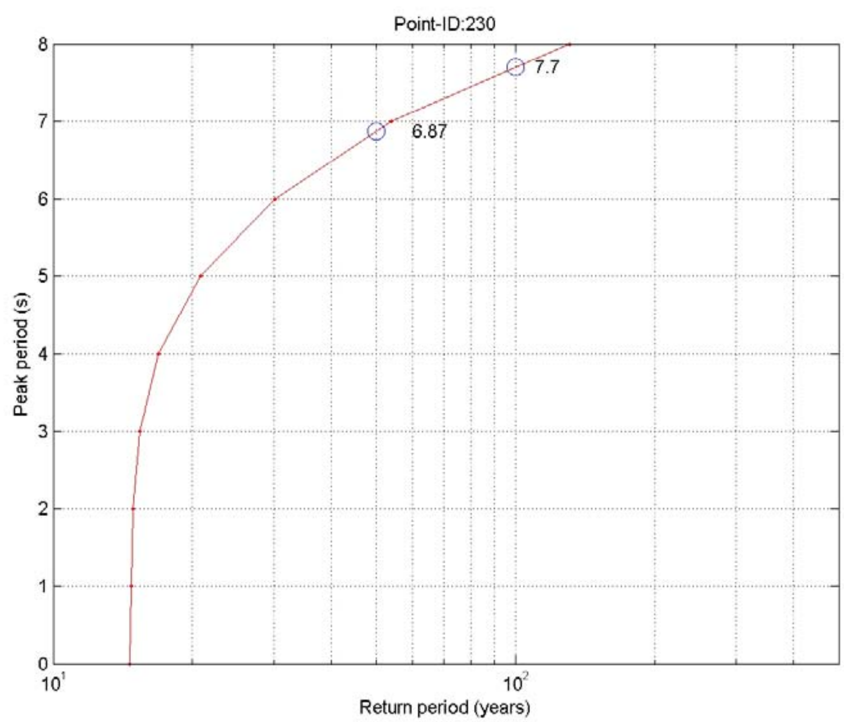


Figure 8 – Frequency curves of the wave height and wave period at Lake Pontchartrain (point 230) based on the STWAVE results and the JPM-OS method.

From the JPM-OS frequency analysis, 1% surge elevations, 1% wave heights, and 1% wave characteristics for existing conditions were applied in the wave run-up and overtopping calculations. Appendix A shows the 1% values for the surge levels and wave characteristics that have been used in this design report. These values do not consider any future changes due to factors such as subsidence and sea level rise. An additional analysis has been performed representing conditions that may occur 50 years in the future and is discussed in Section 2.6. This future condition (year 2057) does consider changes in the surge levels and wave characteristics due to subsidence and sea level rise.

2.2.4 Wave Overtopping

Several methods are presently available for computing the wave overtopping rates. These methods can be divided into empirical methods formulated by Van der Meer and Jansen, and Franco and Franco (TAW2002) 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 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[1]. A second set of formulas developed by Franco and Franco were used to compute wave overtopping at a vertical wall. The equations were placed in an Excel spreadsheet. A sample of the PC-Overslag output and the Franco and Franco spreadsheet is contained in Appendix F.

- **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). An extensive description of this model and the validation tests have been included in Appendix B of this report.

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.

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

The empirical approach is mostly used in this design report. Full Boussinesq results were not available in sufficient time to be used in the design process. As a design tool, the Boussinesq model lacks the capability to execute in a production mode. Compound levee cross-sections could not be modified iteratively in a straightforward and timely process. Several Boussinesq runs were made and have been compared with the empirical approach (see Appendix G). 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).

2.2.5 Wave Forces

For floodwalls, pump station fronting protection, tie-in walls, and other vertical "hard" structures, the Goda formulation for computing wave forces was used (see e.g. USACE, 2001; part VI). A definition sketch is shown in Figure 9. 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.

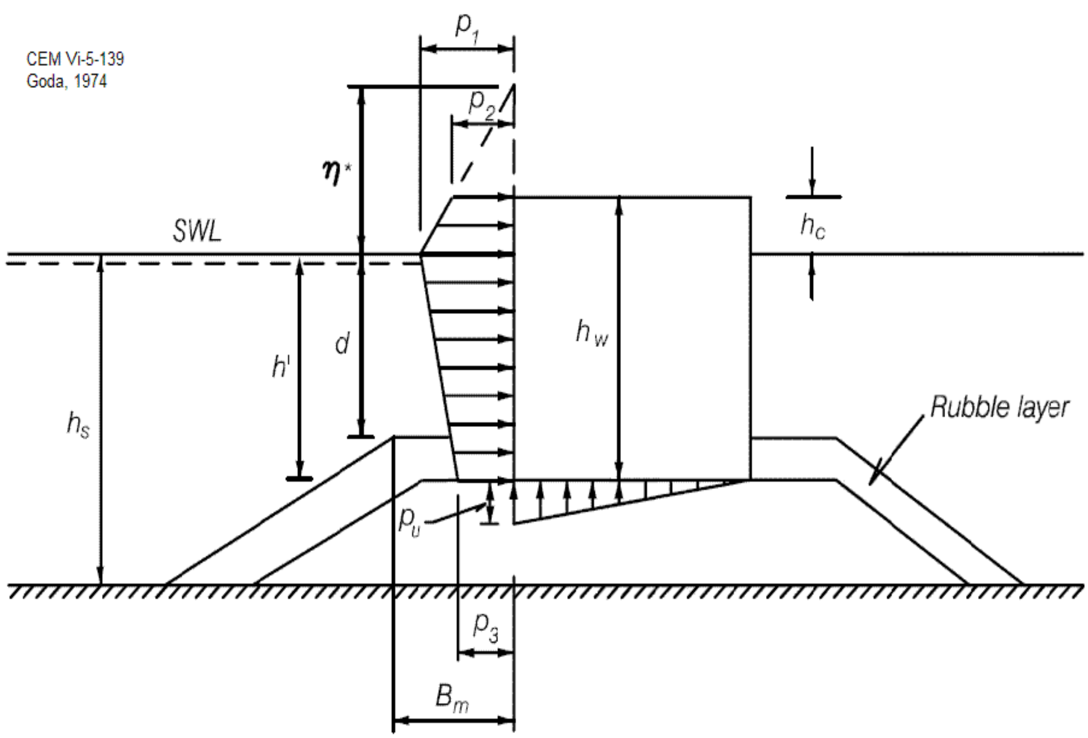


Figure 9 – Definition sketch of wave force calculations (source: Coastal Engineering Manual, 2001)

2.3 Step-wise Design Approach

The approach below gives a step-wise approach for determining final designs of the levees and 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 were 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

2.3.1 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, 2007). 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. Additional information on this assumption is contained in Chapter 6.

2.3.2 Simultaneous Occurrence of Maxima

Another assumption in the design approach is that the maximum water elevation and the maximum wave height occur simultaneously. Figure 10 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.

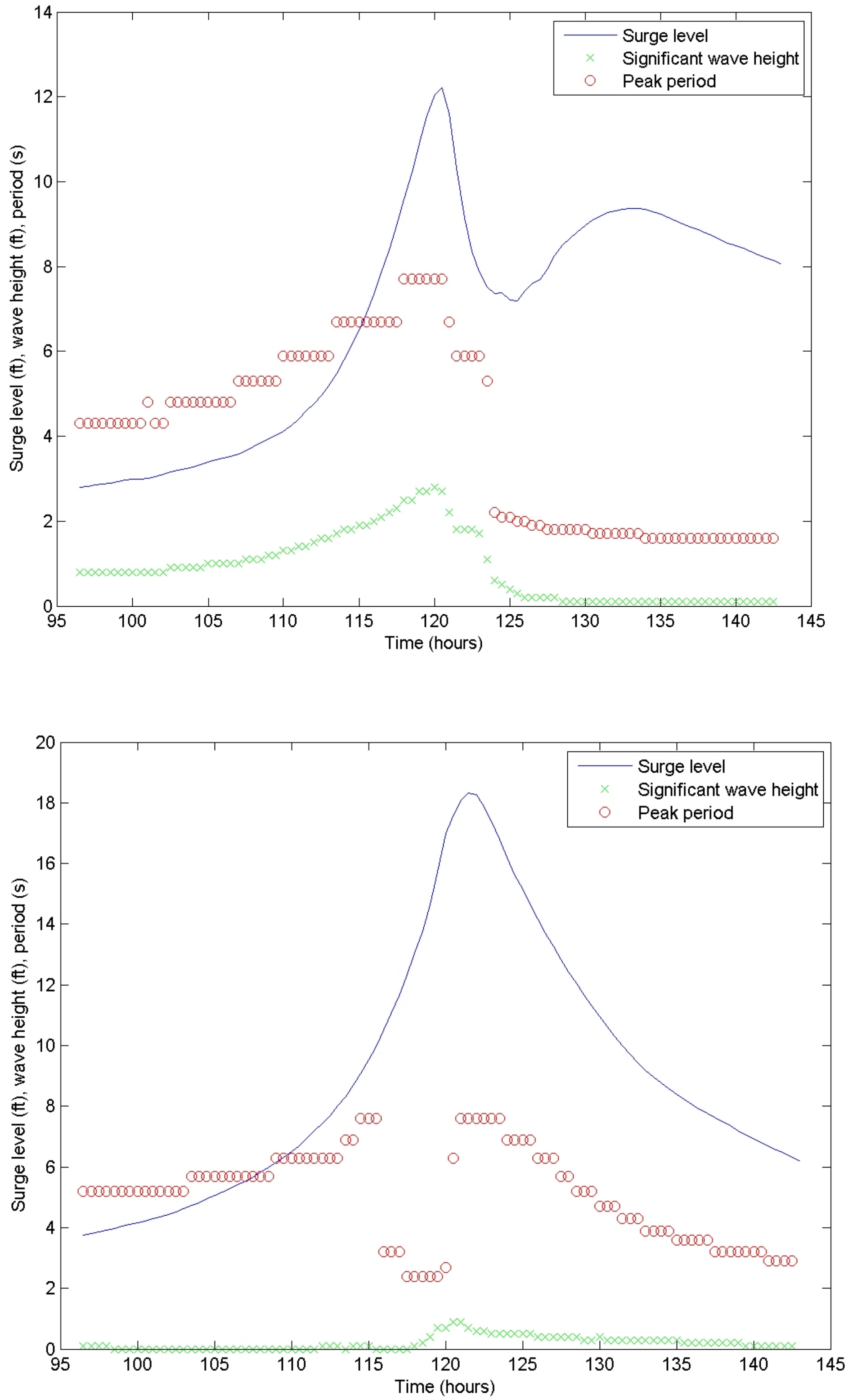


Figure 10 – Time histories of surge elevation and wave characteristics during storm 27 at Lake Pontchartrain (upper panel) and at Lake Borgne (lower panel).

2.3.3 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, expressed as a percentage. 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 50 and 78 percent 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 40 percent 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.

2.3.4 Overtopping Criteria

A literature survey was carried out to underpin the value for the overtopping criterion for levees that must used in this design approach (Appendix E). 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.

Additional information on overtopping rates can be found in Chapter 6.

2.3.5 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 describes the method used to account 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 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 11).

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	-	-	

Table 1 – Input for Monte Carlo Analysis.

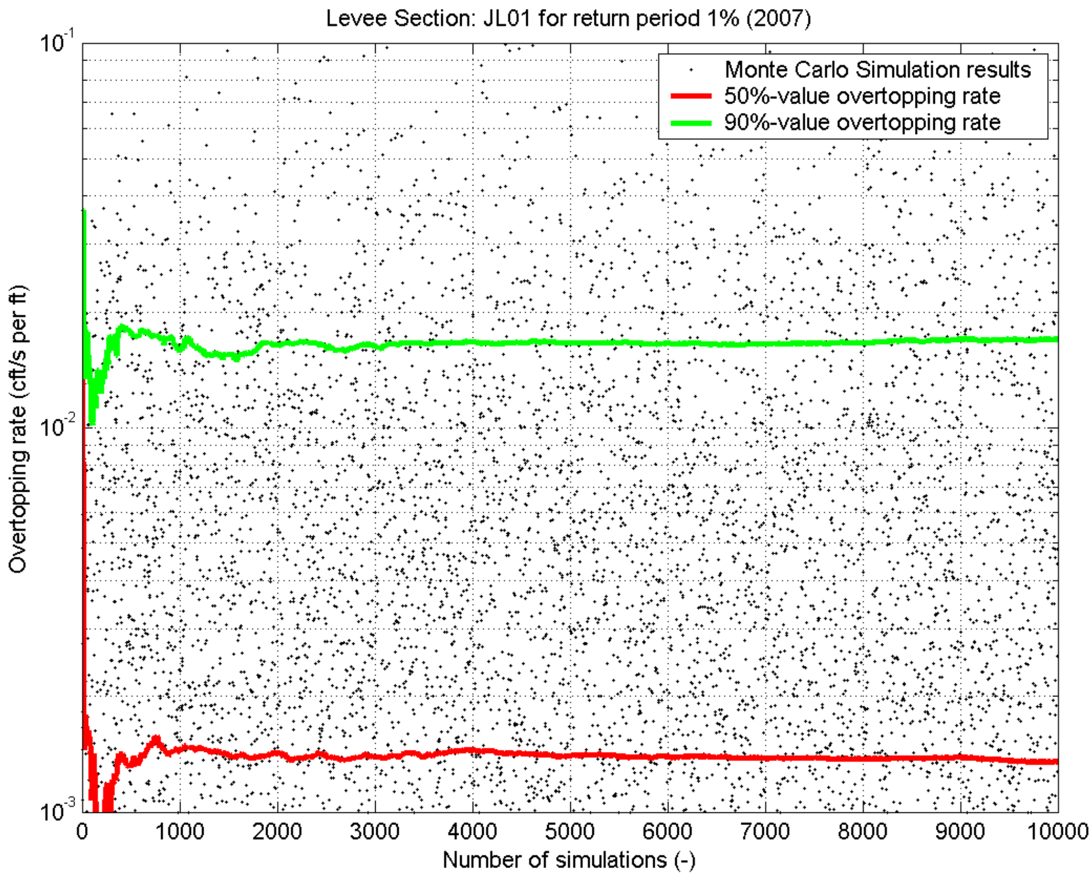


Figure 11 – The 50% and 90% confidence limit value of the overtopping rate as a function of the number of simulations during the Monte Carlo Analysis. The dots represent the actual results from the Monte Carlo Simulation, whereas the red and green lines represent the moving value over the number of simulations.

Figure 12 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.

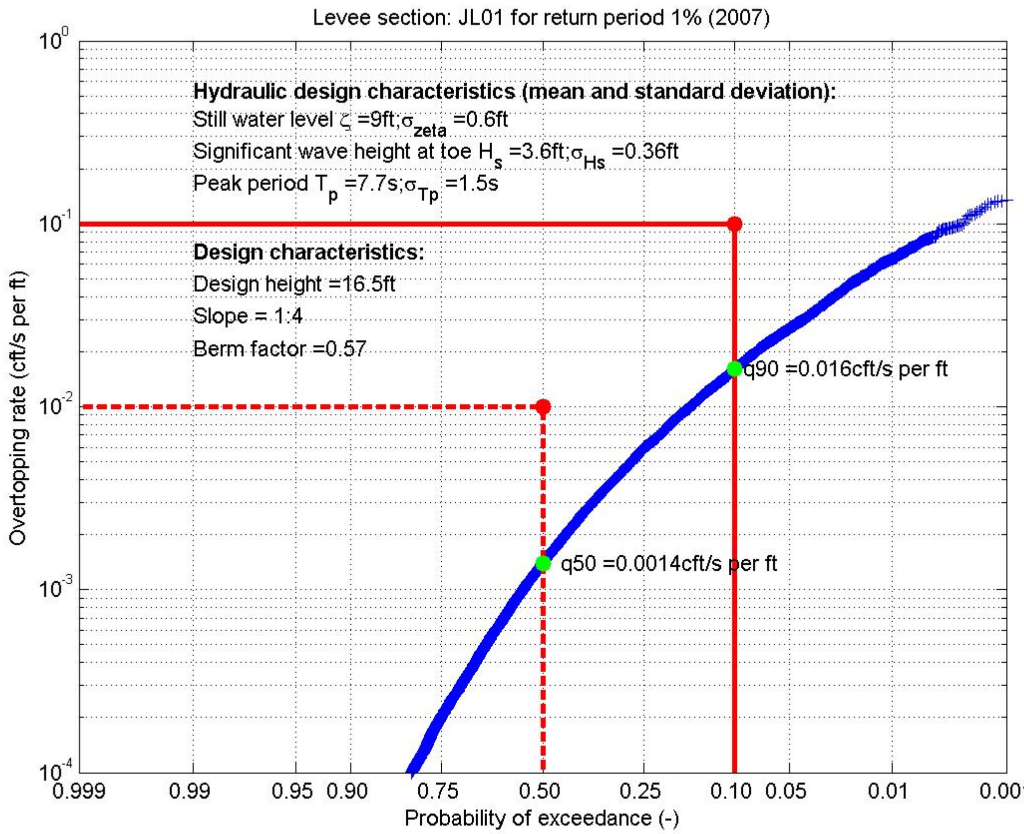


Figure 12 – Result of Monte Carlo Analysis for Jefferson Lakefront levee (existing conditions).

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.

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 was 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 was maintained, whereas the error in the surge level and the wave characteristics were treated independently.

2.3.6 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.

Step 2: Wave characteristics

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 L_0 (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.

Step 3: Overtopping rate

3.1 Apply PC-Overslag with Van der Meer formulations to determine the overtopping rates. If a wall is present, the empirical formulation of Franco and Franco 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).

Step 4: Dealing with uncertainties

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.

Step 5: Resiliency

In this report we evaluate the overtopping rate for the 0.2% exceedence event and compute both the 50% and 90% confidence limits of the overtopping rates 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. Additional comments on resiliency are contained in Chapter 6.

2.4 Example 1: Jefferson Parish Lakefront

The following is an example of the application of the step-wise design approach for a levee location along the Jefferson Parish Lakefront (see Figure 13). The preliminary design numbers used in September 2005 were as follows:

- water elevation 12ft (10ft including 2ft uncertainty)
- significant wave height 7.9ft
- peak period 7.2s

The proposed preliminary levee had an elevation of 16ft and an average slope of 1V:7H. The resulting overtopping rate was about 0.1 cfs per ft.

The step-wise design approach is applied below using the ADCIRC and STWAVE results from the 2007 grid. The output locations along this reach are shown in Figure 13. The output points 228 – 237 and 217 – 219 belong to this reach.

Figure 13 – Jefferson Parish Lakefront (point 217 – 219 and 228 – 237)

<u>Step 1: 1% surge elevation</u>

The 1% surge elevation along Jefferson Parish Lakefront is between 9.3 and 9.6ft (see Table 2). These numbers include the local wave setup just in front of the levee. The maximum 1% surge elevation is 9.0 ft at point 228/230; we have selected output point 230 here. The standard deviation at this point is 0.6 ft.

	2% event		1% event		0.2% event	
Pointid	mean	std	mean	std	mean	std
225	8	0.4	9.3	0.7	11.6	1.1
226	7.9	0.4	9.2	0.6	11.4	1.1
227	7.8	0.4	9.1	0.6	11.3	1.1
228	7.8	0.4	9	0.7	11.3	1.1
229	7.7	0.4	8.9	0.6	11.1	1.1
230	7.7	0.4	9	0.6	11.2	1.1
231	7.7	0.4	9	0.6	11.2	1.1
232	7.7	0.4	9	0.6	11.2	1.1
233	7.7	0.4	9	0.6	11.2	1.1
234	7.6	0.4	8.9	0.7	11.2	1.1
235	7.6	0.4	8.9	0.7	11.2	1.1
236	7.6	0.4	8.9	0.7	11.2	1.1
237	7.6	0.4	8.9	0.7	11.2	1.1
217	7.6	0.4	8.8	0.7	11.2	1.2
218	7.5	0.4	8.8	0.7	11.2	1.2
219	7.5	0.5	8.8	0.7	11.4	1.3

Table 2 – Surge elevations at Jefferson Parish Lakefront (Existing Conditions)

Step 2: Wave characteristics

The significant wave height and wave period are listed in Table 3. The maximum 1% significant wave height is 8.4ft and the maximum peak period is 7.7 seconds. The wave characteristics in Table 4 are at 600 ft from the levee. The bottom elevation 600ft from the shoreline is approximately 0 ft. NAVD88 2004.65.

	2% event		1% event		0.2% event	
Pointid	Hs (ft)	Tp (s)	Hs (ft)	Tp (s)	Hs (ft)	Tp (s)
225	6.3	7.0	7.4	7.8	9.1	9.0
226	6.7	7.0	7.8	7.8	9.5	9.0
227	6.6	6.9	7.7	7.7	9.4	9.0
228	6.4	7.0	7.4	7.7	9.0	9.0
229	7.2	6.7	8.4	7.5	10.2	8.8
230	6.5	6.9	7.5	7.7	9.2	9.0
231	6.5	6.8	7.6	7.7	9.2	9.0
232	6.2	6.8	7.2	7.6	8.9	9.0
233	6.0	6.9	7.0	7.7	8.7	9.1
234	6.2	6.7	7.2	7.6	8.9	9.0
235	6.3	6.7	7.4	7.5	9.1	8.9
236	5.8	6.7	6.8	7.6	8.5	9.0
237	6.0	6.6	7.1	7.5	8.8	8.8
217	6.3	8.7	1.6	3.3	2.6	4.9
218	6.4	8.6	1.6	3.4	2.6	4.6
219	5.9	8.6	1.7	3.2	2.7	4.2

Table 3 – Wave characteristics at Jefferson Parish Lakefront

The 1% surge elevation is 9ft, so the 1% wave height is about 80% of the water depth. This implies that the foreshore can be considered as shallow ($H/h \approx 1$) and breaking will take place towards the toe of the structure. The length of the foreshore is about 400ft, whereas the deep water wave length is about 300 ft. Because the shallow foreshore is longer than one deep water wave length, the maximum significant wave height is assumed to be H_{smax} = 0.4 h (≈ 3.6ft). Summarizing: design wave characteristics are H_s = 3.6ft and T_p = 7.7s.

Step 3: Overtopping rate

The Pre-Katrina authorized cross-sectional profile of the Jefferson Lakefront Levee is shown in Figure 14. The software program PC-Overslag was used to determine the overtopping rate. The average overtopping rate is 0.002 cfs/ft at this cross-section. Notice that the overtopping criterion is (much) below the design criterion for the average overtopping rate (0.01 cfs/ft). This section can be used for existing conditions.

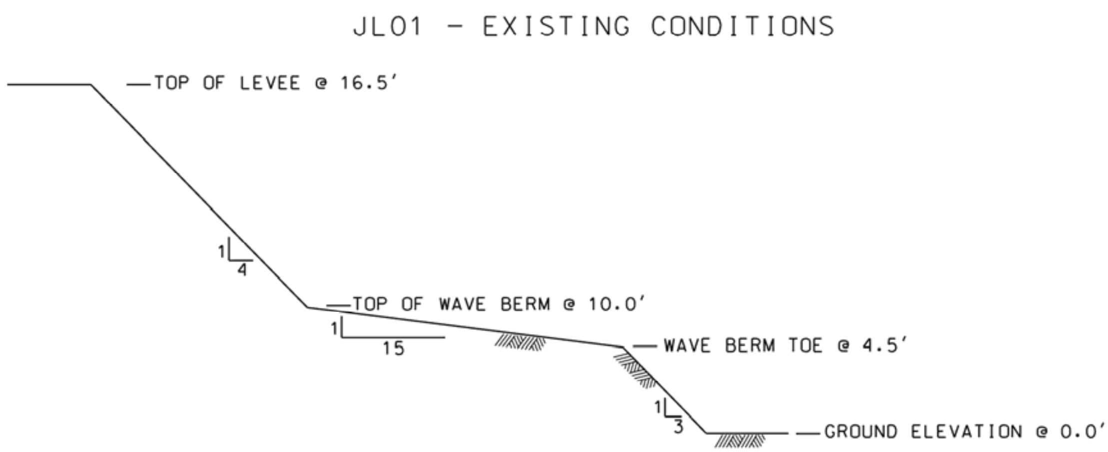


Figure 14 – Cross section Jefferson Lakefront (existing conditions)

Step 4: Dealing with uncertainties

The result of the uncertainty method is shown in Figure 11. It shows the frequency curve of the overtopping rate (levee height 16.5ft including a berm) using the mean / standard deviations of the 1% water elevation (9.0ft / 0.6ft), the wave height at the toe (3.6ft / 0.4ft) and the peak period (7.7s / 1.5s). The overtopping rate is 0.02 cfs/ft at a 90% confidence limit and is 0.001 cfs/ft at a 90% confidence limit. These values meet the design criteria for levees.

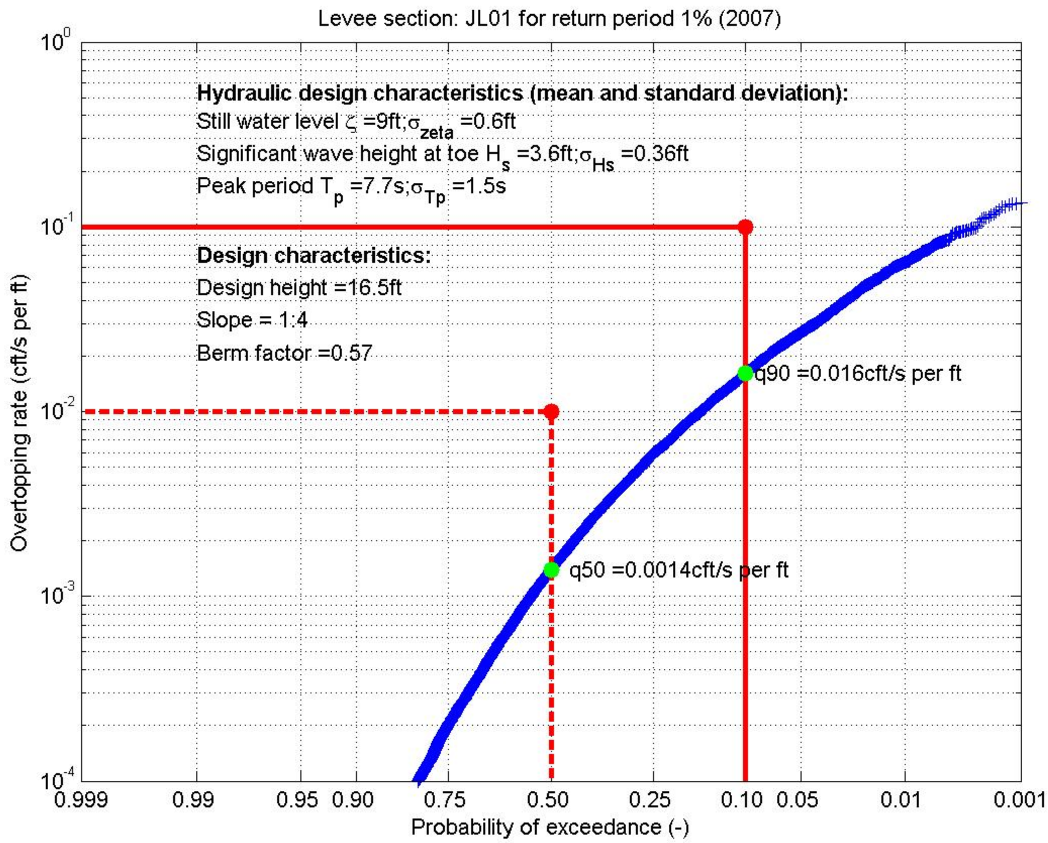


Figure 15 – Overtopping rate as a function of the probability of exceedence for the Jefferson Lakefront Levee (existing conditions) for the 1% event.

Step 5: Resilience for events above design level

The effect of resilience is investigated using the 0.2% values for the hydraulic boundary conditions. These numbers are:

- surge level 11.2 ft
- significant wave height at toe 4.5 ft
- peak period 9.0 s

The exceedence frequency curve of the overtopping rate has been computed using the 1% design cross-section (see Figure 14). The resulting overtopping rate is shown in Figure 16. The 50%-value of the overtopping rate is approximately 0.1 cfs/ft and the 90%-value is 0.5 cfs/ft. These values might indicate that the chance of survival of this levee during a 0.2% event is relatively high.

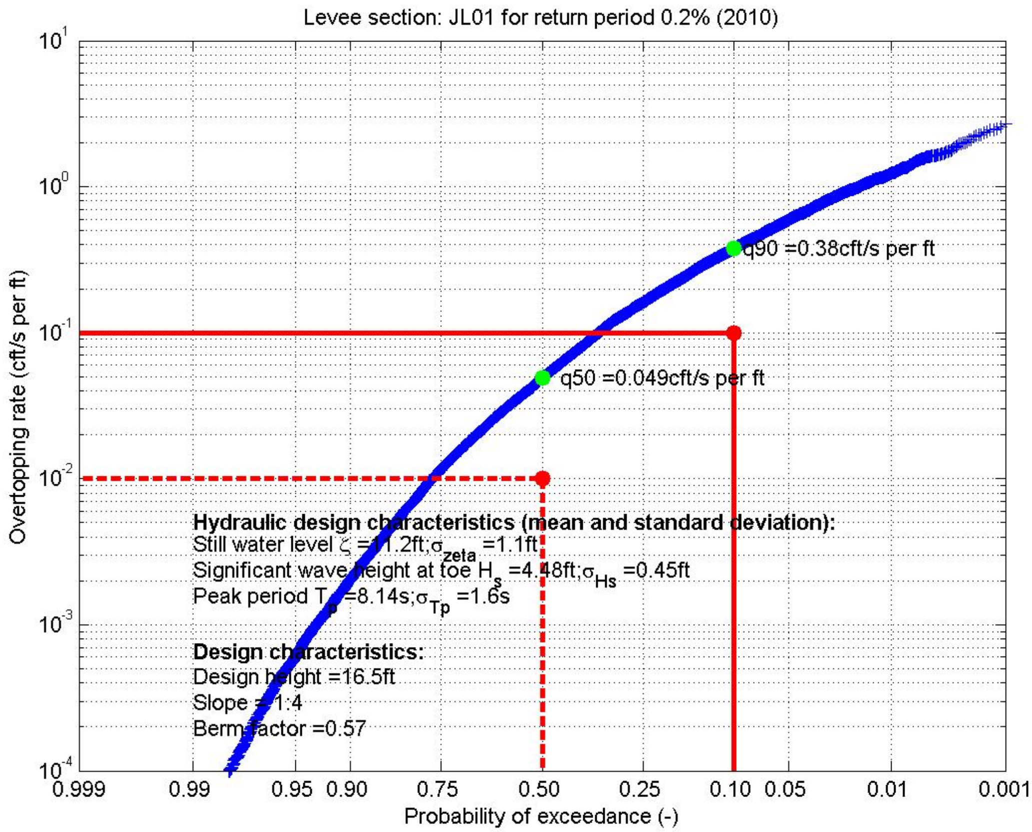


Figure 16 – Overtopping rate as a function of the probability of exceedence for the Jefferson Lakefront Levee (existing conditions) for the 0.2% event.

2.5 Example 2: MRGO

The following is an example of the application of the step-wise design approach for a location along the MRGO levee (Figure 17). The preliminary design numbers used in September 2005 were (segment 1):

- water level 17ft (14.5ft including 2.5ft uncertainty)
- significant wave height 11.0ft
- peak period 12.0s

The proposed preliminary levee height had a crest elevation of 24ft with a composite slope of 1V:12H, and a computed overtopping rate of 0.1 cfs per ft.

The step-wise design approach is applied below using the ADCIRC and STWAVE results from the 2010 grid. The 2010 conditions have been chosen because this area is affected by the gates at MRGO/GIWW. The output locations along this reach are shown in Figure 17. The output points 35 - 54 and 21 - 22 belong to this reach.

Figure 17 – MRGO levee with output points from ADCIRC and STWAVE (point 35 – 54 and 21 - 22)

Step 1: 1% surge elevation

The 1% surge elevation along MRGO is between 14.9 and 18.4ft (see Table 4). The variation in the surge level is quite large (> 3 ft), indicating that this reach should be sub-divided for the final design. This example is only meant to show the step-wise approach. Point 33 was used for the most southern section of this levee. The maximum 1% surge level is 15.6 ft at point 33; the maximum standard deviation is 1.2ft.

	2% event		1% event		0.2% event	
Pointid	mean	std	mean	std	mean	std
33	13.5	0.8	15.6	1.2	19.9	2.1
34	12.9	0.6	14.8	0.9	18.1	1.6
35	12.4	0.8	14.9	1.3	19.4	2.2
36	12.4	0.6	14.3	1.0	17.7	1.7
37	12.5	0.6	14.5	1.0	17.9	1.7
38	12.7	0.9	15.2	1.3	19.8	2.3
39	11.7	0.9	14.7	1.3	19.4	2.3
40	12.0	0.8	14.9	1.2	19.3	2.2
41	11.3	0.8	14.6	1.3	19.1	2.2
42	13.4	0.8	16.1	1.3	20.6	2.2
43	13.2	0.8	15.8	1.2	20.0	2.1
44	13.4	0.7	15.8	1.0	19.5	1.8
45	13.7	0.7	15.9	1.0	19.6	1.8
46	13.9	0.7	16.1	1.1	19.9	1.9
47	14.1	0.7	16.4	1.1	20.2	1.9
48	14.3	0.7	16.7	1.1	20.5	1.9
49	14.5	0.7	17.0	1.1	20.8	1.9
50	14.7	0.7	17.3	1.1	21.1	1.9
51	14.9	0.7	17.6	1.1	21.4	1.9
52	15.1	0.7	17.9	1.1	21.7	1.9
53	15.3	0.7	18.2	1.1	22.0	1.9
54	15.5	0.7	18.4	1.0	22.1	1.8

Table 4 – Surge levels at MRGO for Existing Conditions

Step 2: Wave characteristics

The significant wave height and wave period are listed in Table 5. In the southern section, the maximum 1% significant wave height for point 33 is 5.4ft and the peak period is 7.9s. The bottom elevation 600ft from the shoreline is approximately 0 ft. NAVD88 2004.65. The 1% surge elevation is 15.6ft, so the 1% wave height is about 35% of the water depth. This implies that the foreshore can be considered as shallow ($H/h < 1/3$) and breaking will be very limited towards the toe of the levee. Therefore, the 1% wave height will not be affected by the foreshore. Summarizing: design wave characteristics are H_s = 5.4ft and T_p = 7.9s for this specific location under existing conditions.

	2% event		1% event		0.2% event	
Pointid	Hs (ft)	Tp (s)	Hs (ft)	Tp (s)	Hs (ft)	Tp (s)
33	3.7	7.5	5.4	8.9	9.9	14.4
34	4.2	7.6	5.4	8.5	7.5	9.9
35	2.2	6.5	4.3	8.7	9.0	16.9
36	2.7	6.4	3.4	7.4	4.7	8.9
37	2.0	5.7	2.8	6.8	4.1	8.5
38	1.1	4.7	2.7	6.7	6.8	14.2
39	0.2	2.8	2.5	5.7	6.0	10.1
40	0.2	3.3	2.3	5.7	5.3	8.7
41	0.6	2.1	3.4	5.6	6.2	8.3
42	3.2	4.4	5.3	6.3	10.0	14.3
43	3.3	4.1	5.3	5.8	8.7	9.7
44	3.2	4.7	4.8	6.1	7.5	7.8
45	5.9	5.2	7.1	5.9	9.0	6.9
46	5.4	5.2	6.5	5.9	8.4	6.9
47	5.6	5.2	6.9	5.9	8.9	6.9
48	5.8	5.2	7.1	5.9	9.1	6.9
49	6.0	5.3	7.3	5.9	9.3	7.0
50	6.0	5.1	7.3	5.8	9.4	6.9
51	5.6	5.2	6.9	5.8	9.0	6.8
52	5.8	5.3	7.1	5.9	9.3	6.9
53	5.3	5.1	6.7	5.8	8.7	6.8
54	5.3	5.0	6.4	5.7	8.3	6.8

Table 5 – Wave characteristics at MRGO

Step 3: Overtopping rate

The proposed cross-sectional profile is given in Figure 18. PC-Overslag was used to determine the mean overtopping rate first. The mean overtopping rate is 0.006 cfs/ft for this cross-section.

Step 4: Dealing with uncertainties

The result of the uncertainty analysis is shown in Figure 15. It shows the frequency curve of the overtopping rate given the mean values and standard deviations of the 1% water level (15.6 ft / 1.2 ft), the wave height (5.4ft / 0.5ft) and the wave period (8.9s / 1.8s). The overtopping rate at the upper 90% confidence limit is 0.06 cfs/ft, and the best estimate overtopping rate equals 0.005 cfs/ft. Both overtopping rates show that this cross-section fulfills the design criteria.

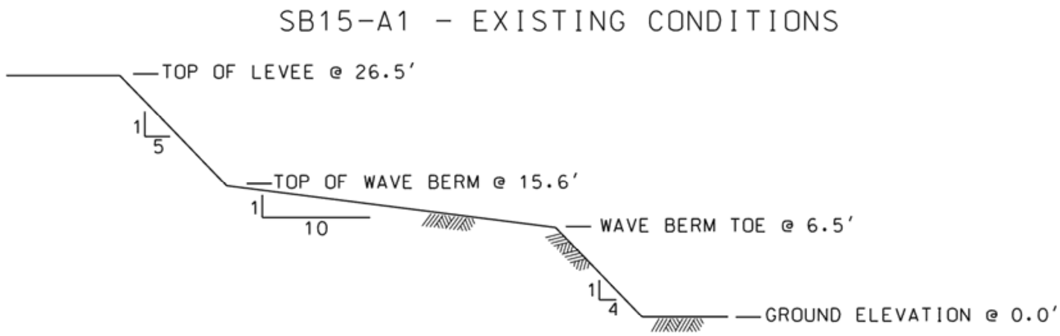


Figure 18 – Proposed cross-section at the southern section of MRGO

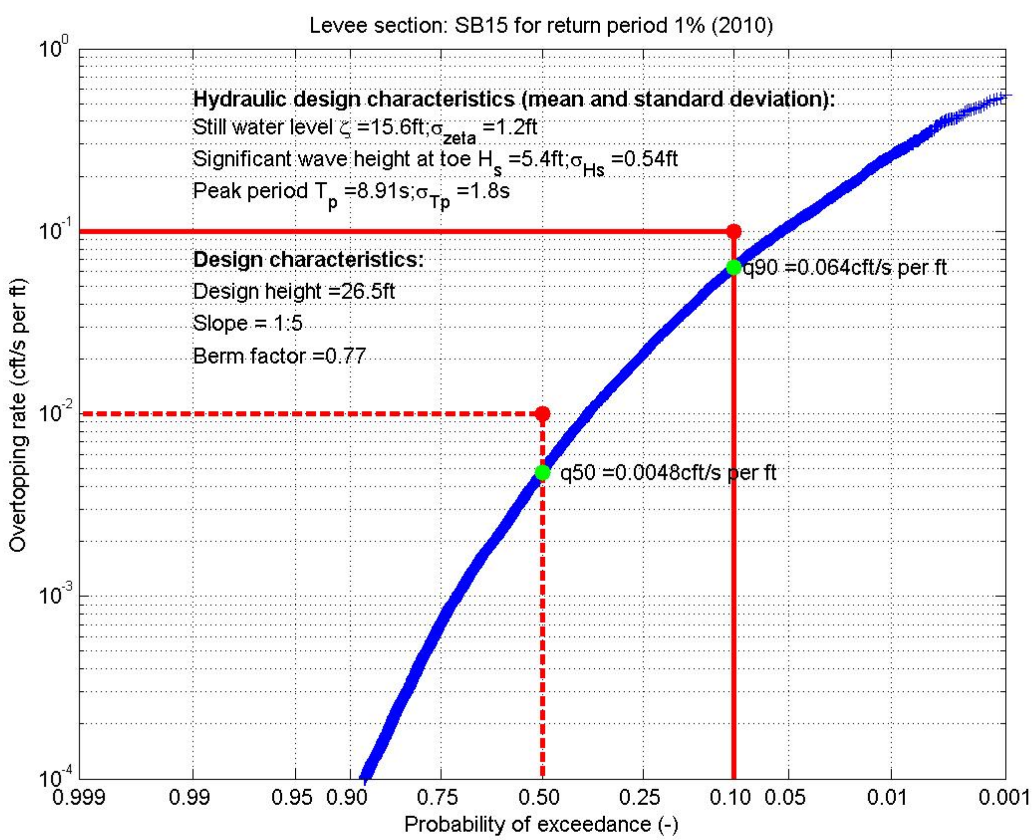


Figure 19 – Overtopping rate as a function of the probability of exceedence for the MRGO levee (existing conditions) for the 1% event

Step 5: Resilience for events above design level

The effect of resilience is investigated using the 0.2% values for the hydraulic boundary conditions. These numbers are:

- surge level 19.9 ft
- significant wave height 8.0 ft
- peak period 14.4 s

The exceedence frequency curve of the 0.2% overtopping rate has been computed using the 1% design values and the 0.2% hydraulic boundary conditions. The results are shown in the figure below. The 50%-value of the overtopping rate is approximately 2 cfs/ft. This is about 200 times higher than the 1% design criterion. This may indicate that the chance of survival of this design during a 0.2% event is low.

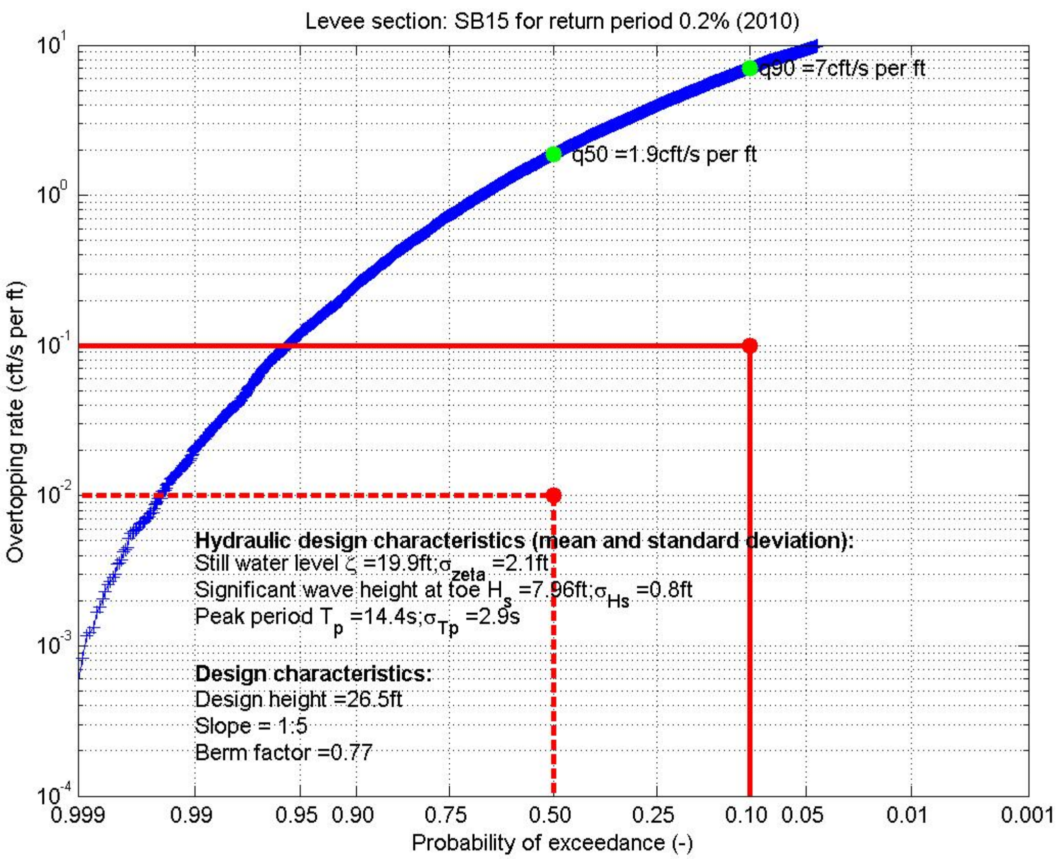


Figure 20 – Overtopping rate as a function of the probability of exceedence for the MRGO levee (existing conditions) for the 0.2% event

2.6 Design Conditions

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

2.6.1 Existing Conditions

Design elevations for this scenario are considered to reflect conditions that are likely to exist when the 100-year protection system is completed in 2011. 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 using the 2007 grid 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.

2.6.2 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 21 shows the combined natural subsidence/eustatic sea level rise for the hurricane protection project area. The values presented in Figure 21 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.

Subsidence Rates for Southern LA in ft/cent. Includes 1.3 ft/cent for sea level rise

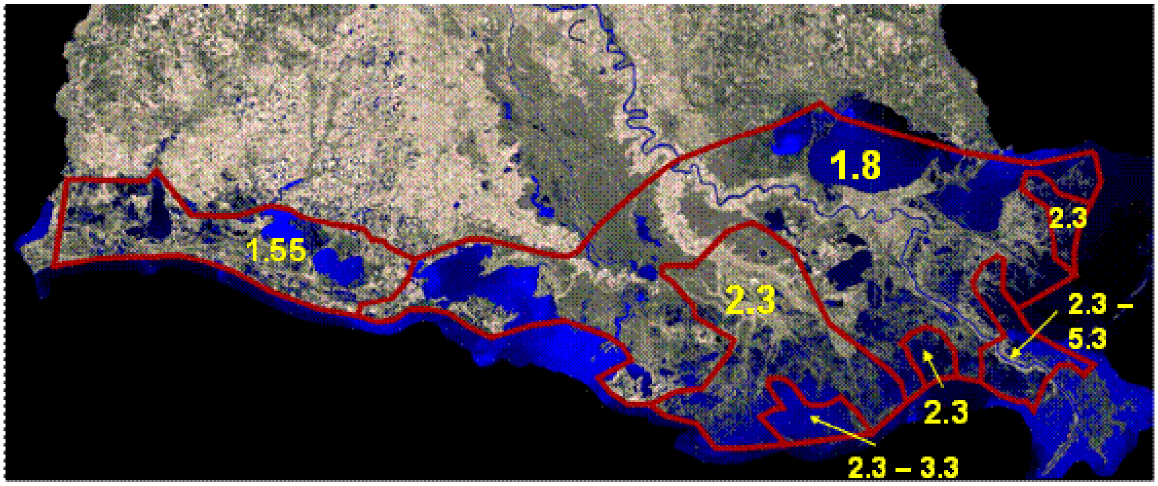


Figure 21 Estimated relative sea level rise during 100 year (subsidence + sea level rise)

Several ADCIRC and STWAVE model runs were performed to investigate the effect of the increasing sea level rise on surge levels and wave characteristics (see Appendix D). 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 the results in Appendix D, the future conditions are summarized below (Table 6):

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)

Table 6 - Future conditions for surge level and wave characteristics

Because the future conditions 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 conditions surge elevation was used in wave computations, wave loads on walls and other "hard" structures, and to determine design elevations.

2.7 *Project Design Heights 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 existing conditions, when the 100-year system is in place, and 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 condition 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. 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 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 future conditions plus 2 ft. for structural superiority. Floodwalls will be constructed to the 2057 level where little or no disruption of services would occur to repair the walls.

The wave forces have been computed for floodwalls and other structures and are calculated for future conditions. 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.

2.8 Concluding Remarks

This chapter presents a design approach for the levees/structure elevations for the Hurricane Protection System around New Orleans and includes a method to evaluate the 50% and 90% confidence limit values of the overtopping rate. The design approach consists of five steps:

- Define the 1% surge elevations
- Define the 1% wave characteristics (significant wave height, peak period)
- Design flood protection measures based on 1% surge elevations and 1% wave characteristics
- Check if the 50% and 90% confidence limit values of the overtopping rate during the 1% event are less than defined thresholds for levees and floodwalls using a Monte Carlo Analysis
- Investigate the resilience of the flood protection design for an event above design level by computing the surge level and the overtopping rates during a 0.2% event

Notice that the present approach does not take into account the correlation between the water levels and the wave characteristics, and also does not account for time lag between the peak of the surge and the peak of the waves. These assumptions lead to a conservative design in line with the recommendations from IPET.

To account for changes due to subsidence and sea level rise over a 50 year period, the surge elevations are adjusted by 1.5 to 2ft. The wave characteristics are adjusted based on half the increase in surge elevations (i.e. +0.75ft and +1ft). The effect on the wave period is determined by assuming that the wave steepness (H/T^2) remains constant.

3 Lake Pontchartrain, LA and Vicinity

3.1 General

The Lake Pontchartrain, LA and Vicinity project includes the levees from St. Charles Parish west of New Orleans to St. Bernard Parish east of New Orleans (see Figure 22). The surge elevations along the Lakefront are caused by the wind setup at the lake and the intrusion of the surge from the Gulf of Mexico. The 1% surge elevations are about 10ft along the entire Lakefront area. The waves near the levees at the Lakefront of Lake Pontchartrain are locally generated wind waves. The 1% wave characteristics just in front of the levee are: significant wave height 7 to 8 ft and peak period 7 to 8 seconds. The model results show that because there is the marsh area in front of St. Charles Parish, the wave heights and wave periods are strongly reduced here, whereas the surge elevation is similar to the Lakefront area.

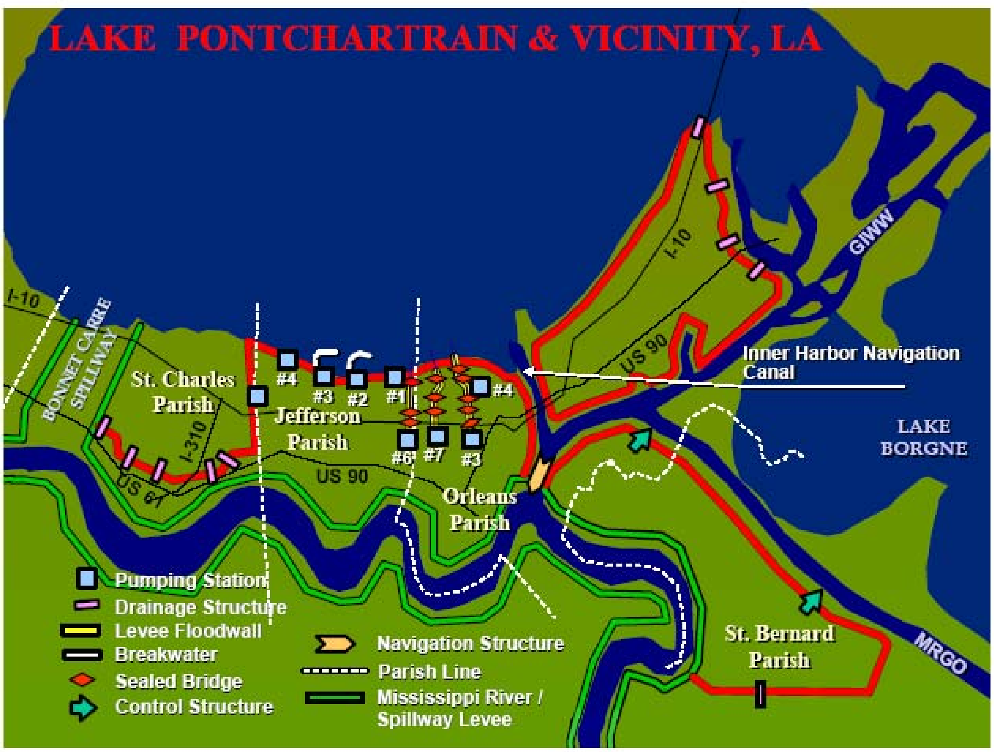


Figure 22 – Lake Pontchartrain, LA and Vicinity

The hydraulic conditions at the eastside of Orleans Parish and St. Bernard Parish are quite different from the Lake Pontchartrain conditions. The 1% surge elevations are much higher (15 to 17ft) and the wave climate is also different. The 1% wave height is generally lower than in Lake Pontchartrain (4 to 6 ft) due to the relatively shallow area, but the wave periods are generally larger (8 to 10s). The wave periods can be quite large (> 12 s) for events above the considered design event (< 1%). Long swell waves from the Gulf of Mexico can have a devastating effect, as suggested by the hindcast modeling of Hurricane Katrina (IPET, 2007).

This chapter discusses the levee and floodwall heights for the existing and future conditions at Lake Pontchartrain, LA and Vicinity from west to east. The outline of this chapter is as follows:

- St. Charles (3.2)
- Jefferson Parish (3.3)
- Orleans Metro Lakefront (3.4)
- Orleans East Lakefront (3.5)
- South Point to GIWW and GIWW outside the gate (3.6)
- IHNC and GIWW with MRGO gate only (3.7)
- IHNC and GIWW with Seabrook and MRGO gate (3.8)
- Closure and levee at MRGO/GIWW gate (3.9)
- St. Bernard Parish (3.10)

Each section discusses the hydraulic boundary conditions, the design elevations, the wave forces at the structures and the resiliency analysis.

3.2 St. Charles Parish

3.2.1 General

The St. Charles Parish portion of the Lake Pontchartrain, LA and Vicinity Hurricane Protection System is located north of Airline Highway (U.S. Highway 61). It runs from the Bonnet Carré Spillway East Guide Levee to the Jefferson-St. Charles Parish boundary at the New Orleans Airport East-West runway terminus. Five drainage structures are included to allow intercepted drainage to flow north into the adjacent bayous and drainage canals and ultimately into Lake Pontchartrain. Floodwalls are located at Interstate 310 (I-310), Shell Pipeline Crossing, Good Hope and at the Gulf South Pipeline Crossing. A double track railroad floodgate is located near the eastern end of the project where the Canadian National Railroad crosses through the protection system. Figure 23 shows the levee and floodwall segments analyzed for the St. Charles Parish.

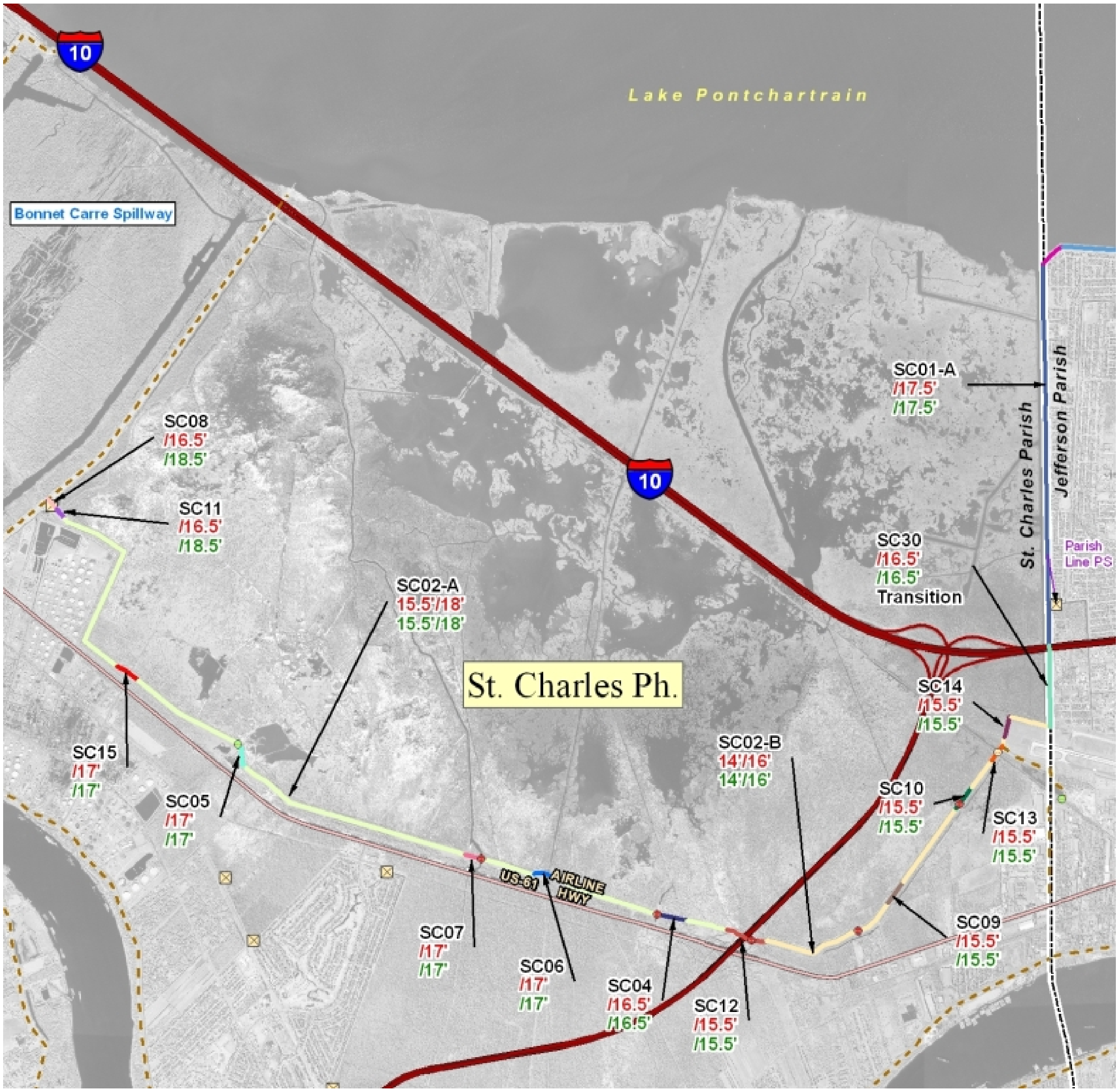


Figure 23 – Levee and floodwall sections in St.Charles. The numbers represent existing/future conditions and are without (red) and with (green) structural superiority

3.2.2 Hydraulic Boundary Conditions

The hydraulic design characteristics for the levees in St. Charles Parish are listed in Table 7. The existing hydraulic conditions are based on the JPM-OS method using the results from ADCIRC and STWAVE. The future conditions are derived by adding 1.5 ft to the surge elevation, and adding 0.75 ft to the wave height. The wave period is computed using the assumption that the wave steepness remains constant. For more information, see Chapter 2.

St Charles Parish Sections 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
SC08	Bayou Trepagnier Pump Station	Structure/Wall	Future	12.8	0.7	2.7	0.2	4.0	0.7
SC11	Bonnet Carre Tie-in Floodwall	Structure/Wall	Future	12.8	0.7	2.7	0.2	4.0	0.7
SC05	Good Hope Floodwall	Structure/Wall	Future	12.9	0.8	3.1	0.2	4.7	0.8
SC02-A	St.Charles Parish Levee west of I-310	Levee	Existing	11.3	0.8	2.3	0.2	4.2	0.8
SC02-A	St.Charles Parish Levee west of I-310	Levee	Future	12.8	0.8	3.1	0.2	4.8	0.8
SC07	Cross Bayou Canal T-Wall	Structure/Wall	Future	12.9	0.8	3.1	0.2	4.7	0.8
SC06	Gulf South Pipeline T-Wall	Structure/Wall	Future	12.9	0.8	3.1	0.2	4.8	0.8
SC04	St. Rose Canal Drainage Structure T-Wall	Structure/Wall	Future	12.6	1.0	2.7	0.2	4.5	0.8
SC12	I-310 Floodwall	Structure/Wall	Future	12.3	0.8	2.3	0.2	3.9	0.6
SC02-B	St.Charles Parish Levee east of I-310	Levee	Existing	10.8	0.8	1.6	0.2	3.2	0.6
SC02-B	St.Charles Parish Levee east of I-310	Levee	Future	12.3	0.8	2.4	0.2	3.9	0.6
SC09	Almedia Drainage Structure	Structure/Wall	Future	12.3	0.8	2.4	0.2	3.9	0.6
SC10	Walker Drainage Structure	Structure/Wall	Future	12.2	0.8	2.5	0.2	3.8	0.6
SC13	Armstrong Airport Floodwall	Structure/Wall	Future	12.1	0.8	2.4	0.2	4.0	0.7
SC14	ICRR Floodgate	Structure/Wall	Future	12.1	0.8	2.4	0.2	4.1	0.7
SC30	Transition	Structure/Wall	Future	11.9	0.8	2.9	0.2	5.0	0.9
SC01-A	St. Charles Return Levee/Wall	Structure/Wall	Future	11.1	0.7	4.1	0.3	6.1	1.1
SC15	Shell Pipeline Floodwall	Structure/Wall	Future	12.9	0.8	3.1	0.2	4.7	0.8

Table 7 – St. Charles Parish Segments - 1% Hydraulic Boundary Conditions

3.2.3 Project Design Heights

The design characteristics for the sections in St. Charles Parish, including levees, floodwalls, drainage structures and pump stations are listed in Table 8. The return levee (SC01-A) is a floodwall on top of a levee and the marsh levee is divided into a section west of I-310 (SC02-A) and east of I-310 (SC02-B). The remaining sections are structures. Note that these structures are only evaluated for future conditions, because they are hard structures.

The height of vertical walls was determined at the drainage structures, floodwalls and pump stations using available topographic and bathymetric information. Information was not available for the Almedia and Walter drainage structures, the floodwall at I-310, and the floodgate at ICRR Railroad; geometry was estimated. Section SC08 and SC11 do include 2ft structural superiority.

St Charles Parish Sections 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
SC08	Bayou Trepagnier Pump Station	Structure/Wall	Future	12.8	18.5	0.001	0.004
SC11	Bonnet Carre Tie-in Floodwall	Structure/Wall	Future	12.8	18.5	0.001	0.004
SC05	Good Hope Floodwall	Structure/Wall	Future	12.9	17.0	0.020	0.078
SC02-A	St.Charles Parish Levee west of I-310	Levee	Existing	11.3	15.5	0.009	0.079
SC02-A	St.Charles Parish Levee west of I-310	Levee	Future	12.8	18.0	0.008	0.071
SC07	Cross Bayou Canal T-Wall	Structure/Wall	Future	12.9	17.0	0.019	0.078
SC06	Gulf South Pipeline T-Wall	Structure/Wall	Future	12.9	17.0	0.020	0.077
SC04	St. Rose Canal Drainage Structure T-Wall	Structure/Wall	Future	12.6	16.5	0.011	0.067
SC12	I-310 Floodwall	Structure/Wall	Future	12.3	15.5	0.009	0.054
SC02-B	St.Charles Parish Levee east of I-310	Levee	Existing	10.8	14.0	0.007	0.064
SC02-B	St.Charles Parish Levee east of I-310	Levee	Future	12.3	16.0	0.008	0.072
SC09	Almedia Drainage Structure	Structure/Wall	Future	12.3	15.5	0.012	0.066
SC10	Walker Drainage Structure	Structure/Wall	Future	12.2	15.5	0.015	0.071
SC13	Armstrong Airport Floodwall	Structure/Wall	Future	12.1	15.5	0.009	0.050
SC14	ICRR Floodgate	Structure/Wall	Future	12.1	15.5	0.009	0.049
SC30	Transition	Structure/Wall	Future	11.9	16.5	0.007	0.031
SC01-A	St. Charles Return Levee/Wall	Structure/Wall	Future	11.1	17.5	0.012	0.041
SC15	Shell Pipeline Floodwall	Structure/Wall	Future	12.9	17.0	0.020	0.075

Table 8 – St. Charles Parish Segments - 1% Design Information

The basic levee design of the marsh levee is shown in Figure 24 (SC02-B) and Figure 25 (SC02-A). The flat slope and the levee height were allowed to vary to meet the design criterion. An elevation of 0 ft was assumed for the toe of the levee. The SC02-B East levee section has a +14ft height (existing conditions) with a 1:3 slope. For future conditions, a wave berm has to be included and the height must be raised to +16ft to meet the design criteria. The SC02-A West levee section is a bit more exposed with a higher surge level and also higher waves. Therefore, the design height is higher and the slope is milder in order to meet the design criteria. A +15.5ft design height is proposed for existing conditions and a +18ft design height for future conditions.

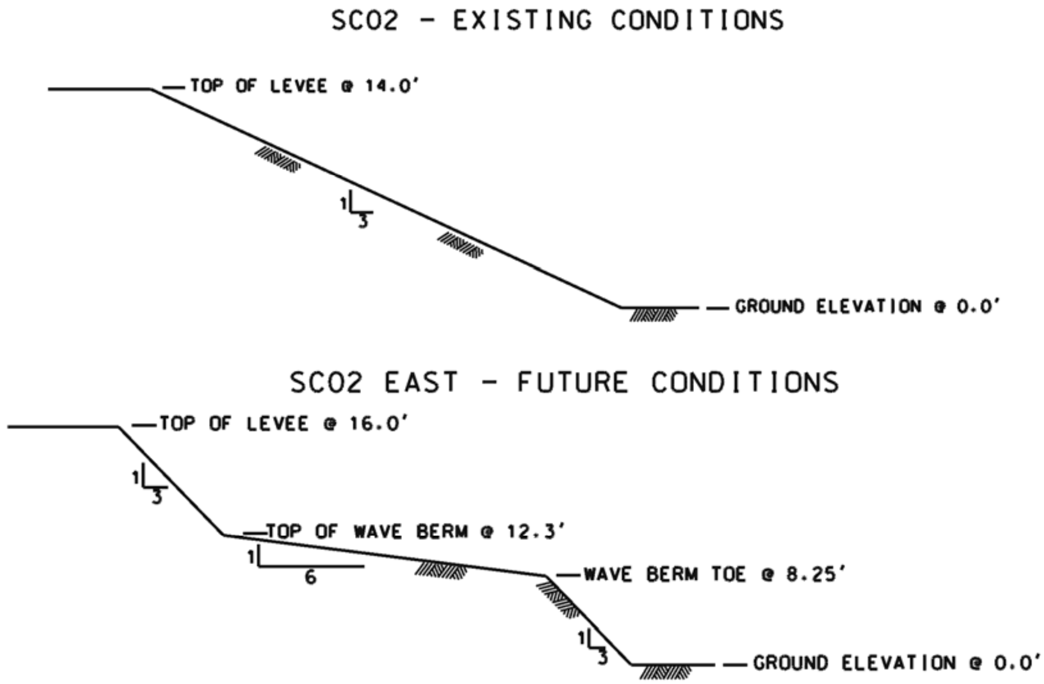


Figure 24 – Cross-section profile St. Charles Marsh levee SC02-B (East section) for present (upper panel) and future conditions (lower panel) in NAVD88 2004.65

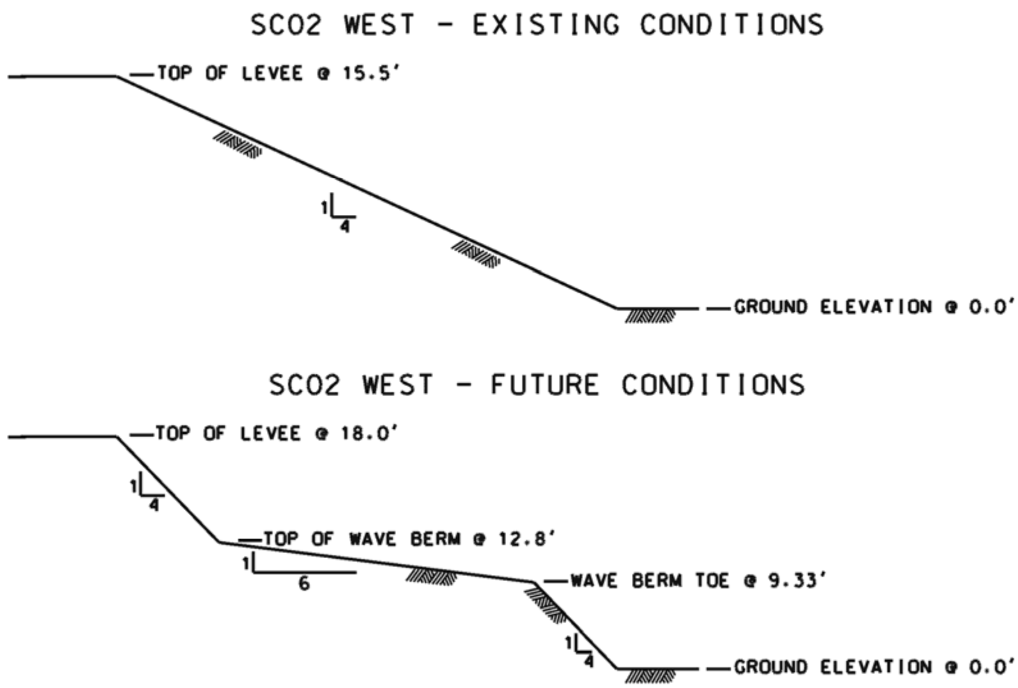


Figure 25 – Cross-section profile St. Charles Marsh levee SC02-A (West section) for present (upper panel) and future conditions (lower panel) in NAVD88 2004.65

3.2.4 Wave Forces

Wave forces were computed for all structures within the St. Charles Parish segment with the Goda method, using future conditions. The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics. The following tables summarize the resulting wave forces. Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

St Charles Sections Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
SC01	Lakeward Segment	5.2	36.3	9.5	5.2	36.3	9.5
SC04	St. Rose Canal Drainage	2.8	23.7	8.5	2.8	23.7	8.5
SC05	Good Hope Floodwall	3.4	30.1	8.8	3.4	30.1	8.8
SC06	Gulf South Pipeline T-Wall	3.5	30.7	8.7	3.5	30.7	8.7
SC07	Cross Bayou Canal T-Wall	3.4	30.0	8.8	3.4	30.0	8.8
SC08	Bayou Trapagnier Pump Station	2.6	24.4	9.2	2.6	24.4	9.2
SC09	Almedia Drainage Structure	2.0	16.9	8.3	2.0	16.9	8.3
SC10	Walker Drainage Structure	2.1	17.7	8.3	2.1	17.7	8.3
SC11	Bonnet Carre Tie-in Floodwall	2.6	24.3	9.2	2.6	24.3	9.2
SC12	I-310 Floodwall	2.0	16.3	8.3	2.0	16.3	8.3
SC13	Armstrong Airport Floodwall	2.1	17.3	8.2	2.1	17.3	8.2
SC14	ICRR Floodgate	2.2	17.7	8.1	2.2	17.7	8.1
SC15	Shell Pipeline Floodwall	3.4	30.1	8.8	3.4	30.1	8.8
SC30	Return Wall	3.3	27.3	8.2	3.3	27.3	8.2

Table 9 – Waves Forces for St. Charles Parish Segments (50% values)

St Charles Sections Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
SC01	Lakeward Segment	6.5	46.4	9.6	6.5	46.4	9.6
SC04	St. Rose Canal Drainage	3.6	31.0	8.6	3.6	31.0	8.6
SC05	Good Hope Floodwall	4.5	39.3	8.8	4.5	39.3	8.8
SC06	Gulf South Pipeline T-Wall	4.6	40.2	8.8	4.6	40.2	8.8
SC07	Cross Bayou Canal T-Wall	4.5	39.5	8.8	4.5	39.5	8.8
SC08	Bayou Trapagnier Pump Station	3.5	33.0	9.3	3.5	33.0	9.3
SC09	Almedia Drainage Structure	2.7	22.1	8.4	2.7	22.1	8.4
SC10	Walker Drainage Structure	2.8	23.1	8.4	2.8	23.1	8.4
SC11	Bonnet Carre Tie-in Floodwall	3.5	32.9	9.3	3.5	32.9	9.3
SC12	I-310 Floodwall	2.6	21.3	8.3	2.6	21.3	8.3
SC13	Armstrong Airport Floodwall	2.8	22.5	8.2	2.8	22.5	8.2
SC14	ICRR Floodgate	2.8	23.2	8.2	2.8	23.2	8.2
SC15	Shell Pipeline Floodwall	4.5	39.2	8.8	4.5	39.2	8.8
SC30	Return Wall	4.3	35.6	8.3	4.3	35.6	8.3

Table 10 – Waves Forces for St. Charles Parish Segments (90% values)

3.2.5 Resiliency

The designs for the levees and structures within St. Charles Parish were examined for resiliency by also computing the best estimate values for the surge level and the overtopping rate for the 0.2 percent event for each design. The results are presented in Table 11. For all sections, the 0.2% surge elevation remains below the top of the flood defense. The overtopping rates are, in some cases, between 1 – 2 cfs/ft.

St Charles Parish Sections Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
SC08	Bayou Trepagnier Pump Station	Structure/Wall	Future	18.5	15.4	0.156
SC11	Bonnet Carre Tie-in Floodwall	Structure/Wall	Future	18.5	15.4	0.155
SC05	Good Hope Floodwall	Structure/Wall	Future	17.0	15.6	1.242
SC02-A	St.Charles Parish Levee west of I-310	Levee	Existing	15.5	14.0	1.672
SC02-A	St.Charles Parish Levee west of I-310	Levee	Future	18.0	15.5	1.167
SC07	Cross Bayou Canal T-Wall	Structure/Wall	Future	17.0	15.6	1.256
SC06	Gulf South Pipeline T-Wall	Structure/Wall	Future	17.0	15.7	1.277
SC04	St. Rose Canal Drainage Structure T-Wall	Structure/Wall	Future	16.5	16.0	1.531
SC12	I-310 Floodwall	Structure/Wall	Future	15.5	15.1	1.408
SC02-B	St.Charles Parish Levee east of I-310	Levee	Existing	14.0	13.5	1.889
SC02-B	St.Charles Parish Levee east of I-310	Levee	Future	16.0	15.0	1.498
SC09	Almedia Drainage Structure	Structure/Wall	Future	15.5	15.0	1.375
SC10	Walker Drainage Structure	Structure/Wall	Future	15.5	14.9	1.348
SC13	Armstrong Airport Floodwall	Structure/Wall	Future	15.5	14.8	1.204
SC14	ICRR Floodgate	Structure/Wall	Future	15.5	14.8	1.188
SC30	Transition	Structure/Wall	Future	16.5	14.7	0.847
SC01-A	St. Charles Return Levee/Wall	Structure/Wall	Future	17.5	13.7	0.413
SC15	Shell Pipeline Floodwall	Structure/Wall	Future	17.0	15.6	1.248

Table 11 – Resiliency for St. Charles Parish Segments

3.3 Jefferson Parish Lakefront

3.3.1 General

The Jefferson Parish lakefront portion of the Lake Pontchartrain, LA and Vicinity Hurricane Protection System is located along the bank of Lake Pontchartrain. This levee runs in an east-west direction from the 17th Street Canal at the Orleans - Jefferson Parish Line to the Jefferson - St. Charles Parish Return Levee. Along this alignment are 4 pumping stations, a section of recurved wall on the levee at the western end of the segment, and several recreation areas at Bonnabel, Williams and Causeway Blvds. The levee length is approximately 10.4 miles. The existing levee was constructed to withstand the Standard Project Hurricane (SPH). Figure 26 shows the levee segments and pumping stations analyzed for the Jefferson Parish lakefront levee.

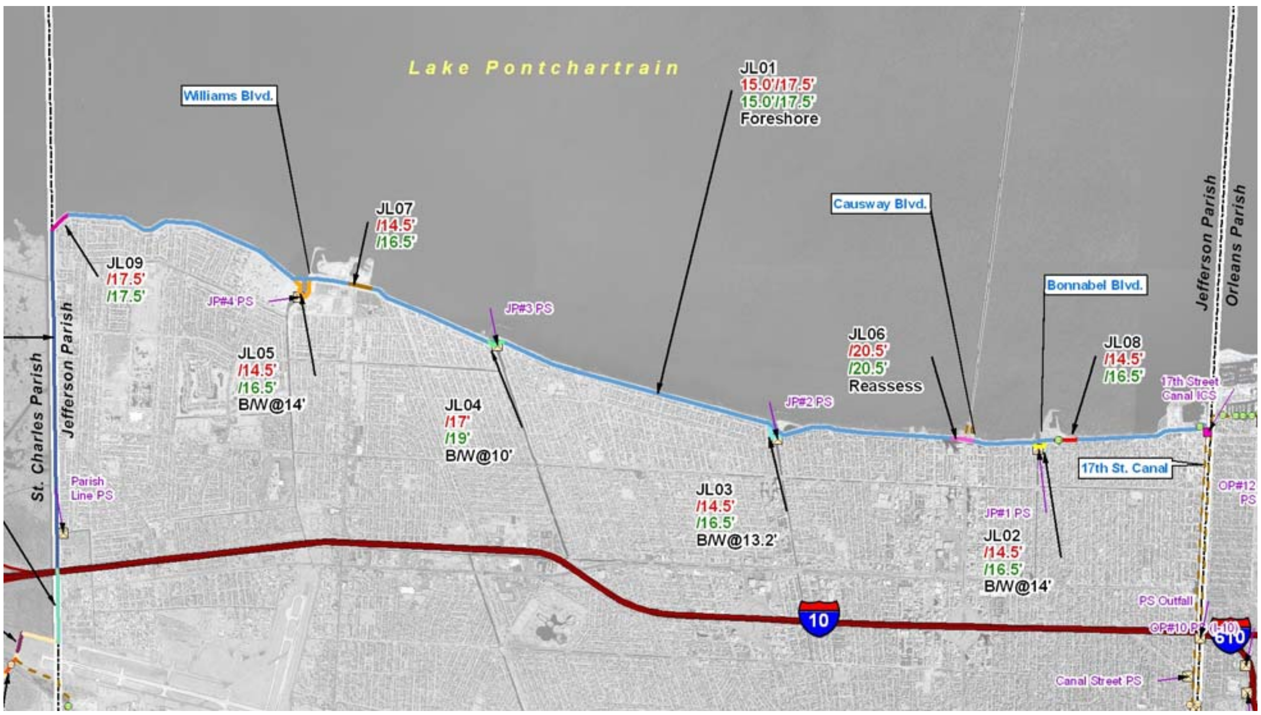


Figure 26 – Levee and floodwall sections in Jefferson Parish lakefront. The numbers represent existing/future conditions and are without (red) and with (green) structural superiority.

3.3.2 Hydraulic Boundary Conditions

The hydraulic design characteristics for the sections in Jefferson Parish lakefront are listed in Table 12. The existing hydraulic conditions are based on the JPM-OS method using the results from ADCIRC and STWAVE. The future conditions are derived by adding 1.5 ft to the surge elevation, and adding 0.75 ft to the wave height. The wave period is computed using the assumption that the wave steepness remains constant. For more information, see Chapter 2.

The offshore 1% hydraulic wave heights have been changed due to the presence of breakwaters in front of the pump stations (JL02 – JL05) and due to the shallow foreshore (JL01 and JL06 – JL09). This will be explained further below.

Jefferson Parish Sections 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
JL01	Lakefront levee	Levee	Existing	9.0	0.6	3.6	0.4	7.7	1.5
JL01	Lakefront levee	Levee	Future	10.5	0.6	4.2	0.4	8.3	1.5
JL02	Pump station 1 with breakwater at 14ft	Structure/Wall	Future	10.3	0.7	2.5	0.3	8.1	1.6
JL03	Pump station 2 with breakwater at 13.2ft	Structure/Wall	Future	10.4	0.7	2.8	0.3	8.1	1.6
JL04	Pump station 3 with breakwater at 10ft	Structure/Wall	Future	10.5	0.6	4.2	0.4	8.1	1.6
JL05	Pump station 4 with breakwater at 14ft	Structure/Wall	Future	10.5	0.7	2.5	0.3	8.1	1.6
JL06	Causeway Crib wall	Structure/Wall	Future	10.3	0.7	6.5	0.6	7.8	1.5
JL07	Williams Blvd Floodgate	Structure/Wall	Future	10.4	0.6	2.8	0.2	8.5	1.5
JL08	Bonnabel Boat Launch Floodgate	Structure/Wall	Future	10.3	0.7	2.7	0.2	8.3	1.5
JL09	Return wall	Structure/Wall	Future	10.8	0.7	4.9	0.4	8.3	1.6

Table 12 – Jefferson Parish Lakefront Segments – 1% Hydraulic Boundary Conditions

There are four pump stations along the Jefferson Parish Lakefront Levee. The typical configuration is shown in Figure 27. Pump Stations 1 and 4 are not protected from waves with breakwaters at present. Pump Stations 2 and 3 have breakwaters that transform and reduce the waves. The fronting protection connects with the tie-in walls to form a continuous wall of protection. The entire wall structure currently has an overall length of approximately 1,070 ft. In the design analysis, the wall height was extended to prevent overtopping. The tie-in wall sections labeled A, C, and E are subjected to more direct wave attack from Lake Pontchartrain than the other walls, as seen in Figure 27. Accordingly, the wave conditions were computed for Section C, and the final design grade obtained for that analysis was then applied to the other sections, assuming that Section C was the most vulnerable section.

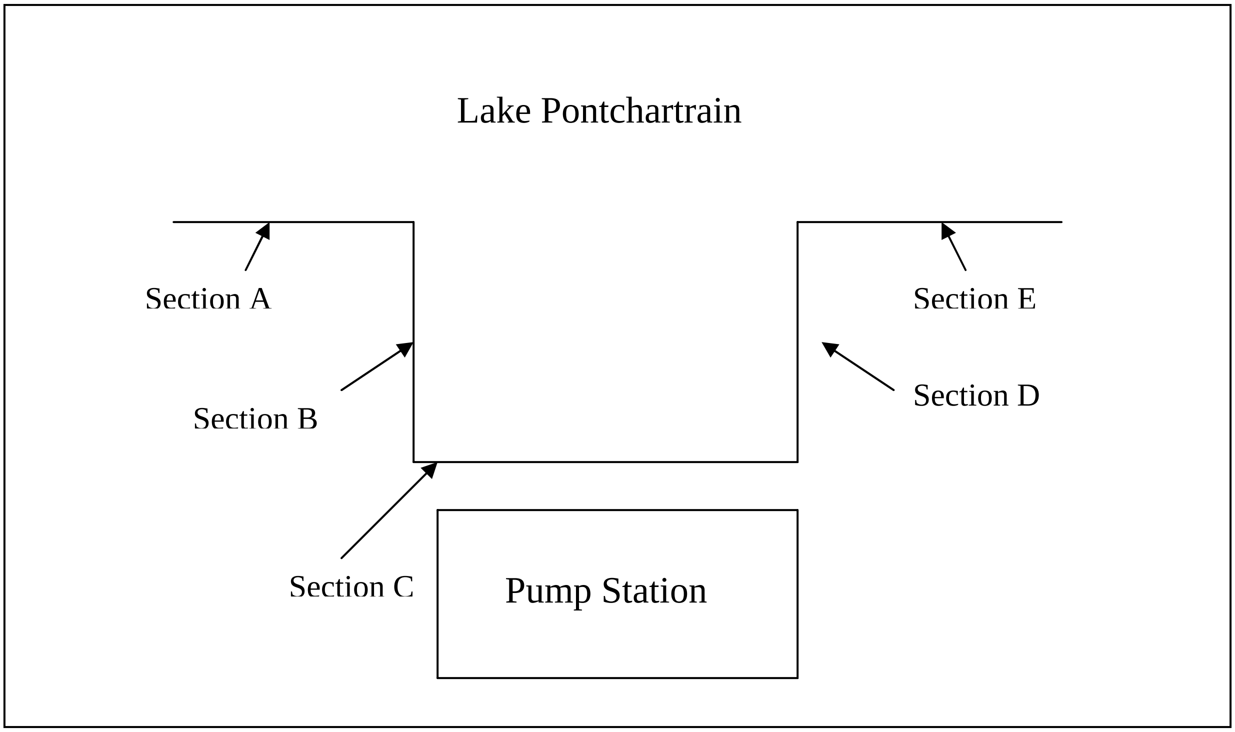


Figure 27 – Situation sketch for Pump Station 1

At present, it is assumed that an impermeable breakwater will be present as fronting protection with a design elevation of +14ft for Pump Station 1 and 4, an elevation of +13.2ft for Pump Station 2 and an elevation of +10ft for Pump Station 3. The breakwaters are vertical walls placed in front of the pump stations at an average bottom surface elevation of -5 ft with riprap protection 2 ft above the toe. The incoming wave height and peak period are almost the same for all pump stations. Herein, we have used H_s = 8.3ft and T_p = 8.1 s for the incoming future wave characteristics at all pump stations. Transmitted wave heights were computed using Automated Coastal Engineering System (ACES) software and the resulting transmitted wave height is listed in Table 12. It was assumed that the wave period would not be affected.

Because of the shallowness of the foreshore, the 1% wave height has been reduced for the levee section (JL01) and the various floodgates and floodwalls (JL06 – JL09). An average elevation of the existing ground in front of the floodwalls, over a distance of approximately one wave length, was used to adjust wave height. The wave height was established as 40 percent of the design water depth. The following is a brief description of the land features.

Lakefront Levee (Section JL01): An average elevation of 0.0ft NAVD88 2004.65 was assumed for the foreshore elevation.

Causeway Crib Walls (Section JL06): The “Crib Wall” is built on the fill that was used to extend the Causeway Bridge approach out into Lake Pontchartrain. An average elevation of -6.0 ft NAVD88 2004.65 was assumed for the foreshore elevation.

Williams Blvd. Floodgate (Section JL07): Land in front of the floodwall varies from as high as elevation +8.5 ft to as low as +2.5 ft over a distance of about 330 ft. An average elevation of +3.5 ft NAVD88 2004.65 was assumed.

Bonnabel Boat Launch Floodgate (Section JL08): Land in front of the floodwall varies from as high as elevation +8.0 ft to as low as +2.5 ft over a distance of about 525 ft. An average elevation of +3.5 ft NAVD88 2004.65 was assumed.

Return floodwall at border St. Charles – Jefferson Parish (Section JL09): The foreshore in front of the return floodwall varies. An elevation of -1 ft NAVD88 2004.65 was assumed for this section.

3.3.3 Project Design Heights

The design characteristics for the sections in Jefferson Parish Lakefront, including levees, floodwalls, gates and pump stations are listed Table 13. Section JL01 is a levee, the remainder sections are structures. Note that these structures are only evaluated for future conditions, because these are hard structures.

Jefferson Parish Sections 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
JL01	Lakefront levee	Levee	Existing	9.0	15.0	0.007	0.056
JL01	Lakefront levee	Levee	Future	10.5	17.5	0.006	0.051
JL02bw	Pump station 1	Structure/Wall	Future	15.3	16.5	0.000	0.002
JL03bw	Pump station 2	Structure/Wall	Future	15.4	16.5	0.001	0.005
JL04bw10	Pump station 3	Structure/Wall	Future	15.5	19.0	0.003	0.011
JL05bw	Pump station 4	Structure/Wall	Future	15.5	16.5	0.000	0.002
JL06	Causeway Crib wall	Structure/Wall	Future	16.3	20.5	0.020	0.059
JL07	Williams Blvd Floodgate	Structure/Wall	Future	6.9	16.5	0.000	0.003
JL08	Bonnabel Boat Launch Floodgate	Structure/Wall	Future	6.8	16.5	0.000	0.003
JL09	Return wall	Structure/Wall	Future	12.3	17.5	0.028	0.086

Table 13 – Jefferson Parish Lakefront Segments - 1% Design Information

The Jefferson Lakefront Levee design elevation and a typical design cross section are shown in Figure 28 for present and future conditions. A 0.0 ft elevation was assumed at the toe of the levee. A levee height of +15.0 ft for segment JL01 meets the overtopping criteria for the existing conditions in combination with the depicted configuration in Figure 28. The wave berm with a 1:15 slope is an important element to reduce the wave overtopping. The levee cross section must be modified to withstand future conditions with the crest must be raised to +17.5 ft.

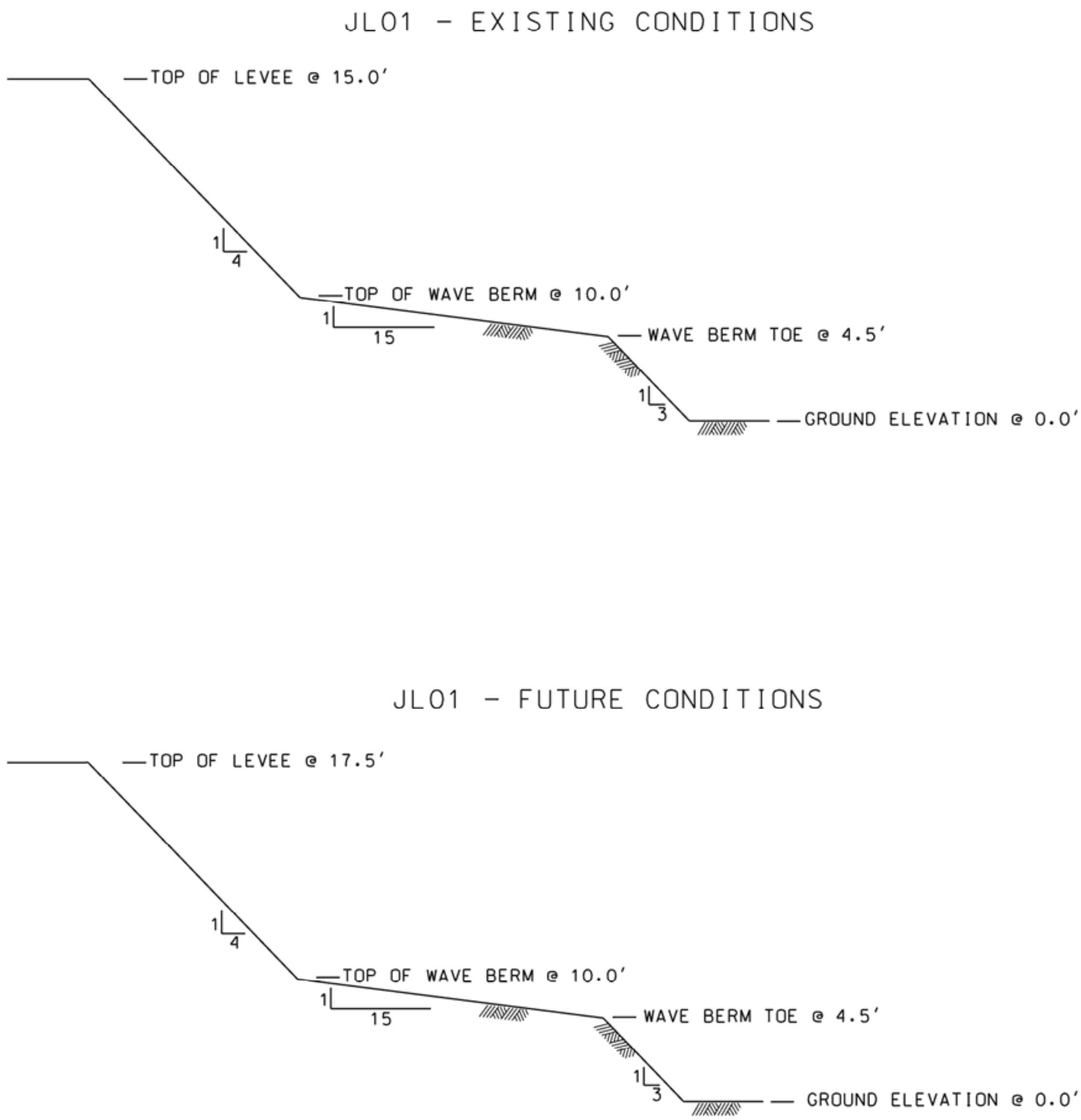


Figure 28 - Typical cross-section design profile for Jefferson Parish Lakefront Levees in NAVD88 2004.65 for existing conditions (upper panel) and future conditions (lower panel).

The design height of the pump stations equals +16.5ft (JL02, JL03, JL05) with a +14ft breakwater in front of these stations. The design height for pump station (JL04) is +19ft with a +10ft breakwater. The design height of these pump stations includes 2ft of structural superiority. The elevations of the tie-in walls near the pump stations were selected to be the same as the fronting protection elevations at each pump station. At all pump stations, the floodwall tie-ins are earthen berms up to +8 ft with 1V:3H slopes. The top of wall elevation is equivalent to the design height of that specific pump station, respectively.

The design height at the Causeway Crib Wall (JL06) needs to be +20.5ft to meet the design criteria for overtopping. Notice that the incoming waves are relatively high compared with the other sections because of the deep foreshore resulting in a high elevation. The design heights of the floodgates at Williams Blvd Floodgate (JL07) and Bonnabel Blvd Floodgate (JL08) are +16.5ft. These floodgates include structural superiority of 2ft.

Initially, the typical cross section existing in the field in August 2006 was used for Section JL09. This recurved wall cross section represents a 1,160-foot segment at the far western end of the Jefferson Parish Lakefront Levee. Current levee crest elevation varies from about +15 to +16 ft NAVD88 2004.65. Based on the analysis the current height will not be enough to meet the criteria of the overtopping rate. Therefore, it is proposed to replace the recurved wall with a floodwall with a design height of +17.5ft.

3.3.4 Wave Forces

Wave forces were computed for all structures within the Jefferson Parish segment with the Goda method, using future conditions. The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics. The wave force computation at the breakwaters in front of the pumping stations is still work in progress.

The following tables summarize the resulting wave forces. Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

Jefferson Parish Sections Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
JL02	Fronting protection Pump station 1	5.4	70.6	2.0	5.4	70.6	2.0
JL03	Fronting protection Pump station 2	6.5	94.1	1.4	6.5	94.1	1.4
JL04	Fronting protection Pump station 3	9.9	141.4	3.2	9.9	141.4	3.2
JL05	Fronting protection Pump station 4	5.5	72.5	2.0	5.5	72.5	2.0
JL06	Causeway Crib wall	14.5	180.8	6.5	14.5	180.8	6.5
JL07	Williams Blvd Floodgate	0.8	2.3	11.2	0.8	2.3	11.2
JL08	Bonnabel Boat Launch Floodgate	0.0	0.0	0.0	0.0	0.0	0.0
JL09	Return wall	6.7	45.2	9.7	6.7	45.2	9.7

Table 14 – Waves Forces for Jefferson Parish Lakefront Segments (50% values)

Jefferson Parish Sections Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
JL02	Fronting protection Pump station 1	7.3	96.9	2.3	7.3	96.9	2.3
JL03	Fronting protection Pump station 2	8.8	128.7	1.6	8.8	128.7	1.6
JL04	Fronting protection Pump station 3	13.1	188.5	3.3	13.1	188.5	3.3
JL05	Fronting protection Pump station 4	7.3	98.8	2.3	7.3	98.8	2.3
JL06	Causeway Crib wall	18.4	232.5	6.6	18.4	232.5	6.6
JL07	Williams Blvd Floodgate	1.0	2.9	11.3	1.0	2.9	11.3
JL08	Bonnabel Boat Launch Floodgate	0.0	0.0	0.0	0.0	0.0	0.0
JL09	Return wall	8.4	57.8	9.9	8.4	57.8	9.9

Table 15 – Waves Forces for Jefferson Parish Lakefront Segments (90% values)

3.3.5 Resiliency

The designs for the levees and structures within Jefferson Parish were examined for resiliency by also computing the overtopping rate for the 0.2 percent event for each design. The water level and overtopping rate was determined for the 50% assurance during the 0.2% event. The results are presented in Table 16. For all sections, the 0.2% surge elevation remains below the top of the flood defense. Apart from the return wall (JL09), the maximum overtopping rate during the 0.2% event is less than 0.2 cfs/ft per ft (best estimates).

Jefferson Parish Sections Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
JL01	Lakefront levee	Levee	Existing	15.0	11.2	0.320
JL01	Lakefront levee	Levee	Future	17.5	12.7	0.220
JL02bw	Pump station 1	Structure/Wall	Future	16.5	12.7	0.008
JL03bw	Pump station 2	Structure/Wall	Future	16.5	12.7	0.018
JL04bw10	Pump station 3	Structure/Wall	Future	19.0	12.7	0.018
JL05bw	Pump station 4	Structure/Wall	Future	16.5	12.8	0.009
JL06	Causeway Crib wall	Structure/Wall	Future	20.5	12.7	0.182
JL07	Williams Blvd Floodgate	Structure/Wall	Future	16.5	12.6	0.055
JL08	Bonnabel Boat Launch Floodgate	Structure/Wall	Future	16.5	12.7	0.064
JL09	Return wall	Structure/Wall	Future	17.5	13.1	0.362

Table 16 – Resiliency for Jefferson Parish Lakefront Segments

3.4 Orleans Parish – Metro Lakefront

3.4.1 General

The Orleans Parish Metro portion of the Lake Pontchartrain, LA and Vicinity Hurricane Protection System is shown in Figure 29. This section deals with the Orleans Metro Lakefront Levee, which covers the lakefront from Jefferson Parish line to IHNC (Inner Harbor Navigation Canal). The levee length is approximately 4 miles. This flood protection system partly consists of levees and partly of floodwalls. Notice that Section 3.5 covers the New Orleans East Lakefront segment to South Point, whereas Section 3.6 discusses the sections between South Point to GIWW and the sections along GIWW and IHNC in the Orleans Parish.

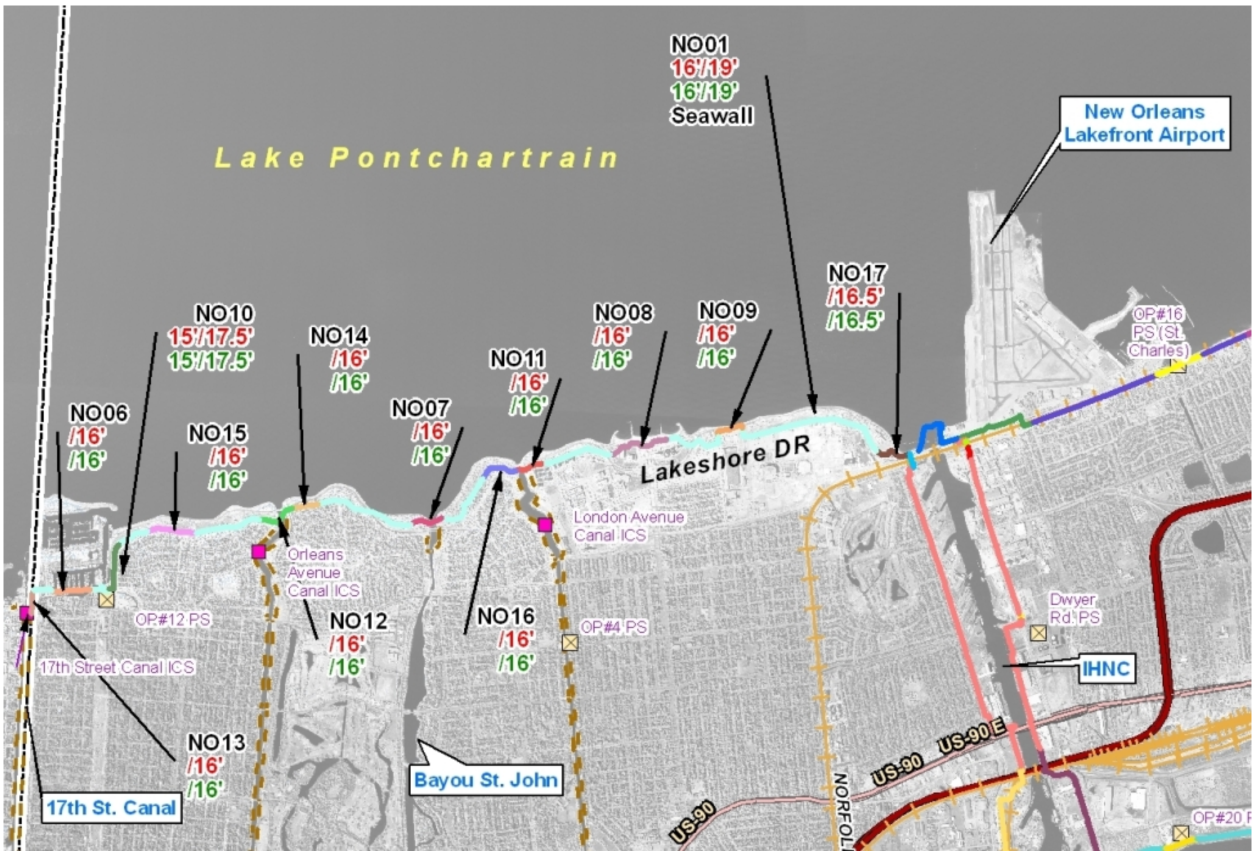


Figure 29 – Levee and floodwall sections in Orleans Parish Metro Lakefront. The numbers represent existing/future conditions and are without (red) and with (green) structural superiority.

3.4.2 Hydraulic Boundary Conditions

The hydraulic design characteristics for the sections in Metro Lakefront in Orleans Parish are listed in Table 17. The existing hydraulic conditions are based on the JPM-OS method using the results from ADCIRC and STWAVE. The future conditions are derived by adding 1.5 ft to the surge elevation, and adding 0.75 ft to the wave height. The wave period is computed using the assumption that the wave steepness remains constant. For more information, see Chapter 2.

Orleans Parish Metro Lakefront Sections 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
NO06	NO Marina	Structure/Wall	Future	10.2	0.7	3.3	0.3	8.0	1.4
NO10	Topaz St. Levee	Levee	Existing	8.7	0.7	2.3	0.2	7.2	1.4
NO10	Topaz St. Levee	Levee	Future	10.2	0.7	2.9	0.2	8.1	1.4
NO15	Type II Floodgate similar to Canal Blvd	Structure/Wall	Future	10.2	0.7	2.3	0.2	8.4	1.4
NO13	17th St. Outfall Canal Closure	Structure/Wall	Future	10.2	0.7	4.0	0.4	4.5	0.9
NO12	Orleans Ave Outfall Canal Closure	Structure/Wall	Future	10.2	0.8	3.0	0.3	4.0	0.8
NO14	Type I Floodgate Similar to Marconi Drive	Structure/Wall	Future	10.2	0.8	2.5	0.2	7.9	1.4
NO16	Lakeshore Drive near Rail St FG	Structure/Wall	Future	10.1	0.8	4.4	0.4	7.4	1.4
NO07	Bayou St. John	Structure/Wall	Future	10.1	0.8	3.0	0.3	4.0	0.8
NO11	London Ave Outfall Canal Closures	Structure/Wall	Future	10.1	0.8	3.0	0.3	4.0	0.8
NO08	Pontchartrain	Structure/Wall	Future	10.1	0.8	3.6	0.3	7.3	1.3
NO09	American Std FW	Structure/Wall	Future	10.1	0.8	4.4	0.4	7.1	1.3
NO01	New Orleans Lakefront Levee	Levee	Existing	8.7	0.7	5.1	0.5	7.2	1.4
NO01	New Orleans Lakefront Levee	Levee	Future	10.2	0.7	5.7	0.5	7.6	1.4
NO17	Leroy Johnson	Structure/Wall	Future	10.1	0.8	4.0	0.3	7.0	1.3

Table 17 – Orleans Parish Metro Lakefront Segments – 1% Hydraulic Boundary Conditions

Notice that for this area, the hydraulic boundary conditions have been based on the 2007 grid and without the Seabrook gate. The New Orleans Metro Lakefront is not affected by the gates. The offshore 1% hydraulic wave characteristics have been changed due to the presence of shallow foreshore and/or sheltered conditions. This will be explained further below.

The Orleans Metro Lakefront consists of 2 levee segments: the New Orleans Lakefront Levee (NO01) and Topaz St. (NO10). Segment NO01 runs from the 17th Street Canal at the Orleans - Jefferson Parish Line to the Inner Harbor Navigation Canal (IHNC). Segment NO10 runs in a north-south direction and extends south from Lake Pontchartrain along Lakeshore Drive in the vicinity of Topaz Drive. Segment NO10 is located immediately east of the New Orleans Marina, and is known as the Topaz Street Levee. At both sections, the wave height has been reduced because of the shallow foreshore.

Topaz St. (Section NO10): This levee segment is approximately 0.4 miles. Land elevations in this area are at an elevation of +3 ft NAVD88 2004.65. The wave height at the toe of the levee is assumed to be 40% of the local water depth.

Lakefront Levee (Section NO01): An average elevation of -4.0 ft NAVD88 2004.65 was assumed for the foreshore elevation in front of the sea wall. The wave height at the toe of the sea wall is assumed to be 40% of the local water depth. The stretch between the seawall and the actual levee varies between 85 and 1,000ft, and the elevation varies from +4 to +6.5ft NAVD88 2004.65. The shortest distance is taken as a reference point in the hydraulic design. Because the distance of 85 ft is much less than one wave length (≈ 300 ft), no further reduction of the wave height is included and the stretch between the seawall and the actual levee acts as a wave berm in the hydraulic design computations.

Besides the two levee sections, there are various floodwalls, closure structures and floodgates along the New Orleans Lakefront. Floodwalls and closure structures were looked at individually for this effort. An average elevation of the existing ground in front of the structure, over a distance of approximately one wave length, was used to adjust wave height. Wave height was established as 40 percent of the design water depth. The following is a brief description of the land features in front of the floodwalls, closure structures and floodgates at the New Orleans Lakefront.

New Orleans Marina Floodwall (NO06): Beginning at the lake, land elevation is +4.0 ft. The elevation descends for some distance then rises to elevation +3.5 ft, descends again and then rises at the floodwall berm to elevation +2.5 ft. A (conservative) elevation of +2.0 ft was assumed along a distance of more than 1,000ft.

Pontchartrain Beach Floodwall (NO08): Land in front of the floodwall varies from as high as elevation +5.0 ft to as low as +2.0 ft over a distance of about 180 ft. More lakeward the elevation is lower (0 to +2 ft). We have applied an average elevation of +1ft in the design computation at a distance of one wave length (≈ 300 ft) from the floodwall.

American Standard Floodwall (NO09): Land in front of the floodwall was originally at +6.0 ft. The land has significantly subsided since construction. The floodwall is about 100 ft from the lakeshore. The slope of the lake is mild (1:100 – 1:1000). Herein, we assume a 1:100 slope and have applied an elevation of -4.0 ft at a distance of one wave length (≈ 300 ft) from the floodwall. However, the seawall and the land just in front of the floodwall will partly break the waves. For this reason, we have applied an average elevation of -1ft for the area in front of the floodwall to account for this effect.

Bayou St. John Floodwall (NO07): The Bayou St. John floodwall is set back from the lake and is fronted by a highway. It was assumed that waves would be reduced to a random nature with a 3.0 ft wave height and a 4.0 second period. Future waves were adjusted based on increase in water depth.

Outfall Canals Closure Structures (NO11, NO12 and NO13): At the mouth of the three outfall canals are temporary closure structures in place until the permanent pump stations are built. They are somewhat sheltered from the waves from the lake with the 17th Street outfall canal being the most exposed. A 3 foot wave height with a 4 second wave period was used for the Orleans Avenue and London Avenue outfall canals and a 4 foot wave height with a 4.5 second wave period was used for the 17th Street Outfall Canal. This assumes that the pump stations are located inside the outfall canals where the temporary pump stations are located. If the pump stations are located closer towards the lake, the wave characteristics have to be re-evaluated.

Lakeshore Drive near Rail Street (NO16): The floodgate at Lakeshore Drive near Rail Street is located at the top of the existing ramp where Lakeshore Drive crosses the existing Lakefront levee. The base of the floodgate is at approximately 14.5 ft NAVD88 2004.65 and is close to the lakeshore (approximately 100ft). We have estimated an average elevation of -4ft in the design computation at a distance of one wave length (≈ 300 ft) from the floodgate. However, the seawall and the land just in front of the floodwall will partly break the waves. For this reason, we have applied an average elevation of -1ft for the area in front of the floodwall to account for this effect.

Lakeshore Drive near the Hickey Bridge Floodwall (NO17): The floodwall at Lakeshore Dr near the Hickey Bridge is located lakeward next to the Hickey Bridge near the IHNC. The base of the floodwall is at approximately +7.5 ft NAVD88 2004.65. An average elevation of 0ft was used at a distance of one wave length (≈ 300 ft) from the floodwall.

Marconi Drive floodgate (NO14): The average ground elevation one wave length from the toe of this floodgate is estimated to be +4.0 ft.

Canal Boulevard (NO15): The average ground elevation one wave length from the toe of this floodgate is estimated to be +4.5 ft.

3.4.3 Project Design Heights

The design characteristics for the sections in Orleans Parish, including levees, floodwalls, gates and pump stations are listed in Table 18. Sections NO01 (Metro Lakefront Levee) and NO10 (Topaz St.) are levees, the remainder sections are floodwalls or gates. Note that these structures are only evaluated for future conditions, because these are hard structures.

Orleans Parish Metro Lakefront Sections 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
NO06	NO Marina	Structure/Wall	Future	8.2	16.0	0.003	0.016
NO10	Topaz St. Levee	Levee	Existing	5.7	15.0	0.002	0.015
NO10	Topaz St. Levee	Levee	Future	7.2	17.5	0.005	0.029
NO15	Type II Floodgate similar to Canal Blvd	Structure/Wall	Future	5.7	16.0	0.000	0.001
NO13	17th St. Outfall Canal Closure	Structure/Wall	Future	10.2	16.0	0.015	0.056
NO12	Orleans Ave Outfall Canal Closure	Structure/Wall	Future	10.2	16.0	0.002	0.012
NO14	Type I Floodgate Similar to Marconi Drive	Structure/Wall	Future	6.2	16.0	0.000	0.002
NO16	Lakeshore Drive near Rail St FG	Structure/Wall	Future	11.1	16.0	0.028	0.097
NO07	Bayou St. John	Structure/Wall	Future	10.1	16.0	0.002	0.011
NO11	London Ave Outfall Canal Closures	Structure/Wall	Future	10.1	16.0	0.002	0.011
NO08	Pontchartrain	Structure/Wall	Future	9.1	16.0	0.007	0.033
NO09	American Std FW	Structure/Wall	Future	11.1	16.0	0.028	0.096
NO01	New Orleans Lakefront Levee	Levee	Existing	12.7	16.0	0.006	0.060
NO01	New Orleans Lakefront Levee	Levee	Future	14.2	19.0	0.008	0.066
NO17	Leroy Johnson	Structure/Wall	Future	10.1	16.5	0.009	0.038

Table 18 – Orleans Parish Metro Lakefront Segments - 1% Design Information

The 1% hydraulic design for the existing New Orleans Lakefront Metro levee (segment NO01) is shown in Figure 30. This levee segment is approximately 5.5 miles and is setback from the lakefront seawall from 85 to about 1000 ft. Land elevations in this setback area are at current elevation +3 to +5 ft depending upon location; land has subsided several feet since the original design. The current levee crest elevation is approximately +17 ft, although the pre-Katrina authorized design required elevations between +17.5 to +18.5 ft with 1 on 3 side slopes. Notice that the current cross-section with an elevation of +17ft fulfills the 100-year hydraulic design criteria.

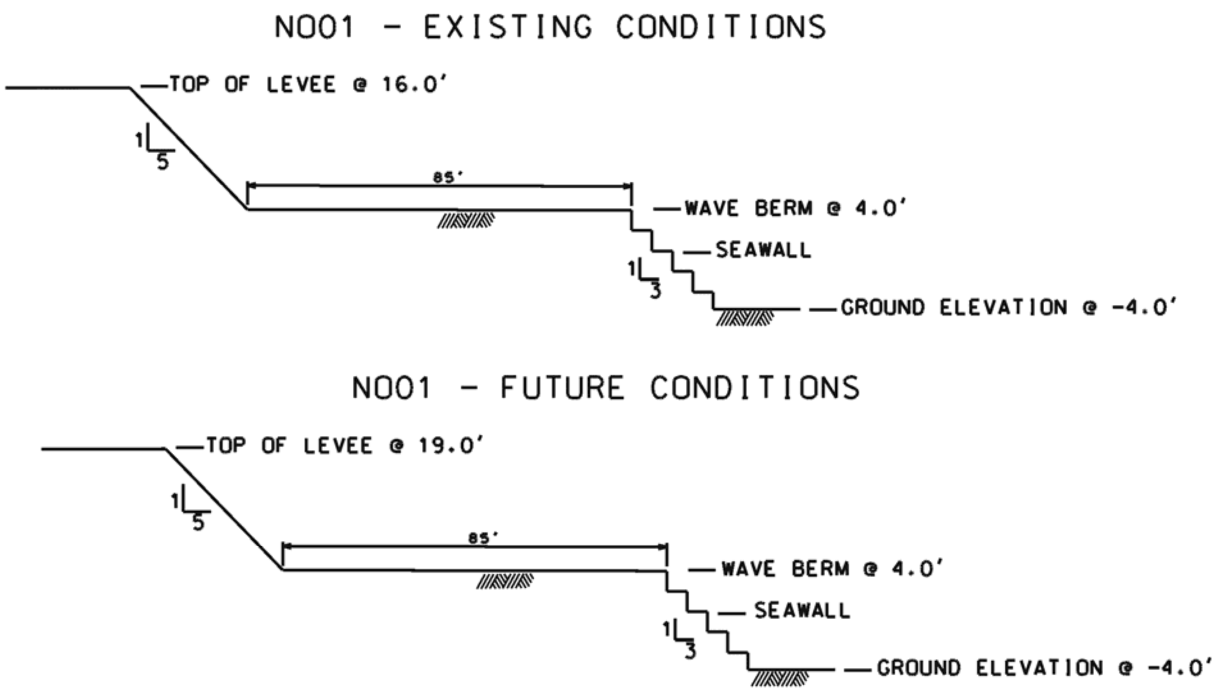


Figure 30 – Cross-section profile for New Orleans Metro Lakefront levee (NO01) for existing (upper panel) and future conditions (lower panel).

The hydraulic design section for the existing Topaz Street Levee (NO10) is shown in Figure 31. This levee segment is approximately 0.4 miles. Land elevations in this area are at an elevation of +3 ft. The existing levee slope of 1V:3H was also used for the proposed levee in this reach due to the limited space for expanding the levee footprint. The 1% design height is set at +15.0ft (existing conditions) and +17.0 ft (future conditions).

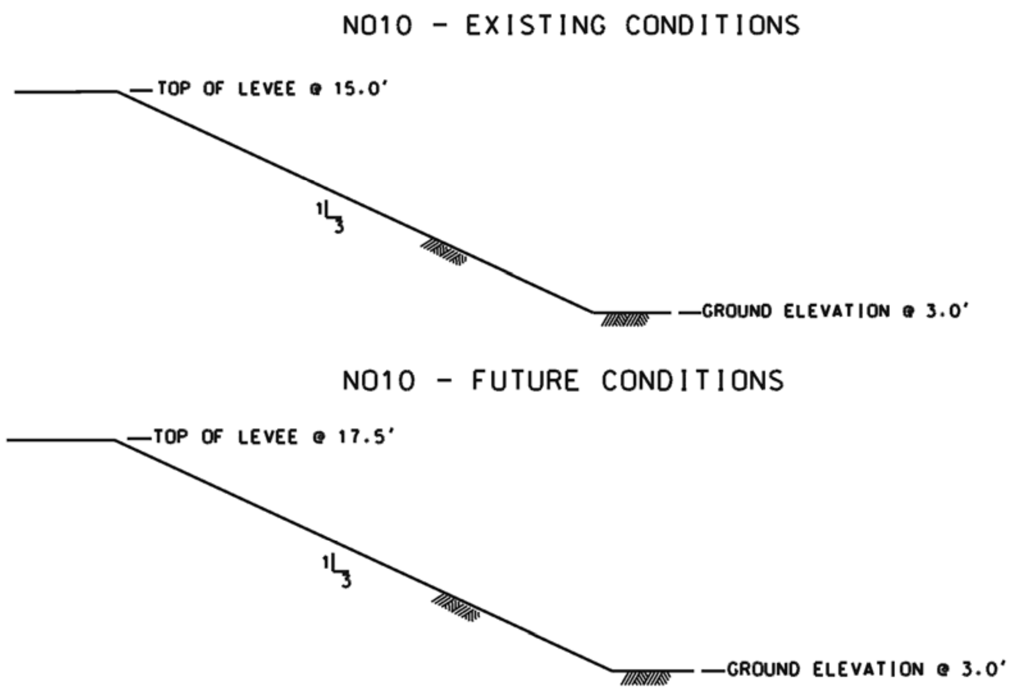


Figure 31 – Cross-section profile for Topaz Street Levee (NO10) for existing (upper panel) and future conditions (lower panel).

The various floodwalls and gates in the Orleans Parish Lakefront Metro area have design elevations ranging from +16 ft to +16.5ft for future conditions.

3.4.4 Wave Forces

Wave forces were computed for all structures within the Orleans Metro Lakefront segment with the Goda method, using future conditions. The characteristics of the floodgates could be grouped into two types of floodgates. The invert elevation and floodgate type are shown in Table 19 below for the floodgates located within the Orleans Metro area. For Type 1 structures a toe elevation of +4.5 ft has been applied and for Type 2 structures +4.0ft. A more detailed analysis will require the designer to look at each floodgate individually.

Orleans Metro Lakefront Floodgates		
Floodgate Name	Invert (Ft. NAVD88 2004.65)	Type
Lakeshore Drive near New Orleans Marina	4.7	1
Topaz Street	5.7	1
Canal Boulevard	13.0	2
Marconi Drive	6.0	1
Lake Terrace Drive	15.4	2
Lakeshore Drive West of London Avenue Canal	13.1	2
Lakeshore Drive West of Pontchartrain Beach	13.1	2
Lakeshore Drive East of Pontchartrain Beach	14.5	2
Franklin Avenue	13.4	2
Leroy Johnson Drive	14.3	2
Camp Leroy Johnson (NG) Entrance	9.8	1
Lakeshore Drive at Leon C. Simon Drive	9.8	1
Norfolk Southern RR West of the IHNC	5.4	1

Table 19 – Orleans Parish Metro Lakefront Floodgates with Invert Elevation and Floodgate Type

The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics, Table 20 and Table 21. The wave forces for the sections NO15 and NO16 are not listed because the invert level is above the still water level and the Goda formulation is not applicable. Further analysis of this special situation is recommended.

Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

New Orleans Metro Lakefront Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
NO06	NO Marina	4.1	23.6	8.8	4.1	23.6	8.8
NO15	Type II Floodgate similar to Canal Blvd	N/A	N/A	N/A	N/A	N/A	N/A
NO13	17th St. Outfall Canal Closure	5.8	101.9	2.6	5.8	101.9	2.6
NO12	Orleans Ave Outfall Canal Closure	3.9	69.6	2.9	3.9	69.6	2.9
NO14	Type I Floodgate Similar to Marconi Drive	2.2	8.8	10.0	2.2	8.8	10.0
NO16	Lakeshore Drive near Rail St FG	N/A	N/A	N/A	N/A	N/A	N/A
NO07	Bayou St. John	3.4	43.8	4.9	3.4	43.8	4.9
NO11	London Ave Outfall Canal Closures	3.9	69.0	2.9	3.9	69.0	2.9
NO08	Pontchartrain	3.6	18.5	9.6	3.6	18.5	9.6
NO09	American Std FW	3.6	18.3	9.6	3.6	18.3	9.6
NO17	Leroy Johnson	1.1	3.5	10.7	1.1	3.5	10.7

Table 20 – Waves Forces for Orleans Metro Lakefront Segments (50% values)

New Orleans Metro Lakefront Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
NO06	NO Marina	5.0	30.1	9.1	5.0	30.1	9.1
NO15	Type II Floodgate similar to Canal Blvd	N/A	N/A	N/A	N/A	N/A	N/A
NO13	17th St. Outfall Canal Closure	8.2	137.3	3.5	8.2	137.3	3.5
NO12	Orleans Ave Outfall Canal Closure	5.4	92.4	3.3	5.4	92.4	3.3
NO14	Type I Floodgate Similar to Marconi Drive	2.8	12.2	10.4	2.8	12.2	10.4
NO16	Lakeshore Drive near Rail St FG	N/A	N/A	N/A	N/A	N/A	N/A
NO07	Bayou St. John	4.8	61.1	5.2	4.8	61.1	5.2
NO11	London Ave Outfall Canal Closures	5.4	91.9	3.2	5.4	91.9	3.2
NO08	Pontchartrain	4.6	24.4	9.8	4.6	24.4	9.8
NO09	American Std FW	4.5	24.2	9.8	4.5	24.2	9.8
NO17	Leroy Johnson	1.3	4.5	10.9	1.3	4.5	10.9

Table 21 – Waves Forces for Orleans Metro Lakefront Segments (90% values)

3.4.5 Resiliency

The designs for the levees and structures within Orleans Parish – Metro Lakefront were examined for resiliency by also computing the overtopping rate for the 0.2 percent event for each design. The water level and overtopping rate was determined for the 50% assurance during the 0.2% event. The results are presented in Table 22. For all sections, the 0.2% surge elevation remains below the top of the flood defense, and the overtopping rate is less than 1 cfs/ft per ft (best estimates).

Orleans Parish Metro Lakefront Sections Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
NO06	NO Marina	Structure/Wall	Future	16.0	12.8	0.244
NO10	Topaz St. Levee	Levee	Existing	15.0	11.3	0.314
NO10	Topaz St. Levee	Levee	Future	17.5	12.8	0.323
NO15	Type II Floodgate similar to Canal Blvd	Structure/Wall	Future	16.0	12.8	0.074
NO13	17th St. Outfall Canal Closure	Structure/Wall	Future	16.0	12.8	0.211
NO12	Orleans Ave Outfall Canal Closure	Structure/Wall	Future	16.0	13.1	0.076
NO14	Type I Floodgate Similar to Marconi Drive	Structure/Wall	Future	16.0	13.1	0.156
NO16	Lakeshore Drive near Rail St FG	Structure/Wall	Future	16.0	13.1	0.854
NO07	Bayou St. John	Structure/Wall	Future	16.0	13.1	0.075
NO11	London Ave Outfall Canal Closures	Structure/Wall	Future	16.0	12.9	0.059
NO08	Pontchartrain	Structure/Wall	Future	16.0	12.9	0.396
NO09	American Std FW	Structure/Wall	Future	16.0	12.8	0.648
NO01	New Orleans Lakefront Levee	Levee	Existing	16.0	11.3	0.335
NO01	New Orleans Lakefront Levee	Levee	Future	19.0	12.8	0.276
NO17	Leroy Johnson	Structure/Wall	Future	16.5	12.8	0.336

Table 22 – Resiliency for Orleans Parish Metro Lakefront

3.5 Orleans Parish – Lakefront East

3.5.1 General

The Orleans Parish portion of the Lake Pontchartrain, LA and Vicinity Hurricane Protection System is shown in Figure 29. This section deals with the New Orleans East Lakefront segment to South Point. It consists of two large levee sections (Citrus Lakefront Levee and New Orleans East Lakefront levee) with several small stretches of floodwalls and structures in between. The levee length is approximately 9 miles. Along the entire stretch a railroad, a breakwater, and a foreshore protection exist that reduce the overtopping. These elements are considered to be part of the flood protection for existing and future conditions.

Figure 32 – Levee and floodwall sections in Orleans Parish Lakefront East. The numbers represent existing/future conditions and are without (red) and with (green) structural superiority.

3.5.2 Hydraulic Boundary Conditions

The hydraulic design characteristics for the sections in Orleans Parish – Lakefront East are listed in Table 23. The existing hydraulic conditions are based on the JPM-OS method using the results from ADCIRC and STWAVE. The future conditions are derived by adding 1.5 ft to the surge elevation, and adding 0.75 ft to the wave height. The wave period is computed using the assumption that the wave steepness remains constant. For more information, see Chapter 2. Notice that the hydraulic boundary conditions have been based on numerical computations using the 2007 grid without the Seabrook gate for the New Orleans East Lakefront because the gates appear to have no effect on the hydraulic boundary conditions in this area.

The offshore 1% hydraulic wave characteristics have been changed due to the presence of shallow foreshore and/or sheltered conditions. This will be explained further below.

Orleans Parish East Lakefront Sections 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
NE04	NO Lakefront Airport West	Structure/Wall	Future	10.0	0.7	3.2	0.3	7.5	1.4
NE03	NO Lakefront Airport East	Structure/Wall	Future	9.9	0.7	3.2	0.3	7.4	1.3
NE09	St Charles Pump station	Structure/Wall	Future	9.9	0.7	4.0	0.3	7.3	1.3
NE07	Citrus Pump station	Structure/Wall	Future	10.0	0.7	4.0	0.3	7.2	1.3
NE01	Citrus Lakefront Levee	Levee	Existing	8.6	0.7	2.0	0.2	6.7	1.3
NE01	Citrus Lakefront Levee	Levee	Future	10.1	0.7	2.5	0.3	7.1	1.4
NE08	Jahncke Pump station	Structure/Wall	Future	10.0	0.7	4.0	0.3	7.3	1.3
NE05	Lincoln Beach	Structure/Wall	Future	10.1	0.7	2.4	0.2	7.6	1.3
NE06	Collins Pipeline Crossing	Structure/Wall	Future	10.4	0.7	3.8	0.3	7.1	1.3
NE30	Transition Reach NE01 to NE02	Levee	Existing	8.6	0.7	2.9	0.3	6.7	1.3
NE30	Transition Reach NE01 to NE02	Levee	Future	10.1	0.7	3.4	0.3	7.1	1.4
NE02	New Orleans East Lakefront Levee	Levee	Existing	8.9	0.7	3.7	0.4	6.7	1.3
NE02	New Orleans East Lakefront Levee	Levee	Future	10.4	0.7	4.3	0.4	7.1	1.4
NE31	South Point transition reach	Levee	Existing	9.0	0.8	3.7	0.4	6.7	1.3
NE31	South Point transition reach	Levee	Future	10.5	0.8	4.3	0.4	7.1	1.4

Table 23 – Orleans Parish Lakefront East Segments – 1% Hydraulic Boundary Conditions

Citrus Lakefront Levee (NE01): The current Citrus Lakefront levee cross-section is shown in Figure 33. This levee runs in an east-west direction from the IHNC eastward to Paris Road. The existing levee crest elevation is approximately +12 to +13 ft with the breakwater at approximately +9 to +10 ft, although the pre-Katrina authorized design required an elevation of +15 ft for the levee and +13.5 ft for the breakwater.

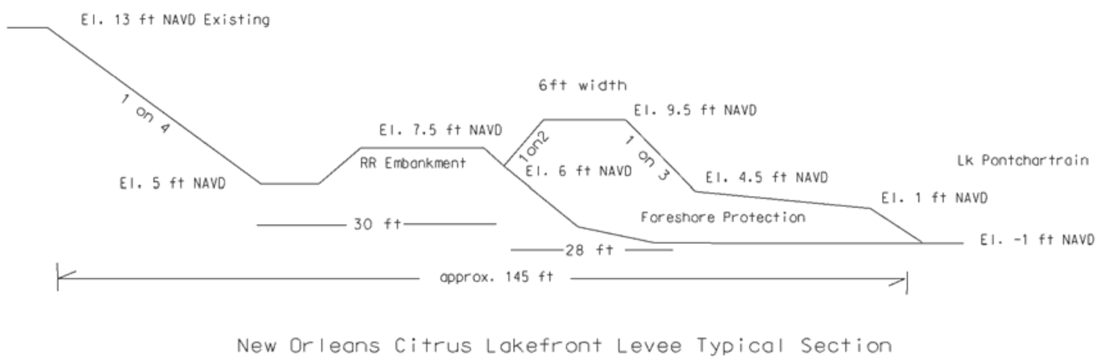


Figure 33 – Current Cross-Section Profile Citrus Lakefront Levee (NE01).

The offshore wave heights of 6-7 ft cannot be supported in the depths at the toe of the breakwater structure. So the design wave heights at the toe were reduced, using a maximum wave height of 40 percent of the design water depth as the depth-limiting criterion. The waves would be further reduced by the breakwater. The current breakwater, with an estimated elevation of +9 to +10 ft, provides substantial wave reduction for both existing conditions and future conditions.

The designs for the Citrus Lakefront levees in this report were based on the assumption that the breakwater would be maintained at the current elevation (+9ft). Transmitted wave heights through the breakwater were computed using ACES. The 1% significant wave height behind the breakwater for existing conditions turns out to be around 2ft, whereas the wave height for future conditions is about 2.5ft. The incoming wave period of about 7 s has not been changed due to the presence of the breakwater.

The railroad between the breakwater and the Citrus Lakefront levee acts as a wave berm. The current elevation of the railroad is at least +6 ft and its width is at least 40ft. These dimensions have been applied in the hydraulic design. Hence, maintaining the railroad at an elevation +6 ft and a width of 30ft is a prerequisite for the presented hydraulic designs in this report.

Transition levee between Citrus Lakefront and New Orleans East Lakefront (NE30): At Paris Road a transition is proposed for a reach between the Citrus Lakefront Levee and the New Orleans Lakefront Levee. The wave height behind the breakwater is set at 3ft (existing conditions) and 3.5 ft (future conditions) (average value of NE01 and NE02). The hydraulic design in this report assumes that the railroad dimensions are maintained at least an elevation of +6ft and a width of 40 ft and the breakwater in front of the railroad at +7.5ft. These dimensions are included in the hydraulic computation of the overtopping rate for this levee section.

New Orleans East Lakefront Levee (NE02): The current New Orleans East Lakefront levee typical cross section is shown in Figure 34. This levee runs in an east-west direction from Paris Road eastward to South Point. This levee segment is approximately 6.2 miles. The current levee fronted by the Norfolk Southern Railroad and foreshore protection. This segment has one section of floodwall/levee combination at the Collins Pipeline crossing.

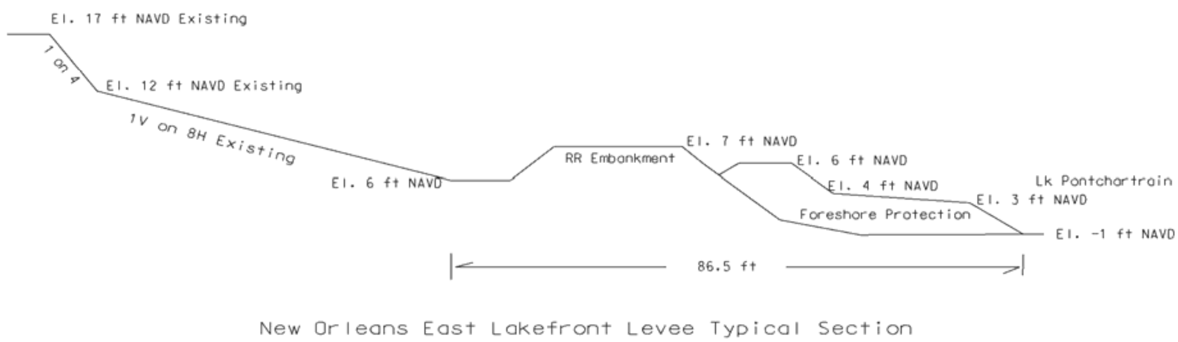


Figure 34 – Current cross-section profile New Orleans East Lakefront Levee (NE02).

The offshore wave heights of 6-7 ft cannot be supported in the depths at the toe of the foreshore protection structure. So the design wave heights at the toe were reduced, using a maximum wave height of 40 percent of the design water depth as the depth-limiting criteria. The waves would be further reduced by the foreshore protection and the railroad. The existing foreshore protection provides substantial wave reduction for both existing conditions and future conditions.

The designs for the New Orleans East Lakefront levee in this report were based on the assumption that the foreshore protection would be maintained at the existing elevation (+6ft). Transmitted wave heights through the foreshore protection were computed using ACES. The 1% significant wave height behind the foreshore protection for existing conditions is calculated to be around 3.7ft, whereas the wave height for future conditions is about 4.3ft. The incoming wave period of about 7 s has not been changed due to the presence of the foreshore protection.

The railroad in front of the New Orleans East Lakefront levee acts as a wave berm and will further reduce the wave height. The hydraulic design in this report assumed that the railroad dimensions are maintained at least an elevation of +6ft and a width of 40ft. These dimensions are included in the hydraulic computation of the overtopping rate for this levee section.

The New Orleans Lakefront Airport Floodwall and the Lincoln Beach Floodwall along with three pump stations are located along this segment of levee. An average elevation of the existing ground in front of the floodwalls, over a distance of approximately one wave length, was used to adjust wave height. Wave height was established as 40 percent of the design water depth. The following is a brief description of the land features.

Transition levee New Orleans East Lakefront levee and South Point to US Highway 90 levee (NE31): This stretch forms the transition between the New Orleans Lakefront Levee and the South Point to US Highway 90 levee (see Section 3.6). An average elevation of 0ft in front of the levee is assumed for this design.

New Orleans Lakefront Airport Floodwall (NE03 and NE04): Beginning at the lake, land elevation is +4.0 ft. The elevation ascends for some distance to elevation +4.5 ft, descends again and then rises at the floodwall berm to elevation +4.0 ft for a minimum distance of 400 ft. However, this only holds for waves coming perpendicular to the shoreline. In case of waves coming from the northwest or from the northeast, the sheltering effect of the Lakefront Airport is probably less because of the shorter distance to the lake. To be conservative, we have assumed an elevation of 0.0 ft NAVD88 2004.65 at one wave length from the floodwall.

Jahncke, St. Charles and Citrus Pump station (NE05, NE09, NE07): An average elevation of 0.0 ft NAVD88 2004.65 was assumed in front of the pump stations.

Lincoln Beach Floodwall (NO05): Land in front of the floodwall gradually slopes upward from the lake to an elevation of +4.4 ft over a distance of about 500 ft. An average elevation of +4.0 ft NAVD88 2004.65 was assumed at the toe of the floodwall.

Collins Pipeline crossing Floodwall (NO06): An average elevation of +1.0 ft NAVD88 2004.65 was assumed in front of this floodwall.

3.5.3 Project Design Heights

The design characteristics for the sections in Orleans Parish – Lakefront East are listed in Table 24. Sections NE01, NE02, NE30 and NE31 are levees, the remainder sections are floodwalls or structures. Note that these structures are only evaluated for future conditions, because these are hard structures. The only structure that includes structural superiority of 2ft is NO East Lakefront Collins Pipeline Crossing (NE06).

Orleans Parish East Lakefront Sections 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
NE04	NO Lakefront Airport West	Structure/Wall	Future	8.0	15.5	0.004	0.019
NE03	NO Lakefront Airport East	Structure/Wall	Future	7.9	15.5	0.003	0.015
NE09	St Charles Pump station	Structure/Wall	Future	9.9	15.5	0.017	0.060
NE07	Citrus Pump station	Structure/Wall	Future	10.0	15.5	0.020	0.069
NE01	Citrus Lakefront Levee	Levee	Existing	8.6	13.0	0.007	0.044
NE01	Citrus Lakefront Levee	Levee	Future	10.1	15.5	0.010	0.057
NE08	Jahncke Pump station	Structure/Wall	Future	10.0	15.5	0.020	0.069
NE05	Lincoln Beach	Structure/Wall	Future	6.1	15.5	0.000	0.003
NE06	Collins Pipeline Crossing	Structure/Wall	Future	9.4	17.5	0.003	0.012
NE30	Transition Reach NE01 to NE02	Levee	Existing	9.6	14.5	0.010	0.064
NE30	Transition Reach NE01 to NE02	Levee	Future	11.1	16.5	0.007	0.066
NE02	New Orleans East Lakefront Levee	Levee	Existing	9.9	15.5	0.003	0.033
NE02	New Orleans East Lakefront Levee	Levee	Future	11.4	17.5	0.006	0.062
NE31	South Point transition reach	Levee	Existing	9.0	16.5	0.002	0.025
NE31	South Point transition reach	Levee	Future	10.5	18.5	0.005	0.052

Table 24 – Orleans Parish Lakefront East Segments - 1% Design Information

The 1% hydraulic design for the existing Citrus Lakefront levee (segment NE01) is shown in Figure 30. The 1% design height for existing conditions must be +13ft and +15.5ft for future conditions. The breakwater at +9ft and the railroad (40ft wide, elevation +6ft) are important elements that reduce the wave heights in front of the actual levee. Therefore, the levee height can be relatively low in order to meet the design criteria. The railroad and the foreshore protection are part of the flood defense and these must be maintained at these elevations.

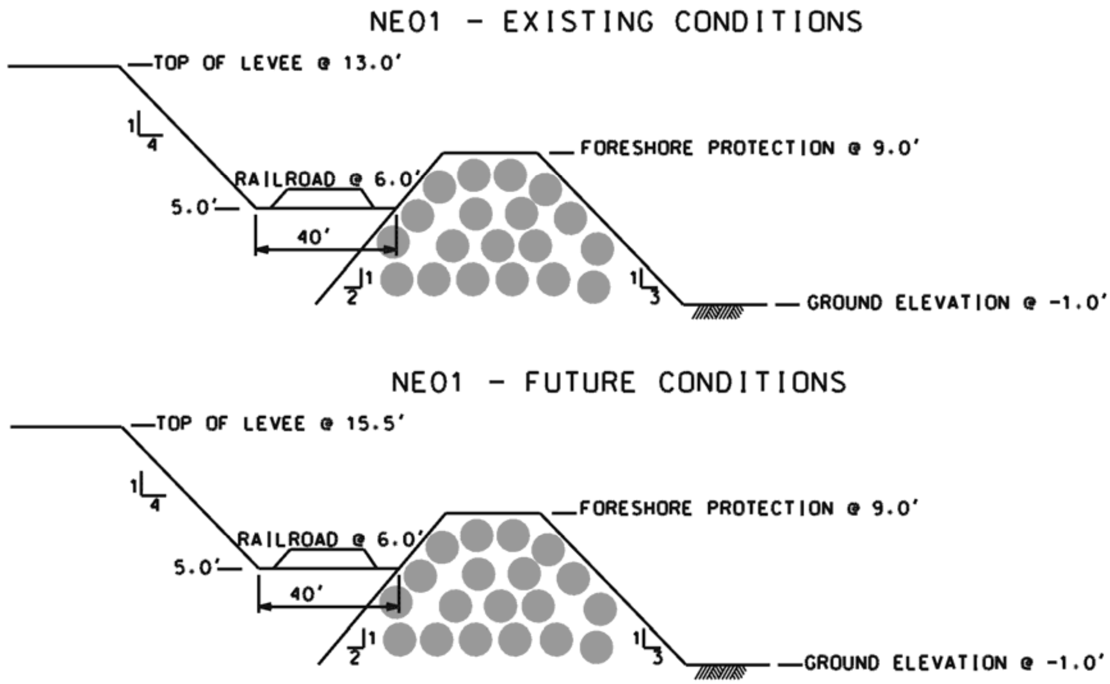


Figure 35 – Cross-section profile for Citrus Lakefront levee (NE01) for existing (upper panel) and future conditions (lower panel).

The hydraulic design section for the New Orleans East Lakefront Levee (NE02) is shown in Figure 31. The 1% design height for existing conditions must be +15.5ft and +17.5ft for future conditions. Notice that these heights are higher than the Citrus Lakefront Levee. This is partly because the surge levels are a bit higher towards the east. Furthermore, the foreshore protection is much lower here (+6ft instead of +9ft) and results in less wave reduction. Nevertheless, the foreshore protection at 6ft and the railroad (40ft wide, +6ft) are important elements that must be maintained at these elevations.

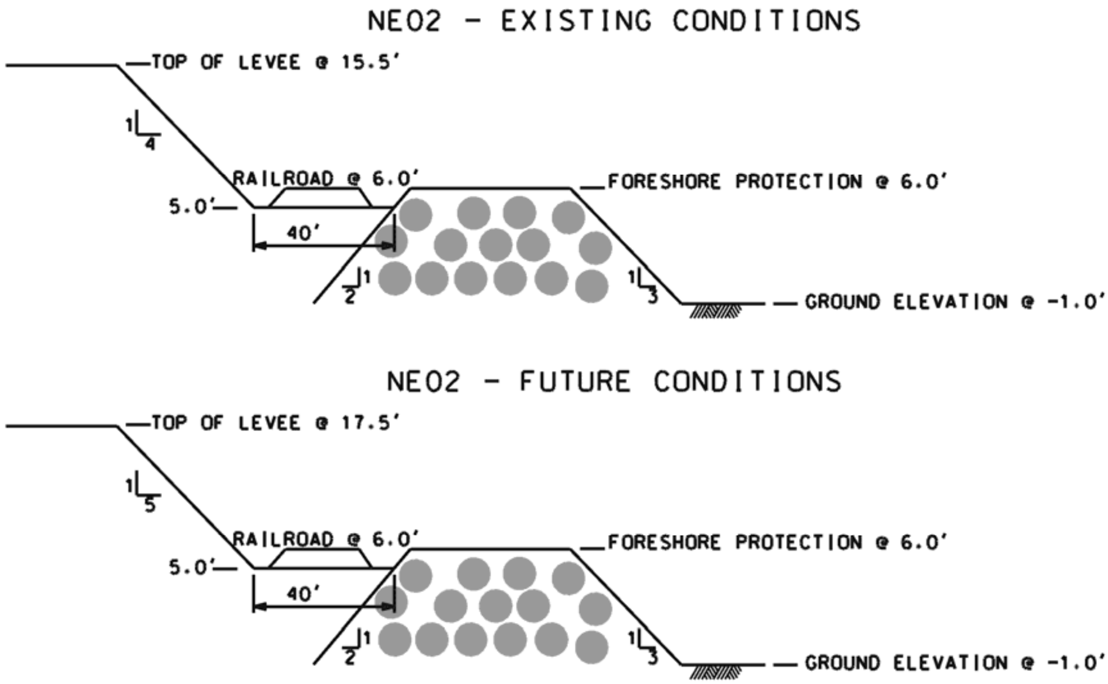


Figure 36 – Cross-section profile for New Orleans East Lakefront Levee (NE02) for existing (upper panel) and future conditions (lower panel).

Figure 37 shows the hydraulic design of the transition levee (NE30) between the Citrus Lakefront Levee and the New Orleans East Lakefront Levee. Figure 38 presents the transition between the New Orleans East Lakefront levee and the South Point to US Highway 90 levee (NE31).

The various floodwalls and gates in the Orleans Parish Lakefront East area have design elevations ranging from +15.5 to +17.5ft for future conditions.

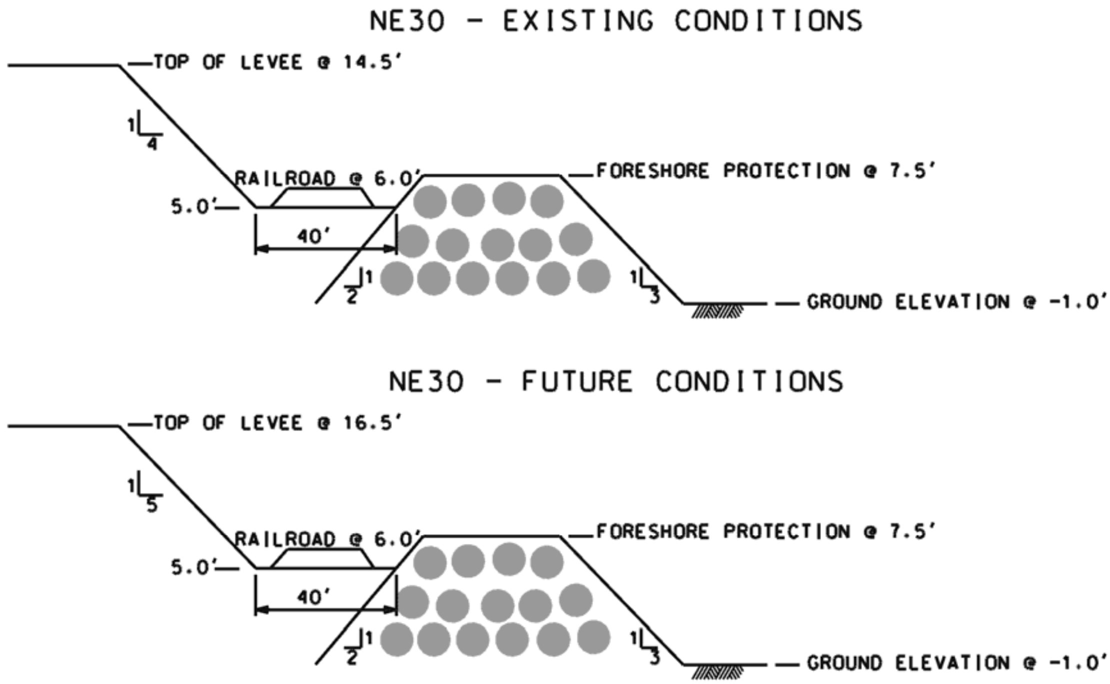


Figure 37 – Cross-section profile for transition levee between Citrus Lakefront Levee and New Orleans East Lakefront Levee (NE30) for existing (upper panel) and future conditions (lower panel).

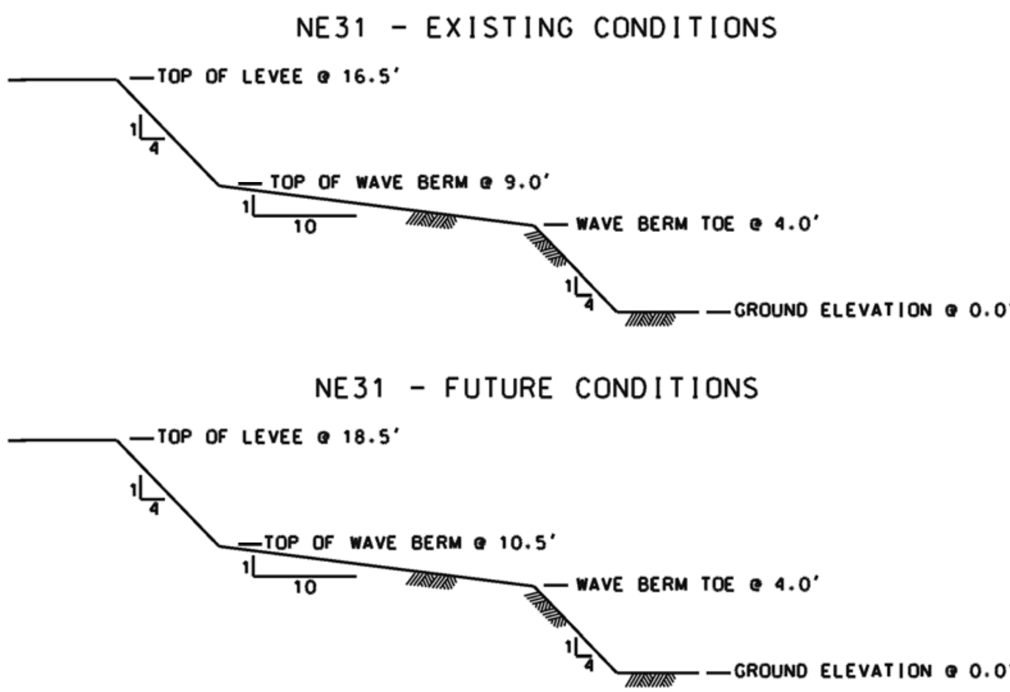


Figure 38 – Cross-section profile for transition levee between New Orleans East Lakefront levee and the South Point to US Highway 90 (NE31) for existing (upper panel) and future conditions (lower panel).

3.5.4 Wave Forces

Wave forces were computed for all structures within the Orleans Parish Lakefront East segment with the Goda method, using future conditions. The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics. The following tables summarize the resulting wave forces. Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

New Orleans East Lakefront Sections Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
NE04	NO Lakefront Airport West	0.7	1.9	10.7	0.7	1.9	10.7
NE03	NO Lakefront Airport East	0.7	1.7	10.6	0.7	1.7	10.6
NE09	St Charles Pump station	0.7	1.7	10.6	0.7	1.7	10.6
NE07	Citrus Pump station	0.7	1.8	10.7	0.7	1.8	10.7
NE08	Jahncke Pump station	0.7	1.8	10.7	0.7	1.8	10.7
NE05	Lincoln Beach	0.7	2.0	10.7	0.7	2.0	10.7
NE06	Collins Pipeline Crossing	2.4	10.4	10.3	2.4	10.4	10.3

Table 25 – Waves Forces for Orleans Parish Lakefront East Segments (50% values).

New Orleans East Lakefront Sections Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
NE04	NO Lakefront Airport West	0.9	2.4	10.8	0.9	2.4	10.8
NE03	NO Lakefront Airport East	0.8	2.2	10.8	0.8	2.2	10.8
NE09	St Charles Pump station	0.8	2.2	10.8	0.8	2.2	10.8
NE07	Citrus Pump station	0.9	2.4	10.8	0.9	2.4	10.8
NE08	Jahncke Pump station	0.9	2.4	10.8	0.9	2.4	10.8
NE05	Lincoln Beach	0.9	2.6	10.8	0.9	2.6	10.8
NE06	Collins Pipeline Crossing	3.2	15.6	10.8	3.2	15.6	10.8

Table 26 – Waves Forces for Orleans Parish Lakefront East Segments (90% values).

3.5.5 Resiliency

The designs for the levees and structures within Orleans Parish – Lakefront East were examined for resiliency by also computing the overtopping rate for the 0.2 percent event for each design. The water level and overtopping rate was determined for the 50% assurance during the 0.2% event. The results are presented in Table 27. For all sections, the 0.2% surge elevation remains below the top of the flood defense, and the overtopping rate is less than 1 cfs/ft per ft (best estimates).

Orleans Parish East Lakefront Sections Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
NE04	NO Lakefront Airport West	Structure/Wall	Future	15.5	12.6	0.291
NE03	NO Lakefront Airport East	Structure/Wall	Future	15.5	12.3	0.203
NE09	St Charles Pump station	Structure/Wall	Future	15.5	12.3	0.422
NE07	Citrus Pump station	Structure/Wall	Future	15.5	12.4	0.458
NE01	Citrus Lakefront Levee	Levee	Existing	13.0	11.0	0.216
NE01	Citrus Lakefront Levee	Levee	Future	15.5	12.5	0.158
NE08	Jahncke Pump station	Structure/Wall	Future	15.5	12.4	0.456
NE05	Lincoln Beach	Structure/Wall	Future	15.5	12.5	0.101
NE06	Collins Pipeline Crossing	Structure/Wall	Future	17.5	13.0	0.136
NE30	Transition Reach NE01 to NE02	Levee	Existing	14.5	11.1	0.148
NE30	Transition Reach NE01 to NE02	Levee	Future	16.5	12.6	0.100
NE02	New Orleans East Lakefront Levee	Levee	Existing	15.5	11.5	0.065
NE02	New Orleans East Lakefront Levee	Levee	Future	17.5	13.0	0.093
NE31	South Point transition reach	Levee	Existing	16.5	11.7	0.050
NE31	South Point transition reach	Levee	Future	18.5	13.2	0.081

Table 27 – Resiliency for Orleans Parish Lakefront East Segments

3.6 GIWW – Outside the Gates at MRGO/GIWW

3.6.1 General

As of September 2007, the location of the MRGO/GIWW closure gates and the connecting levee is conceptual and will be finalized during the design-build process. The hurricane protection system alignment considered in this section is based on one of several alignments that may be considered. For this alignment, levees and floodwalls along all of the Inner Harbor Navigation Canal (IHNC), and that portion of the Gulf Intracoastal Waterway (GIWW)/Mississippi River Gulf Outlet (MRGO) from the IHNC to the southern side of the Bayou Bienvenue Floodgate and the south-eastern edge of the Michoud Canal will be isolated from hurricane surges emanating from Lake Borgne by a closure complex. The closure will consist of 2 navigable floodgates, one in the MRGO and the other in the GIWW, connected by an earthen levee.

This paragraph discusses the levee/floodwall sections along GIWW outside the new gates. Figure 39 shows the levee segments, floodwalls and pumping stations analyzed for the GIWW in this section. The South Point to GIWW levee is included in this section of the report because the surge levels along this levee are affected by the gates on the MRGO/GIWW. Notice that the transition section (NE31) is already discussed in section 3.5 and will not be discussed in this section.

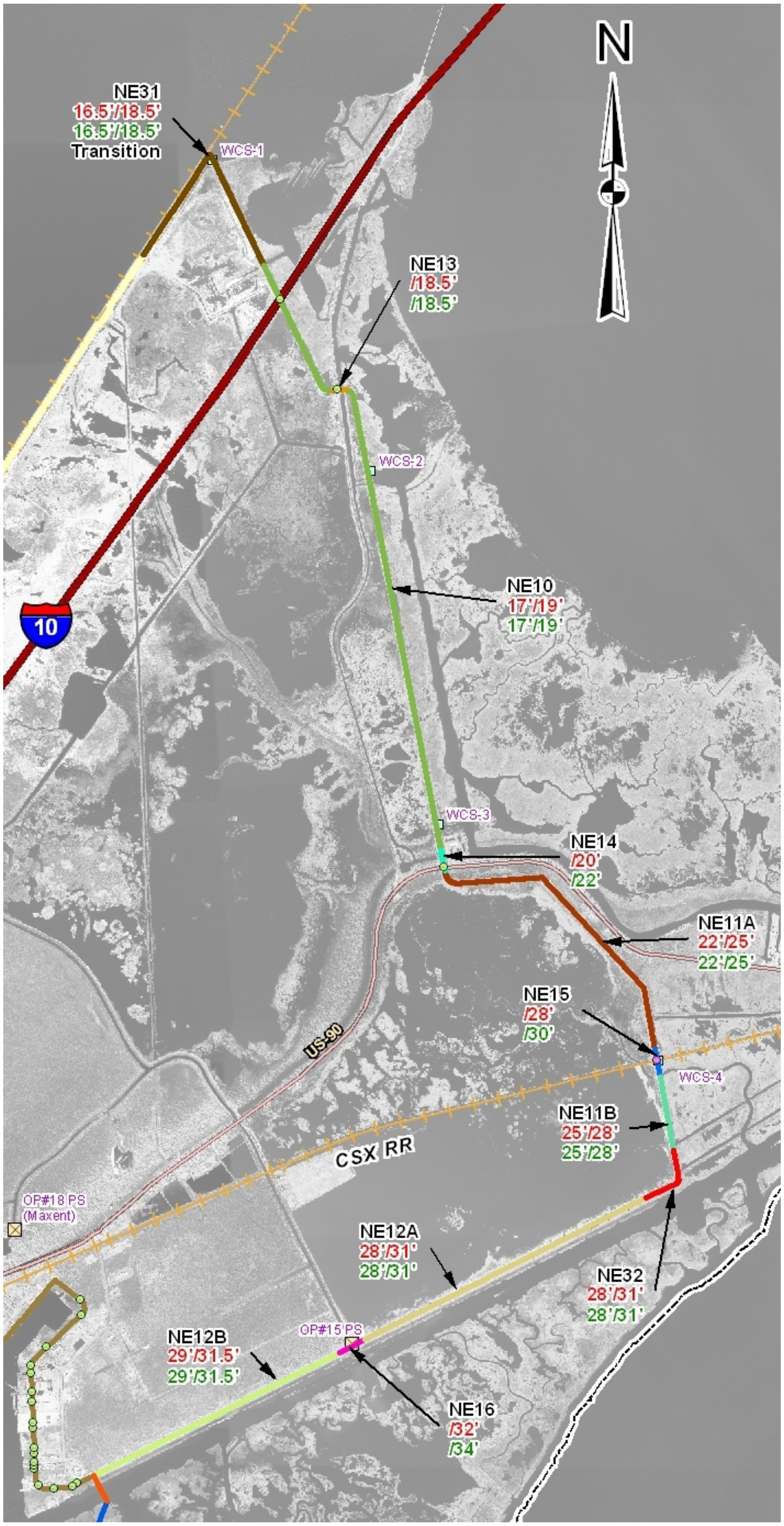


Figure 39 – Levee and floodwall sections in GIWW area. The numbers represent existing/future conditions and are without (red) and with (green) structural superiority.

3.6.2 Hydraulic Boundary Conditions

The hydraulic design characteristics for the sections are listed in Table 28. The existing hydraulic conditions are based on the JPM-OS method using the results from ADCIRC and STWAVE. The future conditions are derived by adding 1.5 ft to the surge elevation, and adding 0.75 ft to the wave height. The wave period is computed using the assumption that the wave steepness remains constant. For more information, see Chapter 2.

Notice that the hydraulic boundary conditions have been based on numerical computations with the gates at MRGO and GIWW in place (2010 grid). The effect on the 1% surge levels is about +0.5ft along the South Point to GIWW levee. Near the gates, this effect increases to about +1ft. The effect on the wave characteristics is limited. Because of the higher surge levels, the wave height and period also increase in the surrounding of the gates.

GIWW Sections (outside MRGO gate) 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
NE13	Highway 11 Floodgate	Structure/Wall	Future	11.1	0.9	4.4	0.4	5.8	1.1
NE10	South Point to Highway 90 Levee	Levee	Existing	10.9	0.9	4.4	0.4	5.4	1.1
NE10	South Point to Highway 90 Levee	Levee	Future	12.4	0.9	5.0	0.4	5.8	1.1
NE14	Highway 90 Floodgate	Structure/Wall	Future	12.5	0.9	5.0	0.4	5.6	1.1
NE11A	Highway 90 to CSX RR Levee	Levee	Existing	14.3	0.9	4.0	0.4	8.3	1.7
NE11A	Highway 90 to CSX RR Levee	Levee	Future	15.8	0.9	4.8	0.4	9.0	1.7
NE15	CSX RR Floodgate	Structure/Wall	Future	17.3	1.0	6.7	0.6	7.1	1.3
NE11B	CSX RR to GIWW Levee	Levee	Existing	16.2	1.0	5.9	0.6	7.7	1.5
NE11B	CSX RR to GIWW Levee	Levee	Future	17.7	1.0	6.7	0.6	8.2	1.5
NE32	Transition Levee	Levee	Existing	16.2	1.0	5.4	0.5	7.9	1.6
NE32	Transition Levee	Levee	Future	17.7	1.0	6.2	0.5	8.5	1.6
NE12A	NO East Back Levee from PS15 East along GIWW	Levee	Existing	17.4	1.0	5.4	0.5	8.0	1.6
NE12A	NO East Back Levee from PS15 East along GIWW	Levee	Future	18.9	1.0	6.2	0.5	8.6	1.6
NE16	NO East Pump Station 15	Structure/Wall	Future	18.9	1.0	5.5	0.5	7.8	1.5
NE12B	NO East Back Levee from Gate to PS15	Levee	Existing	18.4	1.0	7.1	0.7	7.9	1.6
NE12B	NO East Back Levee from Gate to PS15	Levee	Future	19.9	1.0	7.9	0.7	8.3	1.6

Table 28 – GIWW Segments outside MRGO/GIWW Gates - 1% Hydraulic Boundary Conditions

The various hydraulic sections are briefly discussed below:

South Point to GIWW levee (NE10, NE11A and NE11B): This levee is part of the South Point to GIWW levee that runs in a north-south direction from South Point southward to the GIWW. It is divided hydraulically into three sections: South Point to US Highway 90 (NE10), US Highway 90 to the CSX Railroad (NE11A), and the CSX Railroad to the GIWW (NE11B). This levee segment is approximately 8.4 miles including the structures mentioned above. The pre-Katrina authorized levee crest elevation varies from +15 to +18 ft. The ground elevation in front of the levee is assumed to be 0.0 ft NAVD88 2004.65. Notice that the 1% wave heights are not depth-limited for these levee sections.

Transition Levee (NE32): A transition levee has been included in between the CSX Railroad to GIWW levee (NE11B) and the New Orleans East Back Levee (NE12). The ground elevation in front of the levee is assumed to be 0.0 ft NAVD88 2004.65. Notice that the 1% wave heights are not depth-limited for these levee sections.

New Orleans East Back Levee (NE12A and NE12B): The New Orleans East Back Levee runs in an approximately east-west direction along the GIWW (Gulf Intracoastal Waterway) to the closure complex gate. The New Orleans Sewerage and Water Board Pump Station 15 is located along this segment of levee and divides reaches 12A and 12B. Reach 12B is located between the closure gate and Pumping Station 15 and reach 12A continues east of the pumping station. This levee segment is approximately 5 miles. The existing levee was damaged during Hurricane Katrina. The pre-Katrina authorized design elevation is +18 ft. The ground elevation in front of the levee is assumed to be 0.0 ft NAVD88 2004.65. Notice that the 1% wave heights are not depth-limited for these levee sections.

Along this flood protection section, there are several floodgates and pump stations. For all, the ground elevation in front of the floodgate/structure is assumed to be 0.0 ft NAVD88 2004.65 one wave length from the structure (≈ 300ft).

3.6.3 Project Design Heights

The design characteristics for the sections along GIWW and between South Point and GIWW are listed in Table 29. The levees are designed for both existing and future conditions. Note that the floodgates and pump stations are only evaluated for future conditions, because these are hard structures. The structures that include structural superiority of 2ft are the Highway 90 Floodgate (NE14), the CSX Railroad Floodgate (NE15) and the NO East Pump Station 15 (NE16).

GIWW Sections (outside MRGO gate) 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
NE13	Highway 11 Floodgate	Structure/Wall	Future	11.1	18.5	0.007	0.034
NE10	South Point to Highway 90 Levee	Levee	Existing	10.9	17.0	0.006	0.063
NE10	South Point to Highway 90 Levee	Levee	Future	12.4	19.0	0.008	0.077
NE14	Highway 90 Floodgate	Structure/Wall	Future	12.5	22.0	0.004	0.016
NE11A	Highway 90 to CSX RR Levee	Levee	Existing	14.3	22.0	0.006	0.064
NE11A	Highway 90 to CSX RR Levee	Levee	Future	15.8	25.0	0.008	0.071
NE15	CSX RR Floodgate	Structure/Wall	Future	17.3	30.0	0.007	0.025
NE11B	CSX RR to GIWW Levee	Levee	Existing	16.2	25.0	0.005	0.067
NE11B	CSX RR to GIWW Levee	Levee	Future	17.7	28.0	0.005	0.056
NE32	Transition Levee	Levee	Existing	16.2	28.0	0.001	0.020
NE32	Transition Levee	Levee	Future	17.7	31.0	0.004	0.046
NE12A	NO East Back Levee from PS15 East along GIWW	Levee	Existing	17.4	28.0	0.003	0.046
NE12A	NO East Back Levee from PS15 East along GIWW	Levee	Future	18.9	31.0	0.009	0.087
NE16	NO East Pump Station 15	Structure/Wall	Future	18.9	34.0	0.000	0.002
NE12B	NO East Back Levee from Gate to PS15	Levee	Existing	18.4	29.0	0.007	0.080
NE12B	NO East Back Levee from Gate to PS15	Levee	Future	19.9	31.5	0.009	0.085

Table 29 – GIWW Segments outside MRGO/GIWW Gates – 1% Design Information

The hydraulic design sections for the South Point to GIWW levee are shown in Figure 40, Figure 41 and Figure 42. The 1% design heights for existing conditions are +17ft, +22ft and +25ft. The increase logically follows the increase in surge levels towards Lake Borgne from +11ft near the Lake Pontchartrain to +17ft near the GIWW.

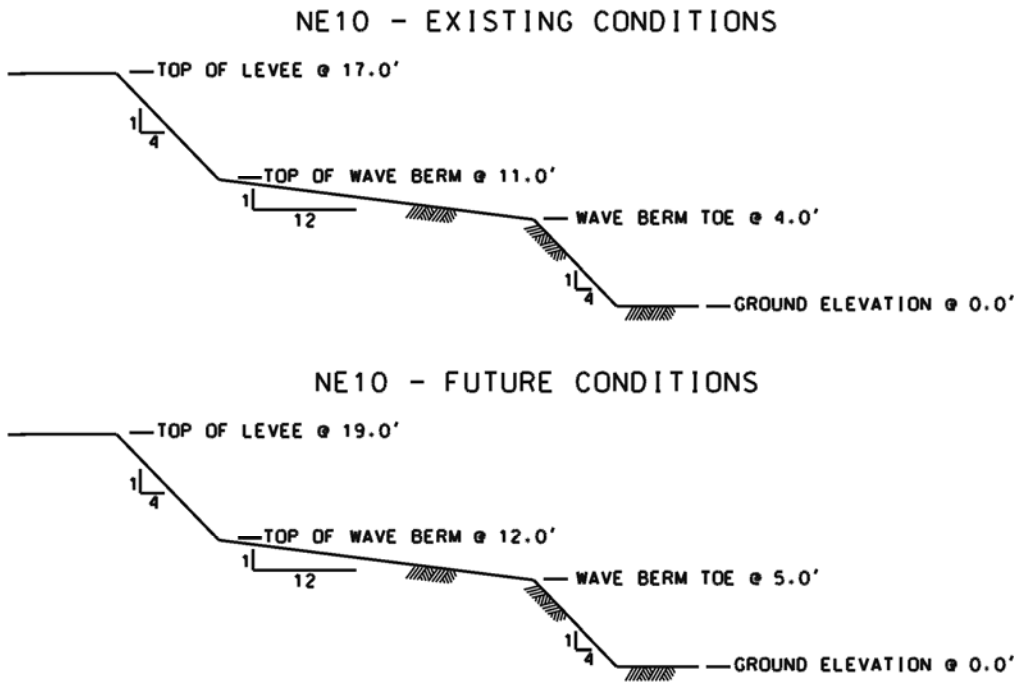


Figure 40 – Typical Levee Design Cross-Section – South Point to US Highway 90 (NE10)

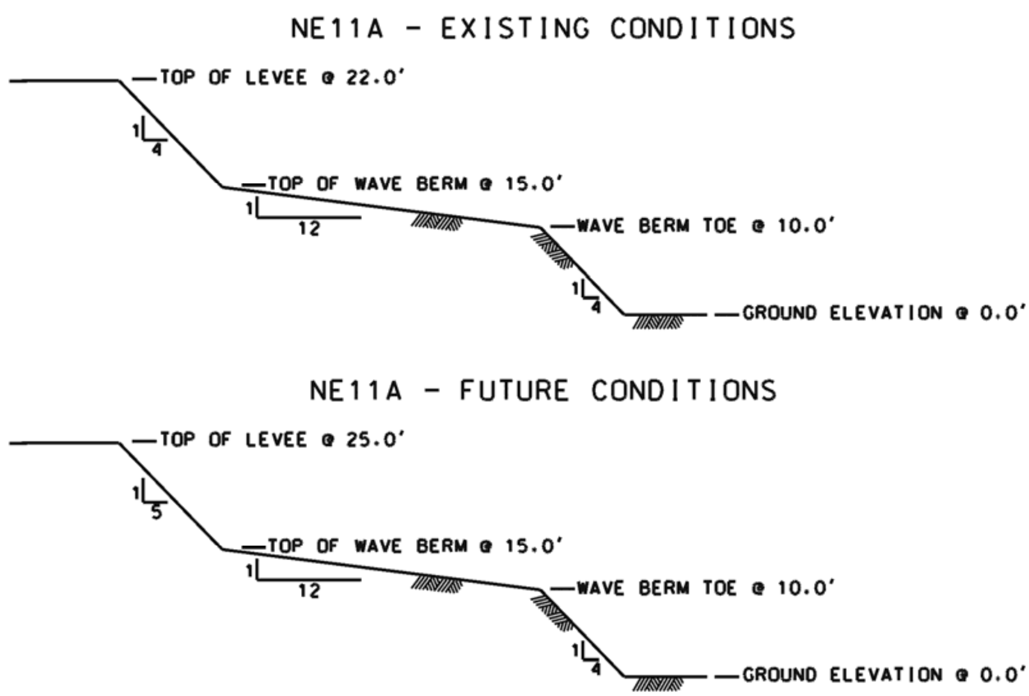


Figure 41 – Typical Levee Design Cross-Section – US Highway 90 to CSX Railroad (NE11A)

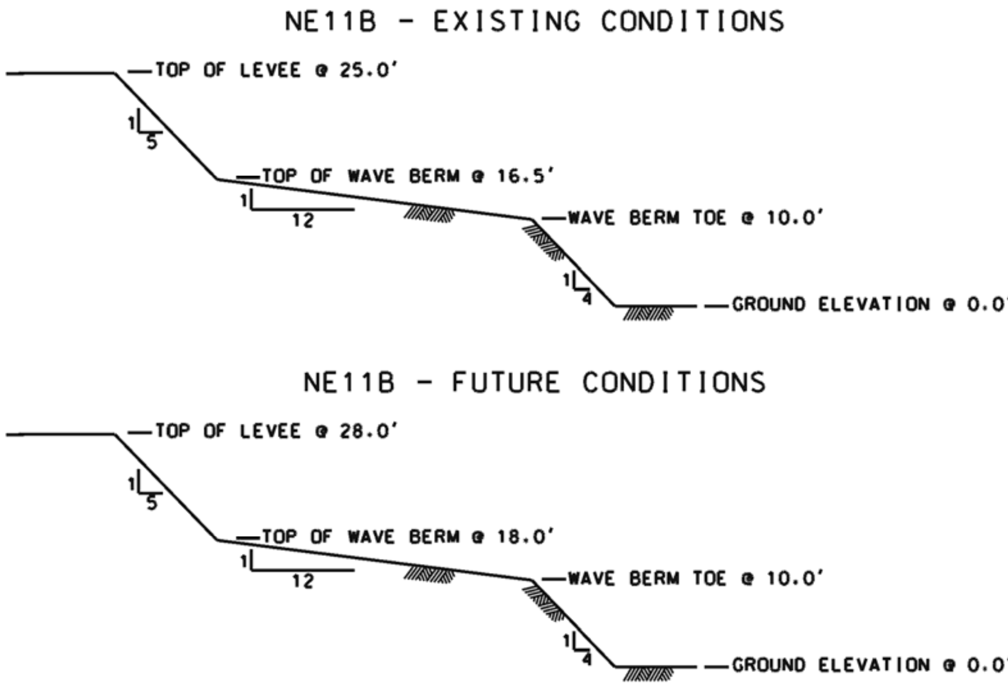


Figure 42 – Typical Levee Design Cross-Section – CSX Railroad to GIWW (NE11B)

The hydraulic design sections for the levee sections along the GIWW outside the gate structure are shown in Figure 43 and Figure 44. The 1% design heights for existing conditions are +28 and +29 ft. These levee sections have high 1% surge levels (+17 to +18 ft) and the wave attack near the toe of the structure is severe (H_s = 5-6ft and T_p = 7-8s) for existing conditions. The design height are increased to +31ft (NE12A) and +31.5ft (NE12B) and the wave berm has to be raised 1.5ft in order to meet the design criteria for future conditions. The transition levee (NE32) between the CSX Railroad to GIWW levee (NE11B) and the New Orleans Back Levee (NE12A) has the same cross-section as section NE12A

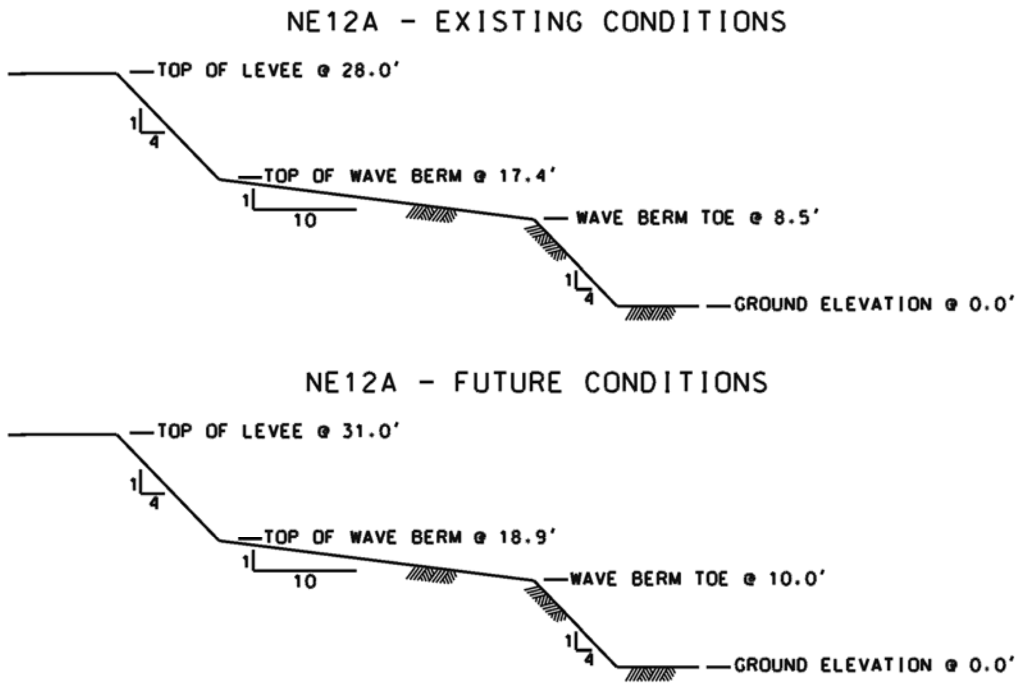


Figure 43 – Typical Levee Design Cross-Section – New Orleans East Back Levee (NE12A) for existing (upper panel) and future conditions (lower panel).

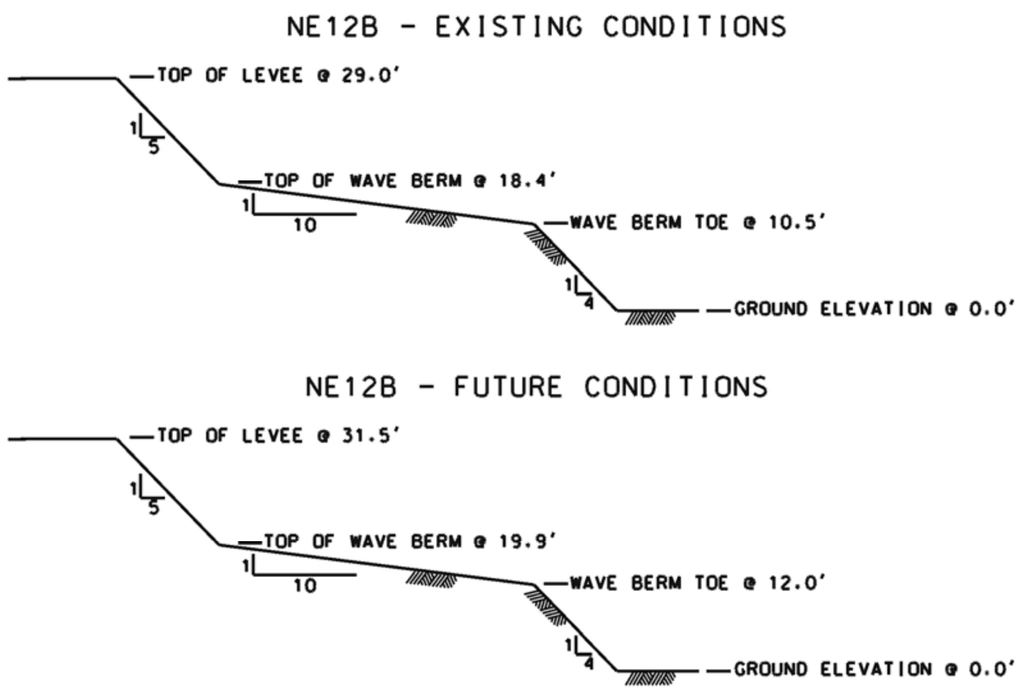


Figure 44 – Typical Levee Design Cross-Section – New Orleans East Back Levee (NE12B) for existing (upper panel) and future conditions (lower panel).

3.6.4 Wave Forces

Wave forces were computed for all structures within the South Point to GIWW segment and the GIWW segment outside the gate with the Goda method, using future conditions. The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics. The following tables summarize the resulting wave forces. Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

GIWW outside the MRGO/GIWW gate Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
NE13	HWY 11 Floodgate	1.5	5.5	11.8	1.5	5.5	11.8
NE14	HWY 90 Floodgate	2.3	11.4	13.4	2.3	11.4	13.4
NE15	CSX RR Floodgate	12.9	134.8	16.6	12.9	134.8	16.6
NE16	NO East PS 15	14.4	244.9	12.2	14.4	244.9	12.2

Table 30 – Waves forces for GIWW Segments outside MROG/GIWW Gates (50% values)

GIWW outside the MRGO/GIWW gate Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
NE13	HWY 11 Floodgate	1.8	7.1	11.9	1.8	7.1	11.9
NE14	HWY 90 Floodgate	2.9	14.5	13.5	2.9	14.5	13.5
NE15	CSX RR Floodgate	16.6	180.6	17.1	16.6	180.6	17.1
NE16	NO East PS 15	19.5	342.9	12.7	19.5	342.9	12.7

Table 31 – Waves Forces for GIWW Segments outside MROG/GIWW Gates (90% values).

3.6.5 Resiliency

The designs for the levees and structures along South Point to GIWW and along GIWW outside the gates were examined for resiliency by also computing the overtopping rate for the 0.2 percent event for each design. The water level and overtopping rate was determined for the 50% assurance during the 0.2% event. The results are presented in Table 32. For all sections, the 0.2% surge elevation remains below the top of the flood defense, and the overtopping rate is less than 1 cfs/ft per ft (best estimates).

GIWW Sections (outside MRGO gate) Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
NE13	Highway 11 Floodgate	Structure/Wall	Future	18.5	14.4	0.432
NE10	South Point to Highway 90 Levee	Levee	Existing	17.0	14.2	0.933
NE10	South Point to Highway 90 Levee	Levee	Future	19.0	15.7	0.860
NE14	Highway 90 Floodgate	Structure/Wall	Future	22.0	15.7	0.160
NE11A	Highway 90 to CSX RR Levee	Levee	Existing	22.0	17.5	0.831
NE11A	Highway 90 to CSX RR Levee	Levee	Future	25.0	19.0	0.586
NE15	CSX RR Floodgate	Structure/Wall	Future	30.0	20.7	0.145
NE11B	CSX RR to GIWW Levee	Levee	Existing	25.0	19.7	0.451
NE11B	CSX RR to GIWW Levee	Levee	Future	28.0	21.2	0.282
NE32	Transition Levee	Levee	Existing	28.0	19.7	0.100
NE32	Transition Levee	Levee	Future	31.0	21.2	0.153
NE12A	NO East Back Levee from PS15 East along GIWW	Levee	Existing	28.0	20.9	0.228
NE12A	NO East Back Levee from PS15 East along GIWW	Levee	Future	31.0	22.4	0.321
NE16	NO East Pump Station 15	Structure/Wall	Future	34.0	22.4	0.024
NE12B	NO East Back Levee from Gate to PS15	Levee	Existing	29.0	22.1	0.380
NE12B	NO East Back Levee from Gate to PS15	Levee	Future	31.5	23.6	0.322

Table 32 – Resiliency for GIWW Segments outside MRGO/GIWW Gates

3.7 IHNC and GIWW (with MRGO/GIWW closure only)

3.7.1 General

The preliminary plan for a closure of the MRGO and GIWW will consist of 2 gates connected by a levee between the gates. For this report, the complex location is approximately 2 miles east of the Paris Road (LA Hwy. 47) Bridge. The gated structure in the GIWW is located near the eastern levee of the Michoud Canal and ties into the existing NO East Back levee alignment. The gated structure in the MRGO is located just south of the Bayou Bienvenue floodgate and ties into the existing alignment of the MRGO hurricane protection levee that parallels the MRGO. The gated structures are connected by a levee (or floodwall) across the Lake Borgne marsh to form a continuous line of protection. As noted earlier, the location of the closures and levee are conceptual and will be finalized during the design-build process.

When the GIWW/MRGO closure is constructed and the Seabrook gate is in place, the 1% storm surge flooding from Lake Borgne and Lake Pontchartrain is eliminated. An alternative is presented that includes the closure structure at the GIWW/MRGO but does not include the Seabrook Gate. This section presents the hydraulic boundary conditions and the design heights for the flood protection along GIWW (inside the gates) and IHNC for the situation with only the GIWW/MRGO closure. The next section presents the 1% design characteristics with both closures.

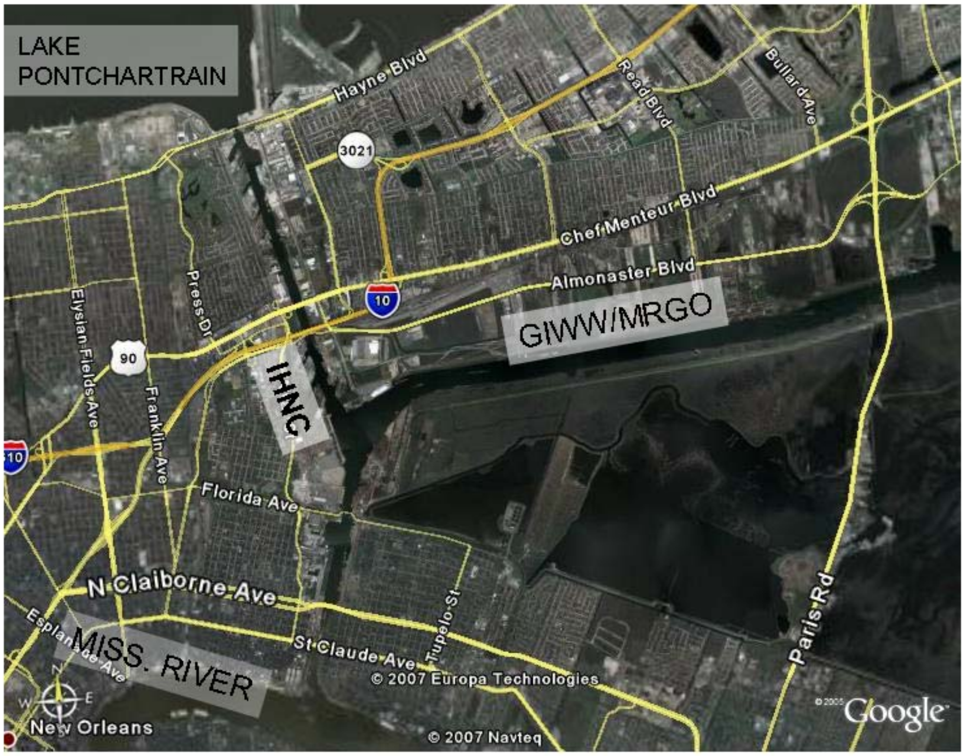


Figure 45 – Map of IHNC and GIWW/MRGO (From Google Earth)

The Inner Harbor Navigation Canal (IHNC) is a navigable waterway, oriented in a north-south direction, which connects the MRGO/GIWW from the east with Lake Pontchartrain to the North and the Mississippi River to the South (See Figure 45 for detail). The IHNC Lock connects the southernmost end of the IHNC with the Mississippi River. The portion of the IHNC south of its junction with the MRGO/GIWW can accommodate deep draft navigation, while the northern reach above this juncture is only navigable by shallow draft vessels. Floodwalls also are part of the protection system along the IHNC. There are several long segments of floodwall both north and south of I-10.

There are three pump stations located in the IHNC: Pump Station No. 19, Pump Station No. 5, and the Dwyer Pump Station. The Dwyer Pump Station (IH05) is located near Lake Pontchartrain (northern section of the IHNC). Pump Stations 19 and 5 (IH10) are located across the canal from each other at the southern end of the IHNC. Moreover, there are two floodgates at the railroad track that passes the IHNC near Lake Pontchartrain ("Norfolk Southern Railroad Floodgates East and West"). The floodgates are located south of the Hickey Bridge on the Westside (NO20) and Eastside (NE20) of the IHNC.

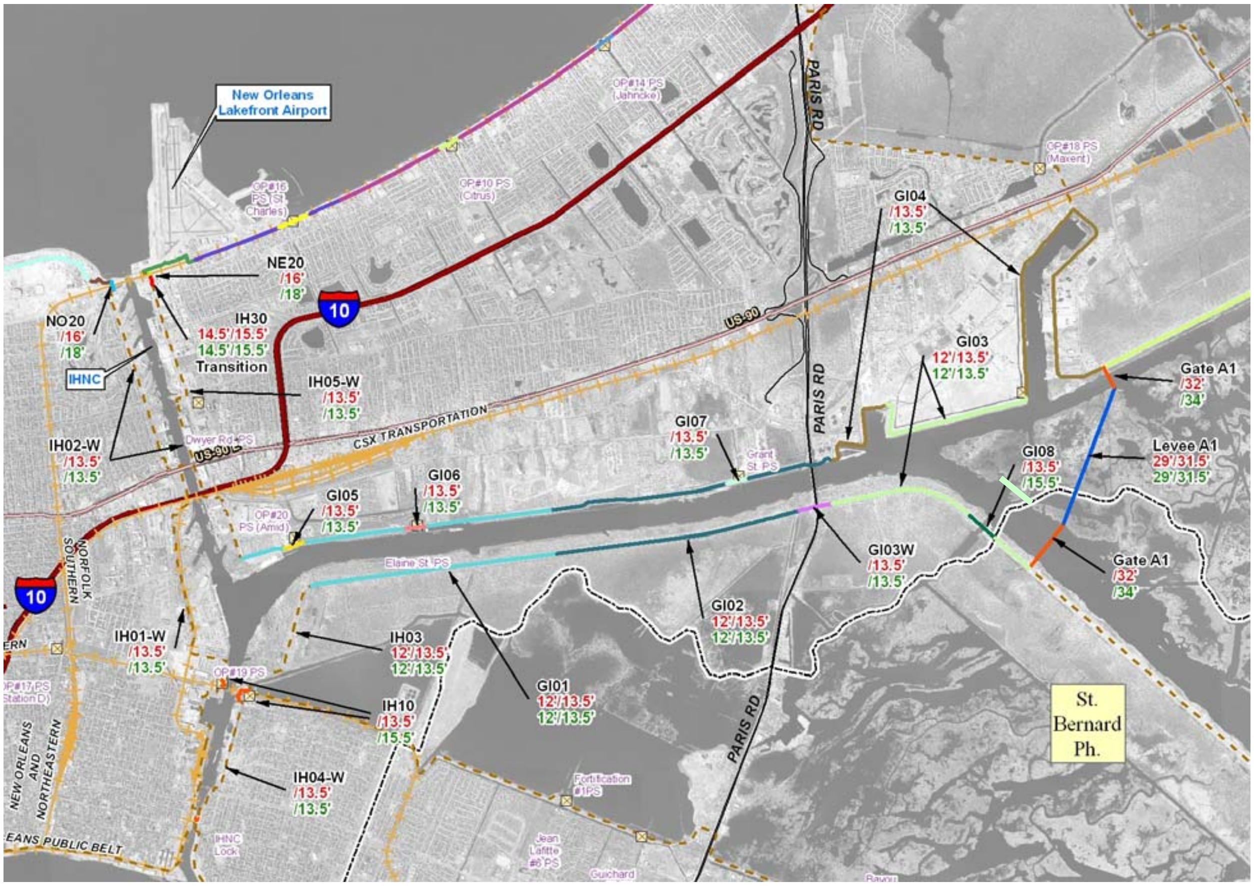


Figure 46 – Levee and floodwall sections in IHNC/GIWW area (with MRGO/GIWW gate only).

The GIWW is a navigable waterway, oriented in an east to west direction that connects the MRGO and the IHNC. This reach of the GIWW was deepened when it was incorporated into the MRGO waterway to provide access from the Gulf of Mexico to the Mississippi River via the IHNC Lock. This portion of the GIWW was analyzed in three segments, GI01, GI02, and GI03.

Segments GI01 and GI02 divide the five mile reach from the IHNC to the Paris Road Bridge into two equal segments. Analyses in segments GI01 and GI02 apply to levees and floodwalls on both sides of the Canal system; available rights-of-way determine whether the protection is levee or floodwall. Analysis of segment GI03 applies to the northern segment of the levee from the Paris Road Bridge to the south-eastern tip of the Michoud Canal line of protection, and to the levee on the south side of the Canal from the Paris Road Bridge to the southside of the Bayou Bienvenue floodgate. The eastern limit of GI03 coincides with the location of the new MRGO/GIWW closure complex. This segment of levee was constructed or improved as part of the Lake Pontchartrain, LA and Vicinity Hurricane Protection Project.

Along this GIWW alignment are three New Orleans Sewage and Water Board drainage pumping stations: the AMID (GI05) and Elaine St. (GI06) Pumping stations in segment GI01 and the Grant St. Pumping Station (GI07) in segment GI02. The Bayou Bienvenue floodgate GI08) is located at the southern end of reach GI03.

3.7.2 Hydraulic Boundary Conditions

The hydraulic design characteristics for the sections are listed in Table 33. The existing surge levels are based on the JPM-OS method using the results from ADCIRC. The future conditions are derived by adding 1.5 ft to the surge elevations. This has been done because the surge elevation is not fully controlled because of the opening at Lake Pontchartrain. Hence, the surge elevations in the IHNC and GIWW behind the closure gate will follow the sea level rise in this case. The 1% wave characteristics are not available from STWAVE. The waves in these small canals are not resolved with STWAVE because these canals are too small for the STWAVE grid resolution.

The wave characteristics at IHNC and GIWW have been estimated using the empirical method from Brettschneider (see e.g. Shore Protection Manual, 1984). This method gives estimates for the fully-developed wave height and the wave period for a given fetch, wind speed and water depth. The fetch and the wind speed are the dominant parameters in this case, because the water depth is quite large in the GIWW and IHNC. Because of the difference in dimensions (width, length), the fetch at the IHNC and GIWW differs significantly. Therefore, a distinction has been made between the wave characteristics at the sections along the GIWW and IHNC.

Along the GIWW, the fetch has been estimated at 0.5 mile which is approximately the width of the GIWW. Wave generation perpendicular to the floodwalls and levees has been assumed to be the most severe condition for overtopping. The applied 1% wind speed is 77 mph (see Appendix C). Under these conditions the resulting significant wave height is 3ft and the peak period is 3.5s according to Brettschneider's formulations. These wave characteristics have been applied uniformly for all levee and floodwall sections along GIWW and for section IH01-W and IH03 along the IHNC. These sections along the IHNC are exposed to waves that are generated at the intersection of the GIWW and the IHNC.

Along the IHNC the width of the canal is much smaller north from the I-10 and south from Pump Station 5. Hence, a fetch of 0.25 mile has been applied in combination with a wind speed of 77 mph during design conditions. The resulting significant wave height is 2.3ft and the peak period is 3.1s. These characteristics have been applied uniformly for all levee and floodwall sections along IHNC (except for IH01-W and IH03 as discussed above).

The wave characteristics for future conditions are taken similar to the ones for existing conditions. The waves are determined by the fetch and the wind speed (and not by the water depth) in these small canals. Thus, only the 1% surge level has been changed to evaluate future conditions.

IHNC and GIWW sections (with MRGO/GIWW gate only) 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
GI08	Bienvenue Floodgate	Structure/Wall	Future	9.6	0.5	3.0	0.3	3.5	0.7
GI03	Michoud Canal to Michoud Slip and Paris Rd Bridge to Bienvenue Floodgate	Levee	Existing	8.1	0.5	3.0	0.3	3.5	0.7
GI03	Michoud Canal to Michoud Slip and Paris Rd Bridge to Bienvenue Floodgate	Levee	Future	9.6	0.5	3.0	0.3	3.5	0.7
GI04	Michoud Canal and Slip	Structure/Wall	Future	9.6	0.5	3.0	0.3	3.5	0.7
GI07	Grant Pump Station	Structure/Wall	Future	9.5	0.5	3.0	0.3	3.5	0.7
GI03-W	Floodwall under Paris Rd Bridge	Structure/Wall	Future	9.5	0.5	3.0	0.3	3.5	0.7
GI02	Paris Road to levee section GI01	Levee	Existing	8.0	0.5	3.0	0.3	3.5	0.7
GI02	Paris Road to levee section GI01	Levee	Future	9.5	0.5	3.0	0.3	3.5	0.7
GI01	Levee Section GI02 to IHNC	Levee	Existing	7.9	0.5	3.0	0.3	3.5	0.7
GI01	Levee Section GI02 to IHNC	Levee	Future	9.4	0.5	3.0	0.3	3.5	0.7
GI06	Elaine Pump Station	Structure/Wall	Future	9.4	0.5	3.0	0.3	3.5	0.7
GI05	Amid Pump Station (PS#20)	Structure/Wall	Future	9.4	0.5	3.0	0.3	3.5	0.7
IH30	Transition Reach	Levee	Existing	8.2	0.8	2.3	0.2	3.1	0.6
IH30	Transition Reach	Levee	Future	9.7	0.8	2.3	0.2	3.1	0.6
IH02-W	IHNC North of I-10	Structure/Wall	Future	9.7	0.8	2.3	0.2	3.1	0.6
IH01-W	IHNC South of I-10 to Pump Station #19	Structure/Wall	Future	9.4	0.5	3.0	0.3	3.5	0.7
IH04-W	IHNC Lock to Pump Stations (PS#5 and PS#19)	Structure/Wall	Future	9.4	0.5	2.3	0.2	3.1	0.6
IH10	Orleans Pump Stations #5 and Pump Station #19	Structure/Wall	Future	9.4	0.5	2.3	0.2	3.1	0.6
IH03	IHNC Levee South from I-10	Levee	Existing	7.8	0.5	3.0	0.3	3.5	0.7
IH03	IHNC Levee South from I-10	Levee	Future	9.3	0.5	3.0	0.3	3.5	0.7
IH05-W	Dwyer Pump Station	Structure/Wall	Future	9.7	0.8	2.3	0.2	3.1	0.6
NO20	NS Railroad gates near Seabrook (west)	Structure/Wall	Future	10.0	0.8	4.0	0.3	6.2	1.1
NE20	NS Railroad gates near Seabrook (east)	Structure/Wall	Future	10.0	0.8	4.0	0.3	6.2	1.1

Table 33 –IHNC and GIWW Segments inside the MRGO/GIWW Gates – 1% Hydraulic Boundary Conditions

The bed elevation in front of the various levees and floodwalls is estimated as follows. The elevation in front of the levee sections GI01, GI02, GI03, IH03 are set at 0ft NAVD88.2004.65. The elevations for the various floodwalls along GIWW and IHNC and the pump stations at IHNC are assumed to be +1ft NAVD88. The Bienvenue Floodgate has an elevation of -14ft NAVD88. The base of the railroad track at the floodgates near the entrance of the IHNC (NE20 and NO20) is at approximately 6ft NAVD88 2004.65. An average elevation of 0ft was used at a distance of one wave length (≈ 300 ft) from the floodgate.

3.7.3 Project Design Heights

The design characteristics along IHNC and GIWW are summarized in Table 34 below for the situation with MRGO/GIWW closure only. The levee sections are designed for both existing and future conditions. Note that the floodwalls and pump stations are only evaluated for future conditions, because these are hard structures. The structures that include structural superiority of 2ft are Pump Station #5 and Pump Station #19 (IH10) and the floodgates near the entrance of the IHNC (NE20 and NO20).

IHNC and GIWW sections (with MRGO gate only) 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
GI08	Bienvenue Floodgate	Structure/Wall	Future	23.6	15.5	0.002	0.008
GI03	Michoud Canal to Michoud Slip	Levee	Existing	8.1	12.0	0.002	0.025
GI03	Michoud Canal to Michoud Slip	Levee	Future	9.6	13.5	0.002	0.029
GI04	Michoud Canal and Slip	Structure/Wall	Future	9.6	13.5	0.022	0.068
GI07	Grant Pump Station	Structure/Wall	Future	8.5	13.5	0.019	0.060
GI03-W	Floodwall under Paris Rd Bridge	Structure/Wall	Future	8.5	13.5	0.019	0.059
GI02	Paris Road to levee section GI02	Levee	Existing	8.0	12.0	0.009	0.071
GI02	Paris Road to levee section GI02	Levee	Future	9.5	13.5	0.002	0.026
GI01	Levee Section GI02 to IHNC	Levee	Existing	7.9	12.0	0.008	0.063
GI01	Levee Section GI02 to IHNC	Levee	Future	9.4	13.5	0.002	0.023
GI06	Elaine Pump Station	Structure/Wall	Future	10.4	13.5	0.018	0.056
GI05	Amid Pump Station (PS#20)	Structure/Wall	Future	10.4	13.5	0.018	0.054
IH30	Transition Reach	Levee	Existing	8.2	14.5	0.000	0.006
IH30	Transition Reach	Levee	Future	9.7	15.5	0.001	0.010
IH02-W	IHNC North of I-10	Structure/Wall	Future	8.7	13.5	0.004	0.027
IH01-W	IHNC South of I-10	Structure/Wall	Future	8.4	13.5	0.016	0.053
IH04-W	IHNC Lock to Pump Station (PS#5)	Structure/Wall	Future	8.4	13.5	0.016	0.051
IH10	Orleans Pump Stations #5 to Pump Station #19	Structure/Wall	Future	8.4	15.5	0.000	0.001
IH03	IHNC Levee South from I-10	Levee	Existing	8.2	12.5	0.005	0.066
IH03	IHNC Levee South from I-10	Levee	Future	9.7	13.5	0.003	0.049
IH05-W	Dwyer Pump Station	Structure/Wall	Future	8.7	13.5	0.004	0.027
NO20	NS Railroad gates near Seabrook (west)	Structure/Wall	Future	10.0	18.0	0.002	0.009
NE20	NS Railroad gates near Seabrook (east)	Structure/Wall	Future	10.0	18.0	0.002	0.009

Table 34 – IHNC and GIWW Segments inside MRGO/GIWW Gates – 1% Design Information

Figure 47 shows typical design cross-sections for the proposed IHNC levee (IH03) for existing and future conditions. No wave berm is needed here because the wave action in this canal is small. The same cross-section is proposed for the GI01 and GI02 section along the GIWW. The GI03 at the eastern end of GIWW is almost the same but has a milder slope for present conditions (1:5) to meet the design criteria (Figure 48). The cross-section of the transition levee (IH30) has a steep slope (1:3) with a 14.5ft (existing) and 15.5ft (future) elevation (Figure 49).

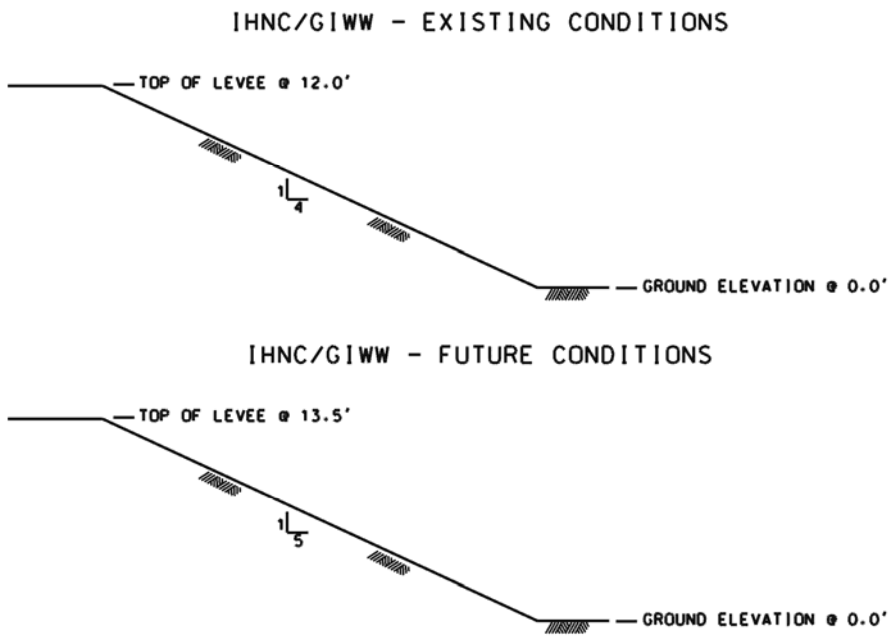


Figure 47 – Typical Levee Design Cross-Section – IHNC (IH03) and GIWW (GI01 and GI02 sections only!) levees for existing (upper panel) and future conditions (lower panel).

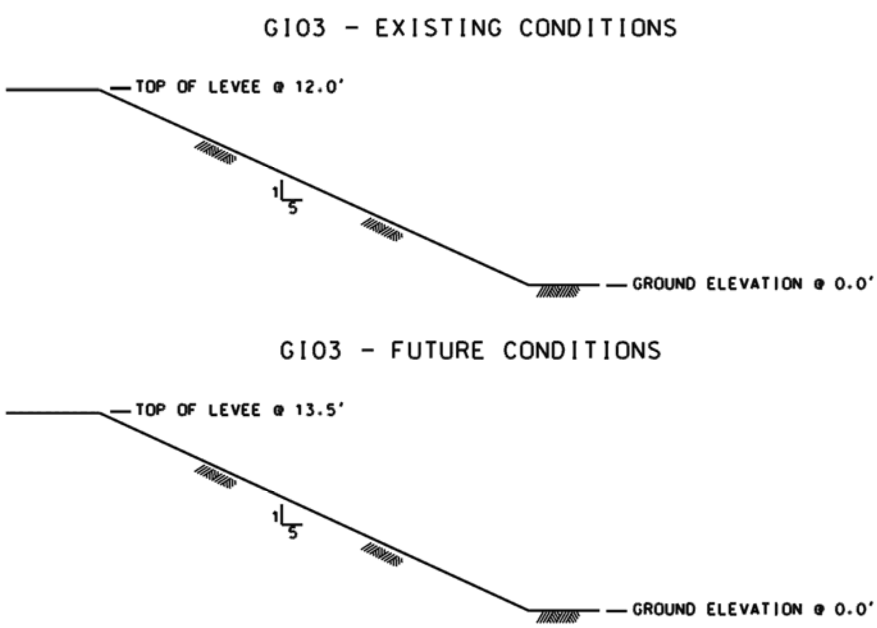


Figure 48 – Typical Levee Design Cross-Section – GI03 section for existing (upper panel) and future conditions (lower panel).

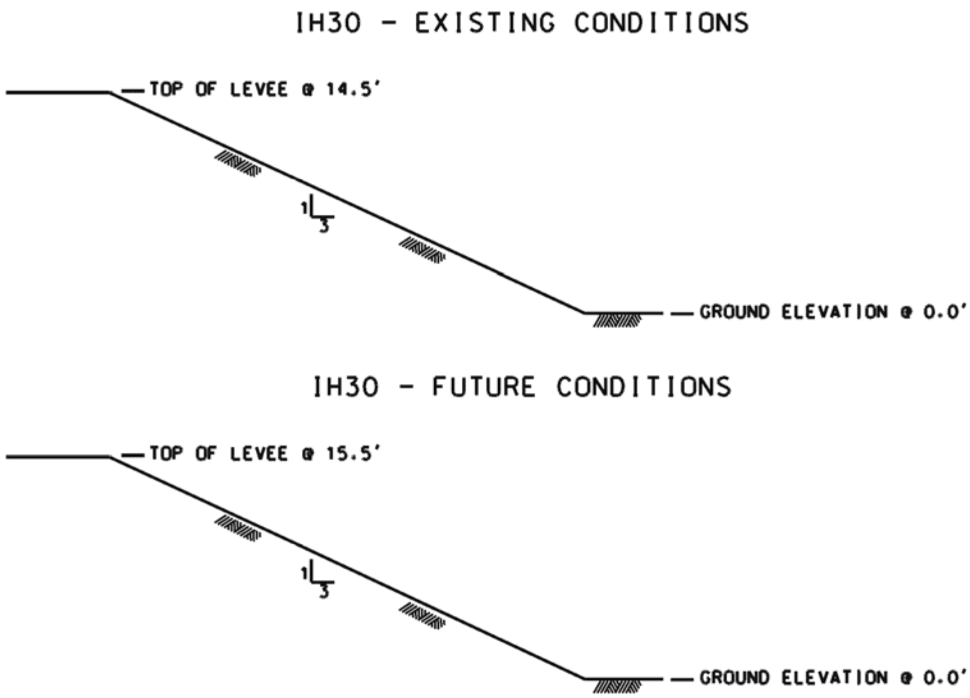


Figure 49 – Typical Levee Design Cross-Section – Transition reach at IHNC (IH30) for existing (upper panel) and future conditions (lower panel).

3.7.4 Wave Forces

Wave forces were computed for all structures within the IHNC and GIWW segment inside the MRGO/GIWW gate with the Goda method, using future conditions. The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics. The following tables summarize the resulting wave forces. Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

IHNC and GIWW (with MRGO closed) Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
GI08	Bienvenue Floodgate	3.5	60.9	3.4	3.5	60.9	3.4
GI04	Michoud Canal and Slip	2.3	14.8	7.5	2.3	14.8	7.5
GI07	Grant PS	2.3	14.7	7.5	2.3	14.7	7.5
GI03-W	Floodwall under Paris Rd Bridge	2.3	14.8	7.5	2.3	14.8	7.5
GI06	Elaine PS	2.4	18.5	6.7	2.4	18.5	6.7
GI05	Amid PS #20	2.5	22.7	6.0	2.5	22.7	6.0
IH02-W	IHNC North of I-10	1.9	21.5	5.4	1.9	21.5	5.4
IH01-W	IHNC South of I-10 to PS#19	2.7	30.3	5.1	2.7	30.3	5.1
IH04-W	IHNC Lock to PS #5 and PS#19	1.5	9.8	7.4	1.5	9.8	7.4
IH10	Oleans PS #5 and PS #19	1.9	22.4	5.5	1.9	22.4	5.5
IH05-W	Dwyer PS	2.0	22.5	5.0	2.0	22.5	5.0
NO20	NS Railroad Gates near Seabrook West	3.1	15.5	9.9	3.1	15.5	9.9
NE20	NS Railroad Gates near Seabrook East	0.9	3.0	11.5	0.9	3.0	11.5

Table 35 – Waves Forces for IHNC and GIWW Segments inside the MRGO/GIWW Gates (50% values)

IHNC and GIWW (with MRGO closed) Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
GI08	Bienvenue Floodgate	4.7	78.0	3.6	4.7	78.0	3.6
GI04	Michoud Canal and Slip	3.2	20.2	7.7	3.2	20.2	7.7
GI07	Grant PS	3.2	20.3	7.7	3.2	20.3	7.7
GI03-W	Floodwall under Paris Rd Bridge	3.2	20.2	7.7	3.2	20.2	7.7
GI06	Elaine PS	3.4	25.5	7.0	3.4	25.5	7.0
GI05	Amid PS #20	3.6	31.3	6.4	3.6	31.3	6.4
IH02-W	IHNC North of I-10	2.5	27.6	5.5	2.5	27.6	5.5
IH01-W	IHNC South of I-10 to PS#19	3.8	40.4	5.6	3.8	40.4	5.6
IH04-W	IHNC Lock to PS #5 and PS#19	2.1	13.4	7.5	2.1	13.4	7.5
IH10	Oleans PS #5 and PS #19	2.7	30.2	5.9	2.7	30.2	5.9
IH05-W	Dwyer PS	2.8	30.2	5.3	2.8	30.2	5.3
NO20	NS Railroad Gates near Seabrook West	4.1	23.1	10.5	4.1	23.1	10.5
NE20	NS Railroad Gates near Seabrook East	1.1	3.8	11.6	1.1	3.8	11.6

Table 36 – Waves Forces for IHNC and GIWW Segments inside the MRGO/GIWW Gates (90% values)

3.7.5 Resiliency

The designs for the levees and structures along IHNC and GIWW inside the MRGO/GIWW gates were examined for resiliency by also computing the overtopping rate for the 0.2 percent event for each design. For these sections, the 0.2% wave characteristics are not known from the STWAVE results. Hence, we have followed the same procedure as for the 1% waves using the empirical formulation from Brettschneider. The assumption for the 0.2% event is that the wind speed is 88 mph (see Appendix C).

The accompanying wave characteristics during a 0.2% event are:

Parameter	**IHNC (north of I-10 and south of Pump Station 5)**	**GIWW and IHNC/GIWW intersection**
Fetch	0.25 mile	0.5 mile
Wind speed	88 mph	88 mph
Water depth	30 ft	40 ft
Significant wave height	2.7ft	3.5ft
Peak period	3.2s	3.8s

Table 37 – Wave characteristics in IHNC and GIWW during 0.2% event

The water level and overtopping rate were determined during the 0.2% event with 50% assurance (best estimates). The results of the resiliency analysis are presented in Table 38. For all sections, the 0.2% surge elevation remains below the top of the flood defense, and the overtopping rate is (much) less than 1 cfs/ft per ft based on the best estimates.

IHNC and GIWW sections (with MRGO gate only) Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
GI08	Bienvenue Floodgate	Structure/Wall	Future	15.5	11.0	0.011
GI03	Michoud Canal to Michoud Slip	Levee	Existing	12.0	9.5	0.034
GI03	Michoud Canal to Michoud Slip	Levee	Future	13.5	11.0	0.034
GI04	Michoud Canal and Slip	Structure/Wall	Future	13.5	11.0	0.123
GI07	Grant Pump Station	Structure/Wall	Future	13.5	10.9	0.108
GI03-W	Floodwall under Paris Rd Bridge	Structure/Wall	Future	13.5	10.9	0.109
GI02	Paris Road to levee section GI02	Levee	Existing	12.0	9.4	0.083
GI02	Paris Road to levee section GI02	Levee	Future	13.5	10.9	0.028
GI01	Levee Section GI02 to IHNC	Levee	Existing	12.0	9.4	0.080
GI01	Levee Section GI02 to IHNC	Levee	Future	13.5	10.9	0.027
GI06	Elaine Pump Station	Structure/Wall	Future	13.5	10.9	0.108
GI05	Amid Pump Station (PS#20)	Structure/Wall	Future	13.5	10.9	0.108
IH30	Transition Reach	Levee	Existing	14.5	10.9	0.017
IH30	Transition Reach	Levee	Future	15.5	12.4	0.036
IH02-W	IHNC North of I-10	Structure/Wall	Future	13.5	12.4	0.300
IH01-W	IHNC South of I-10	Structure/Wall	Future	13.5	10.9	0.105
IH04-W	IHNC Lock to Pump Station (PS#5)	Structure/Wall	Future	13.5	10.9	0.110
IH10	Orleans Pump Stations #5 to Pump Station #19	Structure/Wall	Future	15.5	10.9	0.001
IH03	IHNC Levee South from I-10	Levee	Existing	12.5	10.9	0.377
IH03	IHNC Levee South from I-10	Levee	Future	13.5	12.4	0.507
IH05-W	Dwyer Pump Station	Structure/Wall	Future	13.5	12.4	0.295
NO20	NS Railroad gates near Seabrook (west)	Structure/Wall	Future	18.0	12.8	0.109
NE20	NS Railroad gates near Seabrook (east)	Structure/Wall	Future	18.0	12.8	0.108

Table 38 – Resiliency for IHNC and GIWW Segments inside MRGO/GIWW Gates

3.8 IHNC/GIWW (with MRGO/GIWW and Seabrook closures)

3.8.1 General

This section presents the 1% hydraulic design characteristics and the design heights with the MRGO/GIWW closure and the Seabrook closure. Both closures seal off the entire canal system rom the influence of surges from Lake Borgne and Lake Pontchartrain. For an extensive description of the IHNC/GIWW area, the reader is referred to Section 3.7.1.

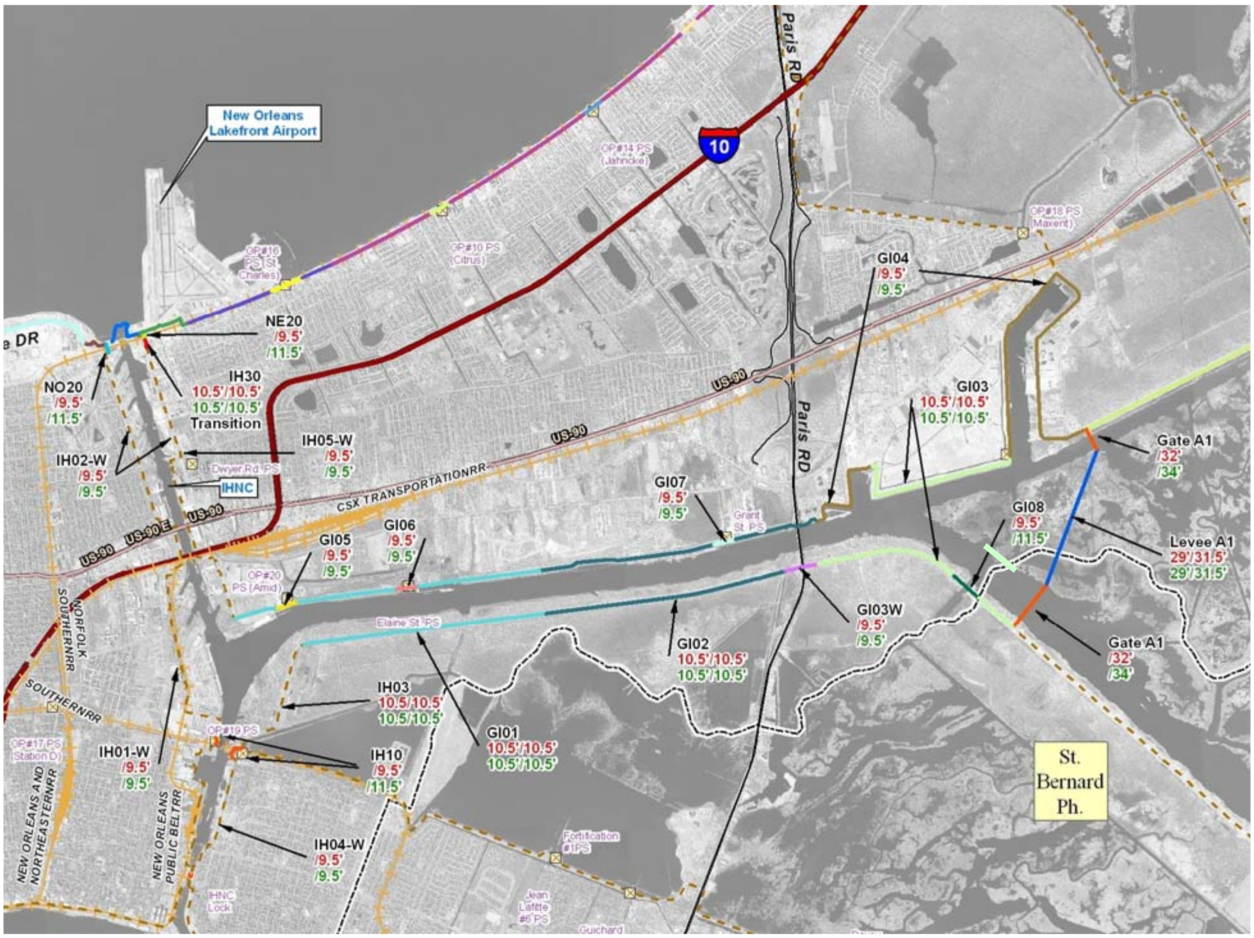


Figure 50 – Levee and floodwall sections in IHNC/GIWW area (with MRGO/GIWW and Seabrook closures).

3.8.2 Hydraulic Boundary Conditions

The hydraulic design characteristics (surge levels, wave characteristics) for the sections along IHNC and GIWW are listed in Table 39. The surge level is purely governed by the closure strategy of the two barriers and the drainage into the canals. Herein, we assumed a 50% (2831cfs) pumping capacity for the 6 stations pumping into the area. Assumed gates would be closed at a surge elevation of +3ft and remained closed for 10 hours. Based on LIDAR we computed a storage-elevation curve for the area behind the gates at Seabrook and

GIWW/MRGO. Next, a 100 yr rainfall event was imposed into the interior areas which are pumped into the IHNC and the IHNC/GIWW. The storage needed for this drainage volume appears to be around 3ft. The maximum surge level was therefore set at +6ft. Because the water level in IHNC/GIWW is fully controlled in this case, the 1% surge level is kept the same for existing and future conditions.

The wave characteristics in Table 39 are equivalent to the ones that have been used for a situation with the MRGO/GIWW closure only. These wave characteristics have been based on empirical relationships because the STWAVE model does not have enough resolution to solve the waves properly in these narrow canals. For a discussion about the derivation of these wave characteristics, the reader is referred to Section 3.7.2.

IHNC and GIWW sections (with MRGO/GIWW and Seabrook closures) 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
GI08	Bienvenue Floodgate	Structure/Wall	Future	6.0	0.5	3.0	0.3	3.5	0.7
GI03	Michoud Canal to Michoud Slip and Paris Rd Bridge to Bienvenue Floodgate	Levee	Existing	6.0	0.5	3.0	0.3	3.5	0.7
GI03	Michoud Canal to Michoud Slip and Paris Rd Bridge to Bienvenue Floodgate	Levee	Future	6.0	0.5	3.0	0.3	3.5	0.7
GI04	Michoud Canal and Slip	Structure/Wall	Future	6.0	0.5	3.0	0.3	3.5	0.7
GI07	Grant Pump Station	Structure/Wall	Future	6.0	0.5	3.0	0.3	3.5	0.7
GI03-W	Floodwall under Paris Rd Bridge	Structure/Wall	Future	6.0	0.5	3.0	0.3	3.5	0.7
GI02	Paris Road to levee section GI01	Levee	Existing	6.0	0.5	3.0	0.3	3.5	0.7
GI02	Paris Road to levee section GI01	Levee	Future	6.0	0.5	3.0	0.3	3.5	0.7
GI01	Levee Section GI02 to IHNC	Levee	Existing	6.0	0.5	3.0	0.3	3.5	0.7
GI01	Levee Section GI02 to IHNC	Levee	Future	6.0	0.5	3.0	0.3	3.5	0.7
GI06	Elaine Pump Station	Structure/Wall	Future	6.0	0.5	3.0	0.3	3.5	0.7
GI05	Amid Pump Station (PS#20)	Structure/Wall	Future	6.0	0.5	3.0	0.3	3.5	0.7
IH30	Transition Reach	Levee	Existing	6.0	0.5	2.3	0.2	3.1	0.6
IH30	Transition Reach	Levee	Future	6.0	0.5	2.3	0.2	3.1	0.6
IH02-W	IHNC North of I-10	Structure/Wall	Future	6.0	0.5	2.3	0.2	3.1	0.6
IH01-W	IHNC South of I-10 to Pump Station #19	Structure/Wall	Future	6.0	0.5	3.0	0.3	3.5	0.7
IH04-W	IHNC Lock to Pump Stations (PS#5 and PS#19)	Structure/Wall	Future	6.0	0.5	2.3	0.2	3.1	0.6
IH10	Orleans Pump Stations #5 and Pump Station #19	Structure/Wall	Future	6.0	0.5	2.3	0.2	3.1	0.6
IH03	IHNC Levee South from I-10	Levee	Existing	6.0	0.5	3.0	0.3	3.5	0.7
IH03	IHNC Levee South from I-10	Levee	Future	6.0	0.5	3.0	0.3	3.5	0.7
IH05-W	Dwyer Pump Station	Structure/Wall	Future	6.0	0.5	2.3	0.2	3.1	0.6
NO20	NS Railroad gates near Seabrook (west)	Structure/Wall	Future	6.0	0.5	2.3	0.2	3.1	0.6
NE20	NS Railroad gates near Seabrook (east)	Structure/Wall	Future	6.0	0.5	2.3	0.2	3.1	0.6

Table 39 – IHNC and GIWW Segments inside the MRGO/GIWW and Seabrook Gates – 1% Hydraulic Boundary Conditions

The bed elevation in front of the various levees and floodwalls is estimated as follows. The elevation in front of the levee sections GI01, GI02, GI03, IH03 are set at 0ft NAVD88.2004.65. The elevation for the various floodwalls along GIWW and IHNC and the pump stations at IHNC is assumed to be +1ft NAVD88.2004.65. The Bienvenue Floodgate has an elevation of -14ft NAVD88.

3.8.3 Project Design Heights

The design characteristics of the IHNC and GIWW sections are summarized in Table 40 below for the situation with MRGO/GIWW and Seabrook closures. The levee sections are designed for both existing and future conditions. Note that the floodwalls and pump stations are only evaluated for future conditions, because these are hard structures. The structures that include structural superiority of 2ft are Pump Station #5 and Pump Station #19 (IH10), the NS Railroad Gates near Seabrook (NE20 and NO20) and Bienvenue Floodgate (GI08).

IHNC and GIWW sections (with MRGO/GIWW and Seabrook closures) 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
GI08	Bienvenue Floodgate	Structure/Wall	Future	20.0	11.5	0.003	0.013
GI03	Michoud Canal to Michoud Slip and	Levee	Existing	6.0	10.5	0.010	0.058
GI03	Michoud Canal to Michoud Slip and	Levee	Future	6.0	10.5	0.010	0.057
GI04	Michoud Canal and Slip	Structure/Wall	Future	6.0	9.5	0.009	0.043
GI07	Grant Pump Station	Structure/Wall	Future	5.0	9.5	0.002	0.018
GI03-W	Floodwall under Paris Rd Bridge	Structure/Wall	Future	5.0	9.5	0.002	0.018
GI02	Paris Road to levee section GI01	Levee	Existing	6.0	10.5	0.010	0.057
GI02	Paris Road to levee section GI01	Levee	Future	6.0	10.5	0.009	0.055
GI01	Levee Section GI02 to IHNC	Levee	Existing	6.0	10.5	0.009	0.056
GI01	Levee Section GI02 to IHNC	Levee	Future	6.0	10.5	0.010	0.056
GI06	Elaine Pump Station	Structure/Wall	Future	7.0	9.5	0.022	0.076
GI05	Amid Pump Station (PS#20)	Structure/Wall	Future	7.0	9.5	0.021	0.075
IH30	Transition Reach	Levee	Existing	6.0	10.5	0.004	0.029
IH30	Transition Reach	Levee	Future	6.0	10.5	0.003	0.029
IH02-W	IHNC North of I-10	Structure/Wall	Future	5.0	9.5	0.002	0.015
IH01-W	IHNC South of I-10 to Pump Station #19	Structure/Wall	Future	5.0	9.5	0.002	0.017
IH04-W	IHNC Lock to Pump Stations (PS#5 and PS#19)	Structure/Wall	Future	5.0	9.5	0.002	0.015
IH10	Orleans Pump Stations #5 and Pump Station #19	Structure/Wall	Future	5.0	11.5	0.000	0.001
IH03	IHNC Levee South from I-10	Levee	Existing	6.0	10.5	0.009	0.056
IH03	IHNC Levee South from I-10	Levee	Future	6.0	10.5	0.010	0.056
IH05-W	Dwyer Pump Station	Structure/Wall	Future	5.0	9.5	0.002	0.015
NO20	NS Railroad gates near Seabrook (west)	Structure/Wall	Future	5.0	11.5	0.000	0.001
NE20	NS Railroad gates near Seabrook (east)	Structure/Wall	Future	5.0	11.5	0.000	0.001

Table 40 – IHNC and GIWW Segments inside MRGO/GIWW and Seabrook Gates – 1% Design Information

The typical cross-section for the IHNC/GIWW levee sections is shown in Figure 51. Notice that the existing and future conditions are equivalent because the surge level will probably not change because it is fully controlled by the gates. The wave characteristics are also not changed for future conditions because these are dominated by the fetch (and not depth-limited).

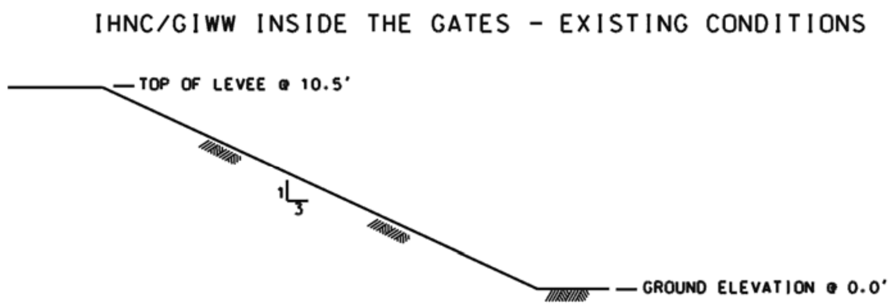


Figure 51 – Typical Levee Design Cross-Section – IHNC/GIWW for existing and future conditions with Seabrook and MRGO/GIWW gates.

3.8.4 Wave Forces

Wave forces were computed for all structures within the IHNC and GIWW segment inside the MRGO/GIWW and Seabrook gate with the Goda method, using future conditions. The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics. The following tables summarize the resulting wave forces. Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

IHNC and GIWW (with Seabrook and MRGO closed) Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
GI08	Bienvenue Floodgate	3.4	48.5	0.3	3.4	48.5	0.3
GI04	Michoud Canal and Slip	2.1	8.6	5.1	2.1	8.6	5.1
GI07	Grant Pump Station	2.1	8.6	5.1	2.1	8.6	5.1
GI03-W	Floodwall under Paris Rd Bridge	2.1	8.6	5.1	2.1	8.6	5.1
GI06	Elaine Pump Station	2.3	12.1	4.3	2.3	12.1	4.3
GI05	Amid Pump Station (PS#20)	2.4	15.8	3.5	2.4	15.8	3.5
IH02-W	IHNC North of I-10	1.8	15.2	2.5	1.8	15.2	2.5
IH01-W	IHNC South of I-10 to PS#19	2.7	22.2	2.3	2.7	22.2	2.3
IH04-W	IHNC Lock to PS #5 and PS#19	1.3	5.4	5.1	1.3	5.4	5.1
IH10	Oleans PS #5 and PS #19	1.9	16.7	2.8	1.9	16.7	2.8
IH05-W	Dwyer Pump Station	1.8	15.2	2.5	1.8	15.2	2.5
NO20	NS Railroad Gates near Seabrook West	0.3	0.7	7.2	0.3	0.7	7.2
NE20	NS Railroad gates near Seabrook (east)	N/A	N/A	N/A	N/A	N/A	N/A

Table 41 – Waves Forces for IHNC and GIWW Segments inside the MRGO/GIWW and Seabrook Gates (50% values).

IHNC and GIWW (with Seabrook and MRGO closed) Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
GI08	Bienvenue Floodgate	4.6	64.2	0.8	4.6	64.2	0.8
GI04	Michoud Canal and Slip	2.7	11.2	5.2	2.7	11.2	5.2
GI07	Grant Pump Station	2.7	11.2	5.2	2.7	11.2	5.2
GI03-W	Floodwall under Paris Rd Bridge	2.7	11.2	5.2	2.7	11.2	5.2
GI06	Elaine Pump Station	3.0	16.1	4.3	3.0	16.1	4.3
GI05	Amid Pump Station (PS#20)	3.3	21.4	3.6	3.3	21.4	3.6
IH02-W	IHNC North of I-10	2.5	20.5	2.8	2.5	20.5	2.8
IH01-W	IHNC South of I-10 to PS#19	3.7	30.1	2.7	3.7	30.1	2.7
IH04-W	IHNC Lock to PS #5 and PS#19	1.7	7.4	5.2	1.7	7.4	5.2
IH10	Oleans PS #5 and PS #19	2.7	23.7	3.1	2.7	23.7	3.1
IH05-W	Dwyer Pump Station	2.5	20.5	2.8	2.5	20.5	2.8
NO20	NS Railroad Gates near Seabrook West	0.4	0.8	7.2	0.4	0.8	7.2
NE20	NS Railroad gates near Seabrook (east)	N/A	N/A	N/A	N/A	N/A	N/A

Table 42 – Waves Forces for IHNC and GIWW Segments inside the MRGO/GIWW and Seabrook Gates (90% values).

3.8.5 Resiliency

For this special case with two closures the designs for the levees and structures along IHNC and GIWW have not been evaluated against resiliency. The reason is that the 0.2% hydraulic load in this case is not well-defined. The hydraulic characteristics inside the canal systems are dependent on: 1) rainfall and interior drainage, and 2) overtopping over the closure gates. We recommend an additional resiliency analysis for this situation.

3.9 Closures at GIWW/MRGO and Seabrook

3.9.1 General

The closure complex at MRGO/GIWW will consist of 2 navigable floodgates, one in the MRGO and the other in the GIWW, connected by an earthen levee. The levee and closure gate in this section have been designed in a similar way as the levees and floodwalls of all other sections. The same overtopping criteria have been applied to the 2 navigable gates and the levee in between. Whether this is true or not is subject for further research in the design of the gates. The design elevations based on the design criteria in this report are depicted in Figure 52. The design sections that are discussed in this paragraph are Gate A1 (MRGO/GIWW closure gate) and Levee A1 (MRGO/GIWW closure levee). As noted in previous sections, the location of the closures is conceptual and will be finalized during the design-build process.

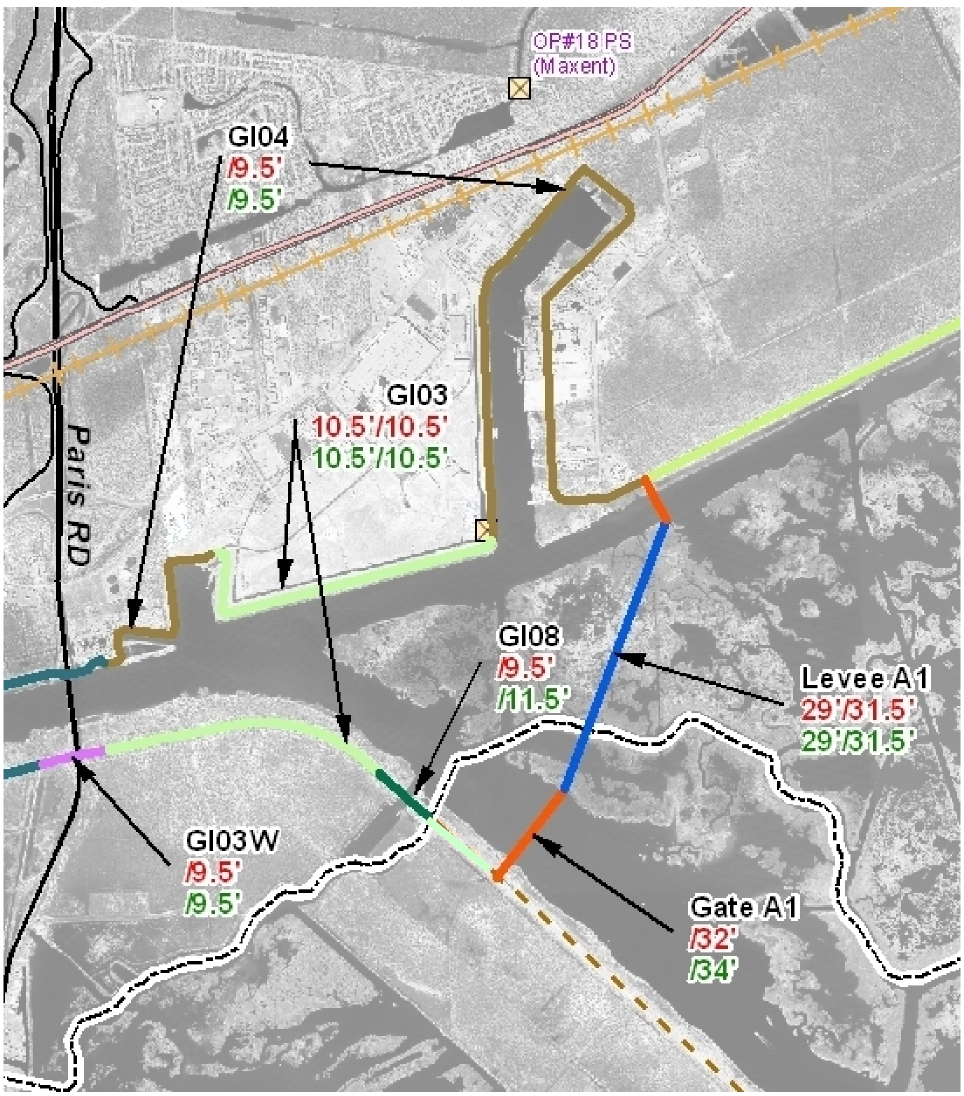


Figure 52 – Levee and gate sections at MRGO/GIWW gates

The closure complex at Seabrook will consist of 1 navigable floodgate. The same overtopping criteria have been applied to the Seabrook gate. Whether this is true or not is subject for further research in the design of the gates. The design elevations based on the design criteria in this report are depicted in Figure 53. Only the Seabrook Gate (Gate A2) is discussed in this paragraph, the other sections have been discussed in previous paragraphs.

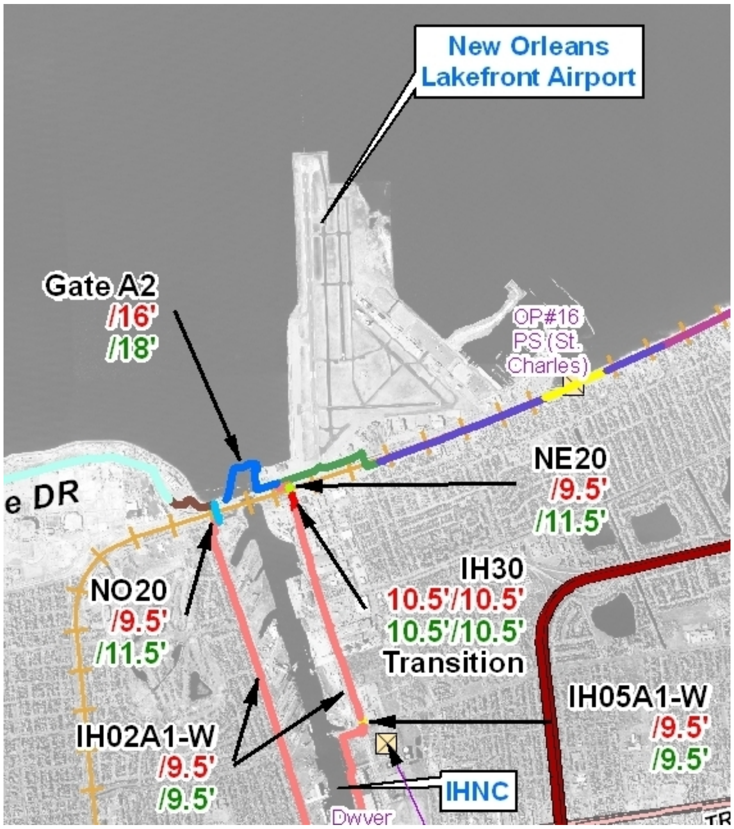


Figure 53 – Gate section at Seabrook

3.9.2 Hydraulic Boundary Conditions

The hydraulic design characteristics for the levee and the gate at the MRGO/GIWW closure are listed in the table below. The existing hydraulic conditions are based on the JPM-OS method using the results from 2010 ADCIRC and STWAVE models. The future conditions are derived by adding 1.5 ft to the surge elevation, and adding 0.75 ft to the wave height. The wave period is computed using the assumption that the wave steepness remains constant. For more information, see Chapter 2.

Closure at GIWW and MRGO (Gate A1, Levee A1): The ground elevation in front of the gates is assumed to be -20.0 ft and in front of the levee 0.0 ft. Notice that the 1% wave heights are depth-limited for levee section only.

MRGO-GIWW and Seabrook Closure Sections 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
GATE-A1	Closure gate at MRGO - GIWW intersection	Structure/Wall	Future	19.9	1.0	7.2	0.6	8.4	1.6
LEVEE-A1	Closure levee at MRGO - GIWW intersection	Levee	Existing	18.4	1.0	7.1	0.7	7.9	1.6
LEVEE-A1	Closure levee at MRGO - GIWW intersection	Levee	Future	19.9	1.0	7.9	0.7	8.3	1.6
GATE-A2	Closure gate at Seabrook	Structure/Wall	Future	10.0	0.8	4.0	0.3	6.2	1.1

Table 43 - MRGO/GIWW and Seabrook Gates and Levee – 1% Hydraulic Boundary Conditions

Closure at Seabrook (Gate A2): The area in front of the Seabrook is relatively shallow although the narrow navigation channel is deep. Therefore, the ground elevation in front of the gates is assumed to be 0 ft to determine the wave characteristics. The 1% wave height is depth-limited in this case. The exact location of the Seabrook gate is not known yet. Furthermore, the STWAVE model has a relatively coarse resolution and the bed geometry is relatively complicated in this case. Therefore, we recommended more detailed wave analysis for the Seabrook gate to establish more accurate wave conditions.

3.9.3 Project Design Heights

The design characteristics of the gate and the levee of the MRGO/GIWW closure complex are summarized in Table 44 below. The levee sections are designed for both existing and future conditions. Note that the gate is only evaluated for future conditions, because it is a hard structure. The MRGO/GIWW structure and the Seabrook structure both include structural superiority of 2ft.

MRGO-GIWW and Seabrook Closure Sections 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
GATE-A1	Closure gate at MRGO - GIWW intersection	Structure/Wall	Future	39.9	34.0	0.007	0.027
LEVEE-A1	Closure levee at MRGO - GIWW intersection	Levee	Existing	18.4	29.0	0.008	0.082
LEVEE-A1	Closure levee at MRGO - GIWW intersection	Levee	Future	19.9	31.5	0.009	0.089
GATE-A2	Closure gate at Seabrook	Structure/Wall	Future	10.0	18.0	0.002	0.009

Table 44 – MRGO/GIWW and Seabrook Gates and Levee – 1% Design Information

Figure 54 presents the MRGO/GIWW closure levee for existing and future conditions. The wave berm at the surge elevation is necessary to reduce the wave overtopping. For future conditions, the wave berm and the crest elevation should be raised to meet the design criteria. Notice that this levee design has been based on the same design criteria as all other sections. This might not be the case and these cross-sections will obviously change if other criteria are applied.

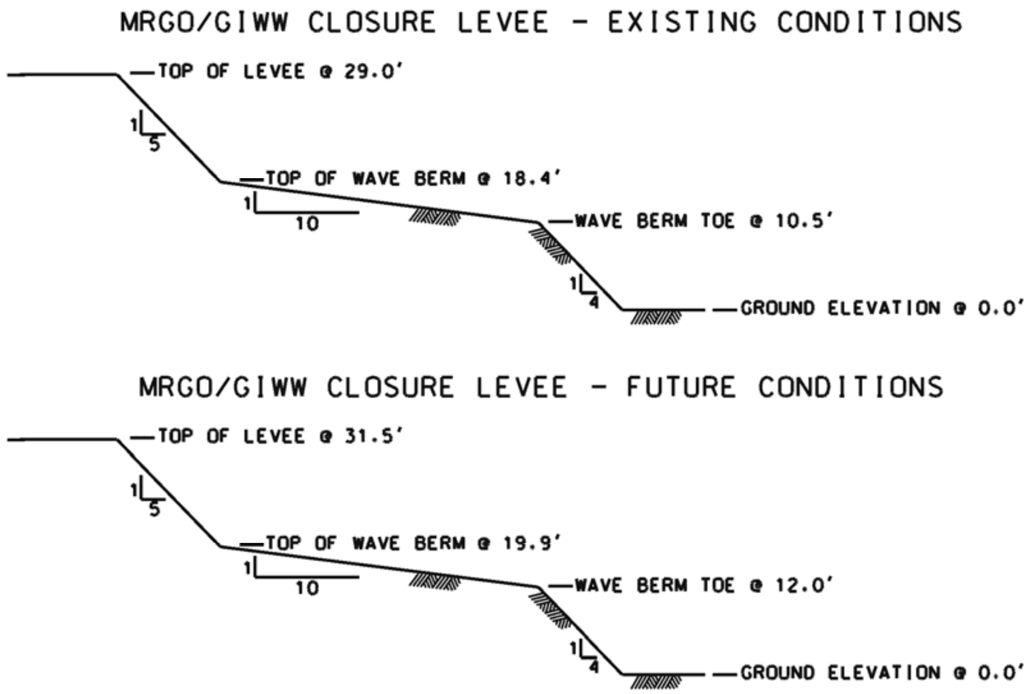


Figure 54 – Typical Levee Design Cross-Section – Levee Closure (Levee Gate A1) for existing (upper panel) and future conditions (lower panel).

3.9.4 Wave Forces

Wave forces were computed for both closure structures with the Goda method, using future conditions. The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics. The following tables summarize the resulting wave forces. Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

MRGO and Seabrook closure gates Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
GATE-A1	MRGO / GIWW gate closure	14.2	222.5	15.6	15.6	244.0	15.6
GATE-A2	Seabrook gate closure	4.3	34.9	8.2	4.7	38.5	8.2

Table 45 – Waves Forces for MRGO/GIWW and Seabrook Gates (50% values)

MRGO and Seabrook closure gates Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
GATE-A1	MRGO / GIWW gate closure	18.8	300.9	16.0	20.3	324.8	16.0
GATE-A2	Seabrook gate closure	5.6	47.7	8.5	6.1	51.3	8.5

Table 46 – Waves Forces for MRGO/GIWW and Seabrook Gates (90% values)

3.9.5 Resiliency

The designs for the levee and the gate structure at MRGO/GIWW closure complex were examined for resiliency by also computing the overtopping rate for the 0.2 percent event for each design. The water level and overtopping rate was determined for the 50% assurance during the 0.2% event. The results are presented in Table 47. For all sections, the 0.2% surge elevation remains below the top of the flood defense, and the overtopping rate is less than 2 cfs/ft per ft (best estimates).

MRGO-GIWW and Seabrook Closure Sections Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
GATE-A1	Closure gate at MRGO - GIWW intersection	Structure/Wall	Future	34.0	23.6	1.780
LEVEE-A1	Closure levee at MRGO - GIWW intersection	Levee	Existing	29.0	22.1	1.780
LEVEE-A1	Closure levee at MRGO - GIWW intersection	Levee	Future	31.5	23.6	1.780
GATE-A2	Closure gate at Seabrook	Structure/Wall	Future	18.0	12.8	1.390

Table 47 – Resiliency for MRGO/GIWW and Seabrook Gates

3.10 St. Bernard Parish

3.10.1 General

The Chalmette Loop and Chalmette Extension is the Hurricane Protection system which, in combination with the Mississippi River levees, completely isolates and protects St. Bernard Parish and that portion of Orleans Parish east of the Inner Harbor Navigation Canal (IHNC) from storm surge flooding. Analyses of levees along the IHNC and GIWW, which form part of that line of protection, are covered in sections 3.7 and 3.8 of this report. The remaining reaches of the Chalmette Loop and Chalmette Extension components of the Hurricane Protection System have been divided into 6 segments, SB11 through SB17. Segments locations and design elevations are shown in Figure 55.

Because of the available land, enlargement of existing earthen levees are proposed. Levee segments SB11, SB12, SB13, and a portion of SB15 define the levee heights along the current levee alignment parallel to the MRGO. Levee reaches SB15, SB16, and SB17 cover the levee from the MRGO to northward turn in the levee at Caernarvon ending at the Mississippi River Levee.

In addition to levees, floodwalls are incorporated into the line of protection. Because of the expense associated with their replacement, these floodwalls have been designed to future design elevations. However, this does not eliminate the need for reevaluation of the project design and its design parameters in the future to insure a consistent degree of protection. The Bayou Dupre Control structure (SB19) is a floodgate and is located within segment SB13. The St. Mary's Pump Station (SB20) is within segment SB16. Tie-in floodwalls adjoin the Bayou Dupre control structure and front the St. Mary Pumping Station.

Figure 55 – Levees, Floodwalls and Pump Stations in the St. Bernard Parish

3.10.2 Hydraulic Boundary Conditions

The hydraulic design characteristics for the sections are listed in Table 48. The existing hydraulic conditions are based on the JPM-OS method using the results from ADCIRC and STWAVE. The future conditions are derived by adding 1.5 ft to the surge elevation, and adding 0.75 ft to the wave height. The wave period is computed using the assumption that the wave steepness remains constant. For more information, see Chapter 2.

Notice that the hydraulic boundary conditions have been based on numerical computations using the 2010 grid with the gates at MRGO and GIWW in place. The effect on the 1% surge levels near the gates is about +1ft. Because of the higher surge levels, the wave height and period also increase in the surrounding of the gates. For all sections, the bed elevation in front of the levee/floodwall has been assumed to be 0ft NAVD88.2004.065.

St Bernard Parish Sections 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
SB11	MRGO levee	Levee	Existing	18.4	1.0	7.1	0.7	7.9	1.6
SB11	MRGO levee	Levee	Future	19.9	1.0	7.9	0.7	8.3	1.6
SB12	MRGO levee	Levee	Existing	17.3	1.1	6.9	0.7	5.9	1.2
SB12	MRGO levee	Levee	Future	18.8	1.1	7.5	0.7	6.2	1.2
SB13	MRGO levee	Levee	Existing	16.4	1.1	6.6	0.7	6.3	1.3
SB13	MRGO levee	Levee	Future	17.9	1.1	7.2	0.7	6.6	1.3
SB15	MRGO levee	Levee	Existing	15.6	1.2	5.4	0.5	8.9	1.8
SB15	MRGO levee	Levee	Future	17.1	1.2	6.2	0.5	9.5	1.8
SB16	Caernarvon levee	Levee	Existing	17.5	1.1	5.4	0.5	8.4	1.7
SB16	Caernarvon levee	Levee	Future	19.0	1.1	6.2	0.5	8.9	1.7
SB17	Caernarvon levee	Levee	Existing	18.0	1.2	5.1	0.5	8.1	1.6
SB17	Caernarvon levee	Levee	Future	19.5	1.2	5.9	0.5	8.7	1.6
SB19	Bayou Dupre Control structure	Structure/Wall	Future	17.3	1.0	5.6	0.5	6.5	1.2
SB20	St Mary Pump Station (PS#8)	Structure/Wall	Future	18.5	1.0	6.2	0.5	8.6	1.6

Table 48 – St. Bernard Parish Segments - 1% Hydraulic Boundary Conditions

3.10.3 Project Design Heights

The design characteristics along the Chalmette Loop and Chalmette Extension are summarized in Table 49. The levee sections are designed for both existing and future conditions. Note that the floodwalls and pump stations are only evaluated for future conditions, because these are hard structures. Both structures (SB19 and SB20) include structural superiority of 2ft.

St Bernard Parish Sections 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
SB11	MRGO levee	Levee	Existing	18.4	29.0	0.008	0.083
SB11	MRGO levee	Levee	Future	19.9	31.5	0.009	0.091
SB12	MRGO levee	Levee	Existing	17.3	27.5	0.001	0.019
SB12	MRGO levee	Levee	Future	18.8	30.0	0.002	0.022
SB13	MRGO levee	Levee	Existing	16.4	26.5	0.002	0.027
SB13	MRGO levee	Levee	Future	17.9	29.0	0.002	0.030
SB15	MRGO levee	Levee	Existing	15.6	26.5	0.005	0.062
SB15	MRGO levee	Levee	Future	17.1	29.0	0.007	0.077
SB16	Caernarvon levee	Levee	Existing	17.5	26.5	0.007	0.087
SB16	Caernarvon levee	Levee	Future	19.0	29.0	0.006	0.072
SB17	Caernarvon levee	Levee	Existing	18.0	26.5	0.002	0.040
SB17	Caernarvon levee	Levee	Future	19.5	29.0	0.009	0.097
SB19	Bayou Dupre Control structure	Structure/Wall	Future	17.3	31.0	0.001	0.004
SB20	St Mary Pump Station (PS#8)	Structure/Wall	Future	18.5	30.5	0.006	0.023

Table 49 – St. Bernard Parish Segments - 1% Design Information

The proposed levee design for both existing conditions and future conditions consist of a levee with 1V:4H or 1V:5H slopes, fronted by a wave berm at the 1% surge elevation. The slope of the wave berm varies for each reach between 1V:8H and 1V:12H. For future conditions, the crest elevation and the wave berm have to be raised. Typical design cross-sections are shown in Figure 56 - Figure 61 for all levee sections.

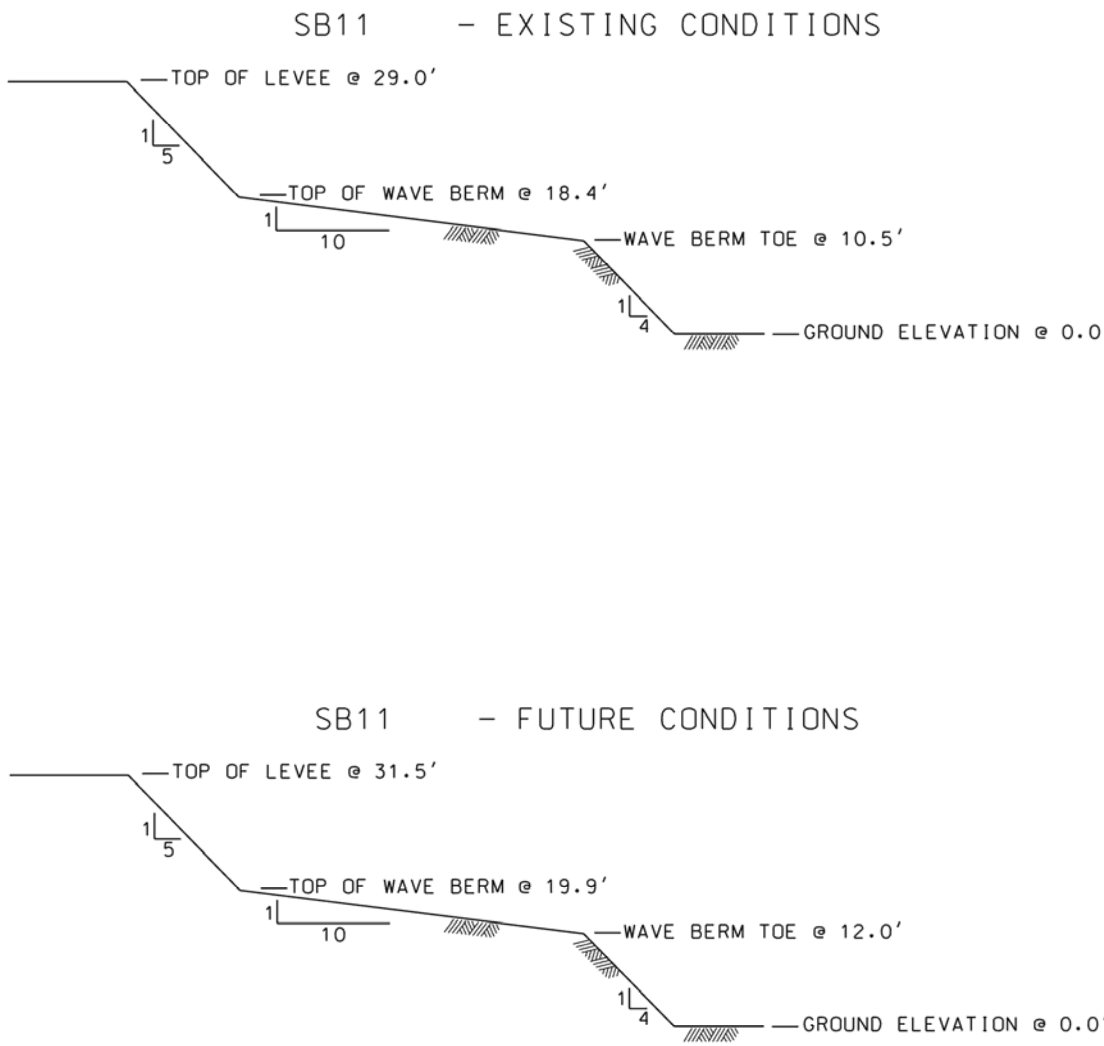


Figure 56 – Typical Levee Design Cross-Section SB11 for existing (upper panel) and future conditions (lower panel).

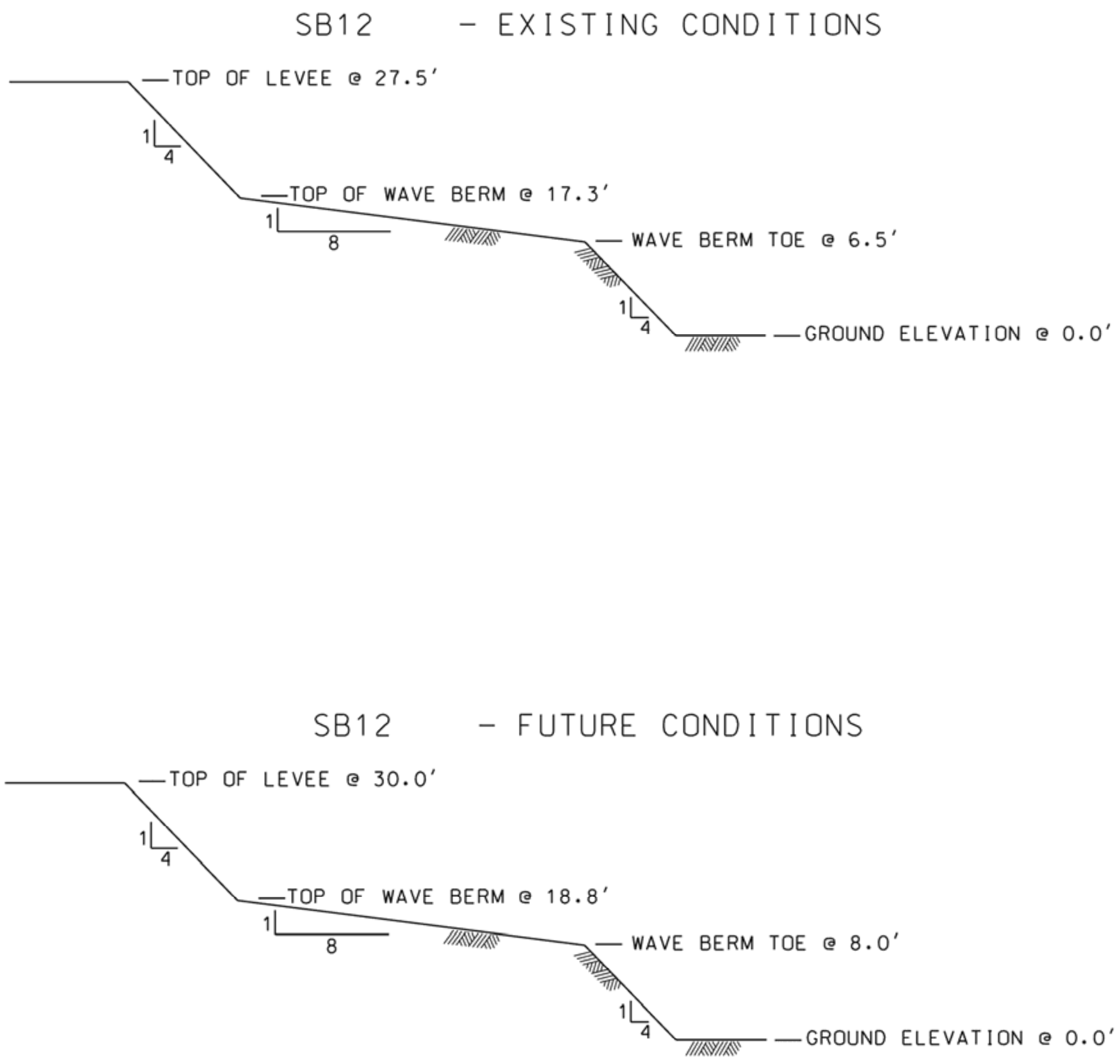


Figure 57 – Typical Levee Design Cross-Section SB12 for existing (upper panel) and future conditions (lower panel).

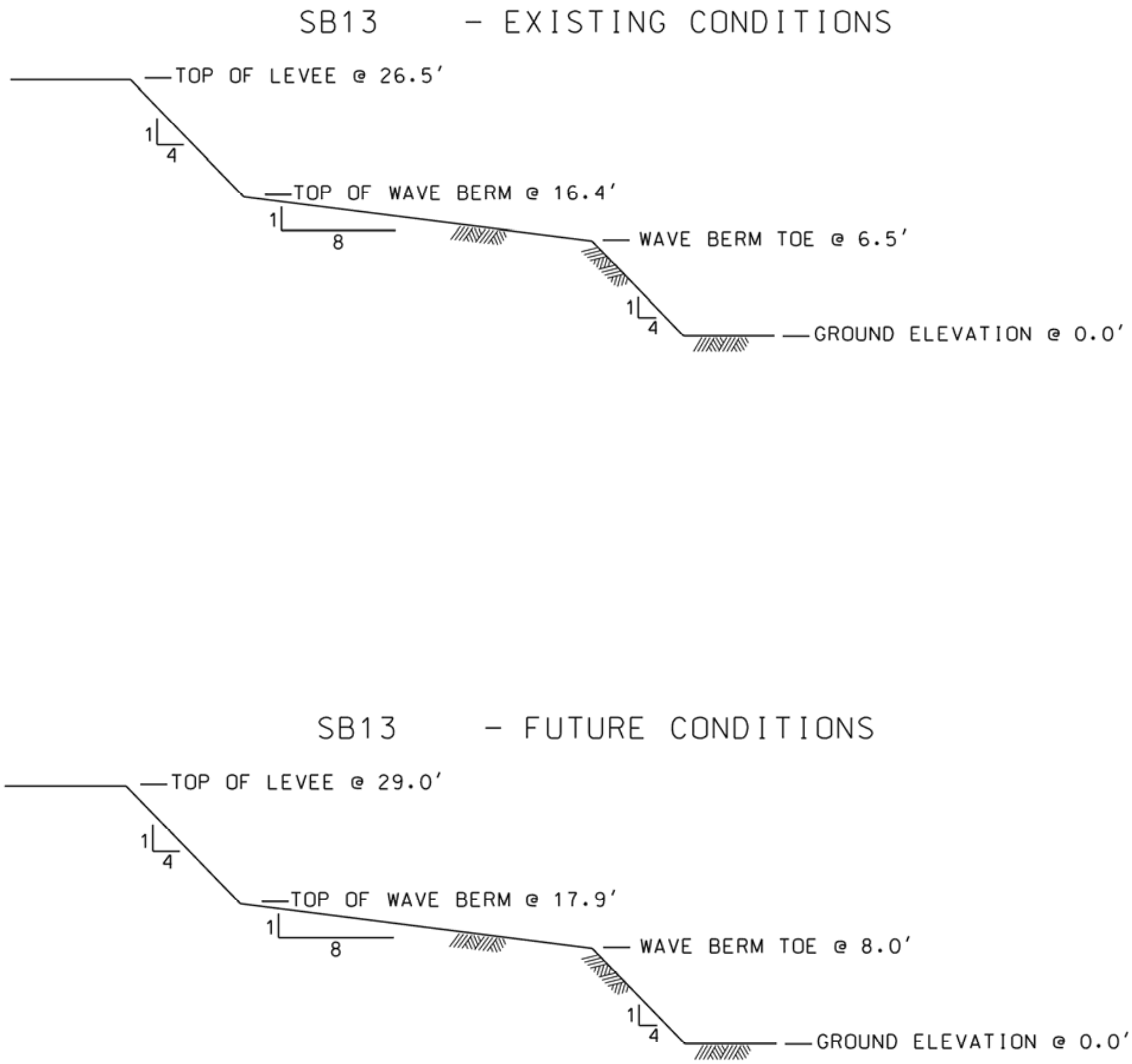
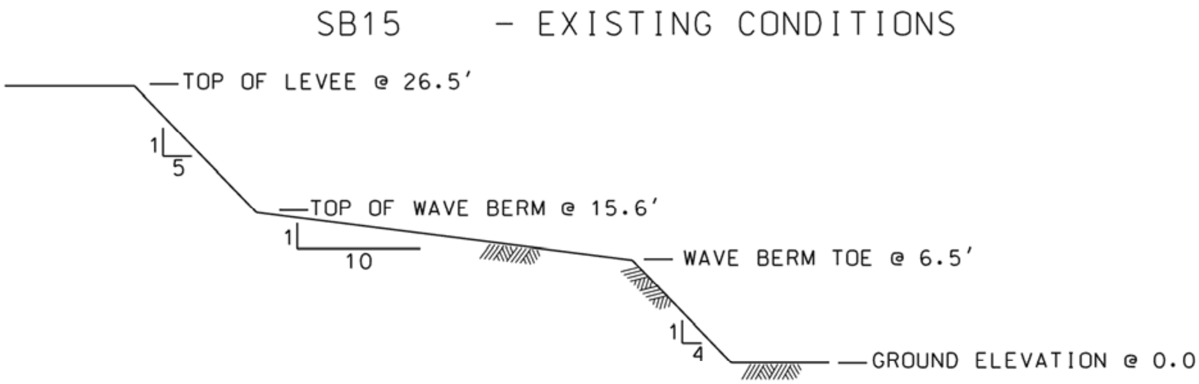


Figure 58 – Typical Levee Design Cross-Section SB13 for existing (upper panel) and future conditions (lower panel).



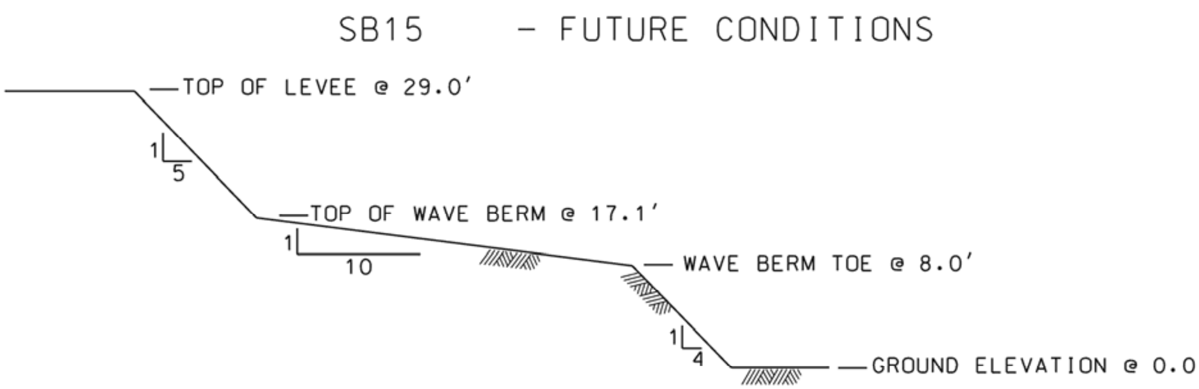
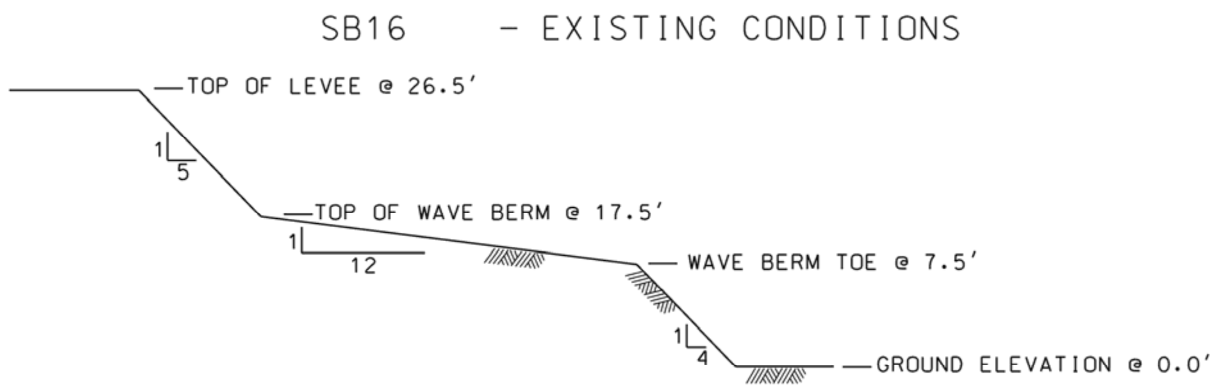


Figure 59 – Typical Levee Design Cross-Section SB15 for existing (upper panel) and future conditions (lower panel).



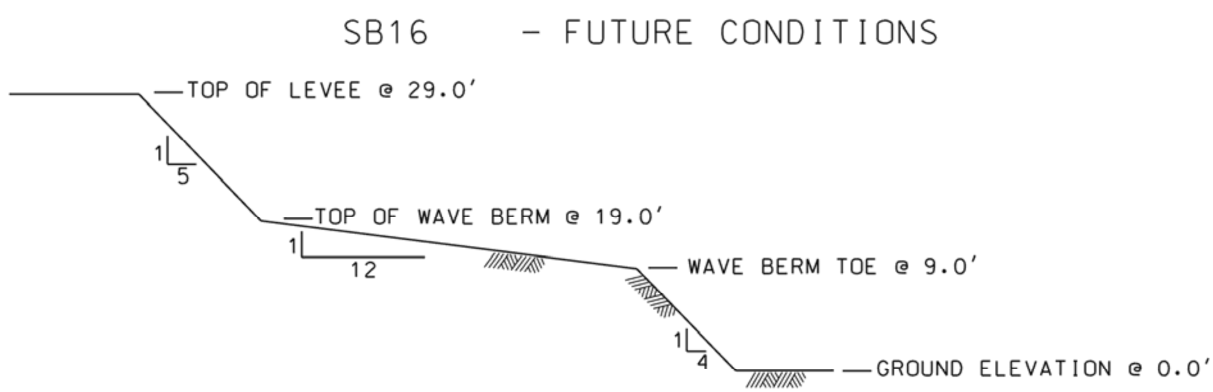


Figure 60 – Typical Levee Design Cross-Section SB16 for existing (upper panel) and future conditions (lower panel).

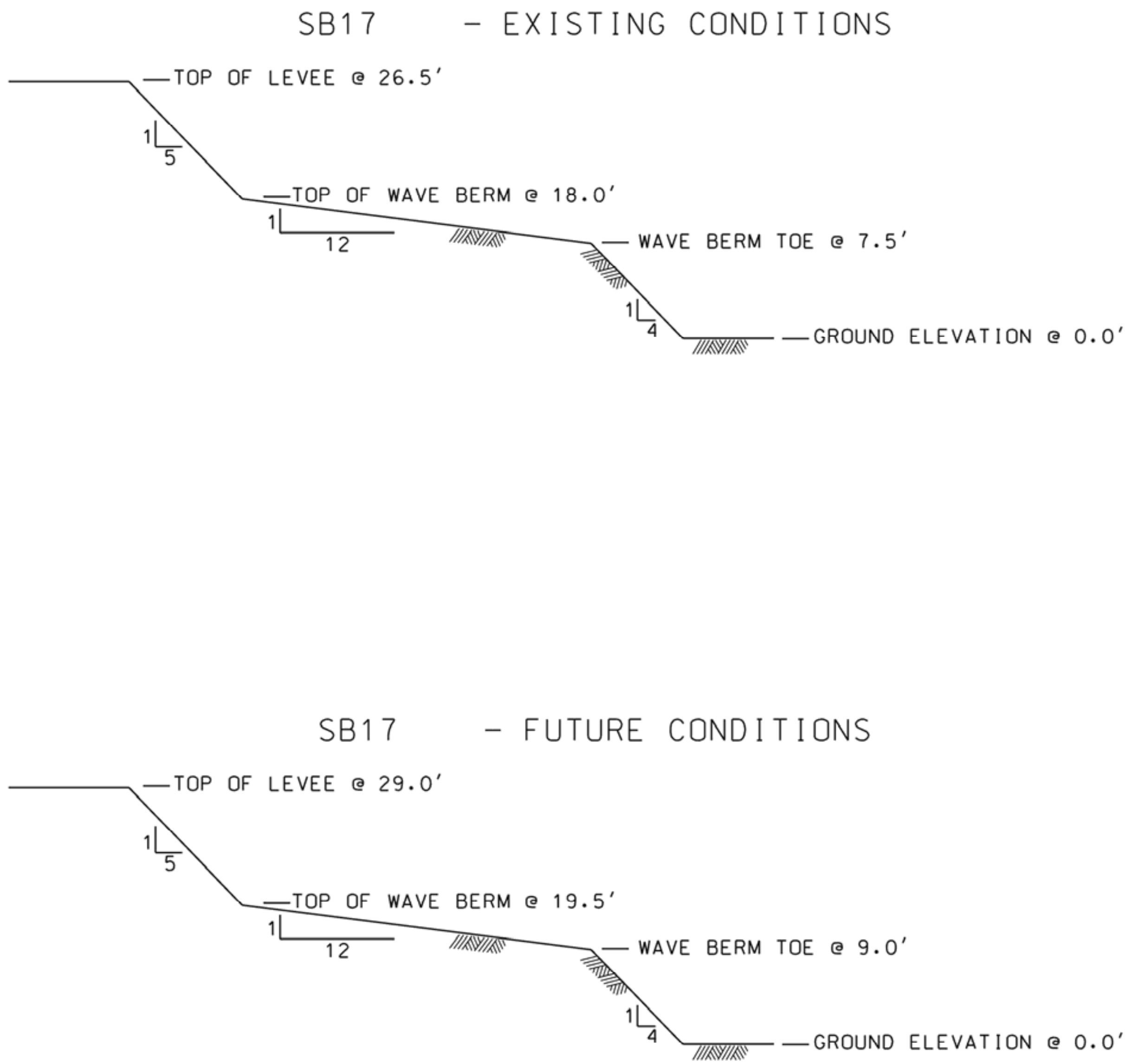


Figure 61 – Typical Levee Design Cross-Section SB17 for existing (upper panel) and future conditions (lower panel).

3.10.4 Wave Forces

Wave forces were computed for the structures along the St. Bernard segment with the Goda method, using future conditions. The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics. The following tables summarize the resulting wave forces. Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

St Bernard Sections Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
SB19	Bayou Dupre Contron Structure	13.7	299.5	7.9	13.7	299.5	7.9
SB20	St. Mary Pump Station #8	15.7	218.2	13.9	15.7	218.2	13.9

Table 50 – Waves Forces for St. Bernard Segments (50% values)

St Bernard Sections Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
SB19	Bayou Dupre Contron Structure	19.2	421.6	8.8	19.2	421.6	8.8
SB20	St. Mary Pump Station #8	19.6	279.5	14.3	19.6	279.5	14.3

Table 51 – Waves Forces for St. Bernard Segments (90% values)

3.10.5 Resiliency

The designs for St. Bernard Parish were examined for resiliency by also computing the overtopping rate for the 0.2 percent event for each design. The water level and overtopping rate was determined for the 50% assurance during the 0.2% event. The results are presented in Table 52. For all sections, the 0.2% surge elevation remains below the top of the flood defense. However, the overtopping rate can be quite significant over the levee during a 0.2% event, e.g. SB13, SB15, SB16, SB17 have an overtopping rate of 1 - 2 cfs/ft per ft (best estimates).

St Bernard Parish Sections Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
SB11	MRGO levee	Levee	Existing	29.0	22.1	0.374
SB11	MRGO levee	Levee	Future	31.5	23.6	0.322
SB12	MRGO levee	Levee	Existing	27.5	21.1	0.163
SB12	MRGO levee	Levee	Future	30.0	22.6	0.150
SB13	MRGO levee	Levee	Existing	26.5	20.2	2.355
SB13	MRGO levee	Levee	Future	29.0	21.7	2.284
SB15	MRGO levee	Levee	Existing	26.5	19.9	1.842
SB15	MRGO levee	Levee	Future	29.0	21.4	1.689
SB16	Caernarvon levee	Levee	Existing	26.5	21.3	1.319
SB16	Caernarvon levee	Levee	Future	29.0	22.8	0.920
SB17	Caernarvon levee	Levee	Existing	26.5	22.1	0.778
SB17	Caernarvon levee	Levee	Future	29.0	23.6	1.028
SB19	Bayou Dupre Control structure	Structure/Wall	Future	31.0	21.0	0.112
SB20	St Mary Pump Station (PS#8)	Structure/Wall	Future	30.5	21.9	0.253

Table 52 – Resiliency for St. Bernard Parish Segments

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4 West Bank and Vicinity

4.1 General

The West Bank and Vicinity portion of the Hurricane Protection Project extends from a point on the west bank of the Mississippi River near south Kenner and the western limits of Jefferson Parish eastward to a point on the Mississippi River near Oakville. The West Bank and Vicinity is divided into three areas: the Lake Cataouatche area, the Westwego to Harvey area, and the East of Harvey canal area. The West Bank and Vicinity area is shown in Figure 62.

The design elevations at the West Bank are dominated by the surge levels. The wave action is generally low, especially in the narrow canals. The 1% surge elevations range from 6.5 ft in the Lake Cataouatche area to 7.3 ft in the East of Harvey area. It should be noted that the number of representative output points from ADCIRC at the West Bank was relatively low. However, the 1% surge levels at the West Bank appear to be realistic compared with earlier findings and are therefore applied herein.

The 1% wave characteristics just in front of the levee ranged from: significant wave height around 2 - 3 ft and peak period 3 to 4 seconds for existing conditions. Notice that the wave characteristics in this area appear to be relatively low compared with what one may expect during these wind speeds. Although it is recognized that the waves in this area are probably reduced by the marsh area in the south, further research is recommended into the accuracy of these wave characteristics. For the time being, these wave characteristics are the best estimates at hand and are therefore applied herein.

This chapter discusses the design elevations for the entire West Bank. This area is split into several logical sub areas (with the section numbers, see Figure 62):

- 4.2: Lake Cataouatche Reach
- 4.3: Westwego to Harvey Canal Reach
- 4.4: East of Harvey Canal Reach

Each paragraph presents the 1% hydraulic boundary conditions, 1% the design elevations, the wave forces at the structures and the resiliency analysis for the 0.2% event.

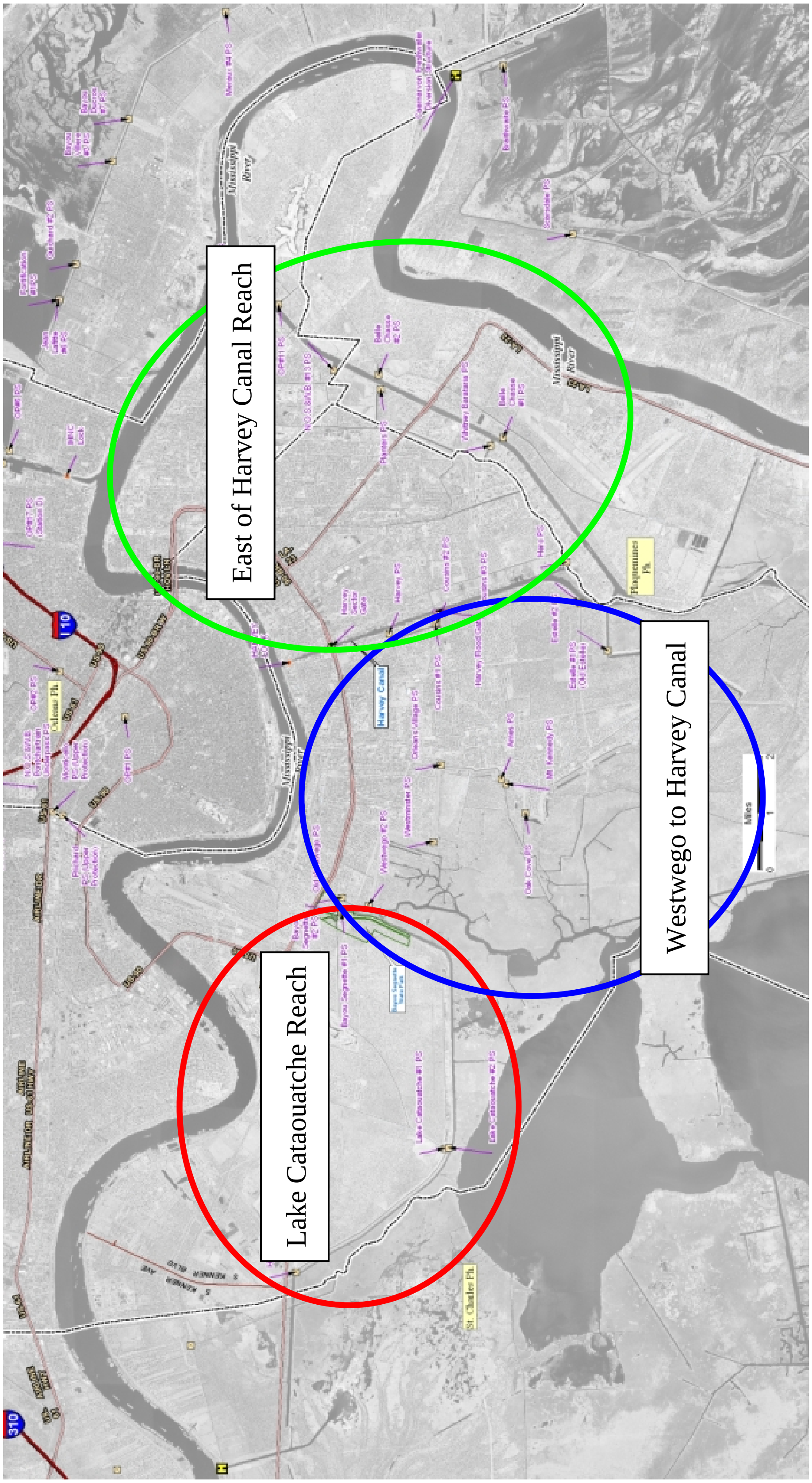


Figure 62 – Levees, Floodwalls and Pump Stations in the West Bank and Vicinity Area

4.2 *Lake Cataouatche Reach*

4.2.1 General

The Lake Cataouatche Area was divided into three main hydraulic reaches (Figure 63):

- WB31, which extends from the Mississippi River near Kenner to US Highway 90 (US90)
- WB01 which extends from US90 to the Bayou Segnette State Park,
- WB43 which extends from Bayou Segnette State Park to the Bayou Segnette pump station.

Storm surges are reduced by US90, as documented in the *Westwego to Harvey Canal, Louisiana Hurricane Protection Project, Lake Cataouatche Area, Post Authorization Change Report*, dated December 1996, so the surge elevation for segment WB31 are less than those for WB01 and WB43.

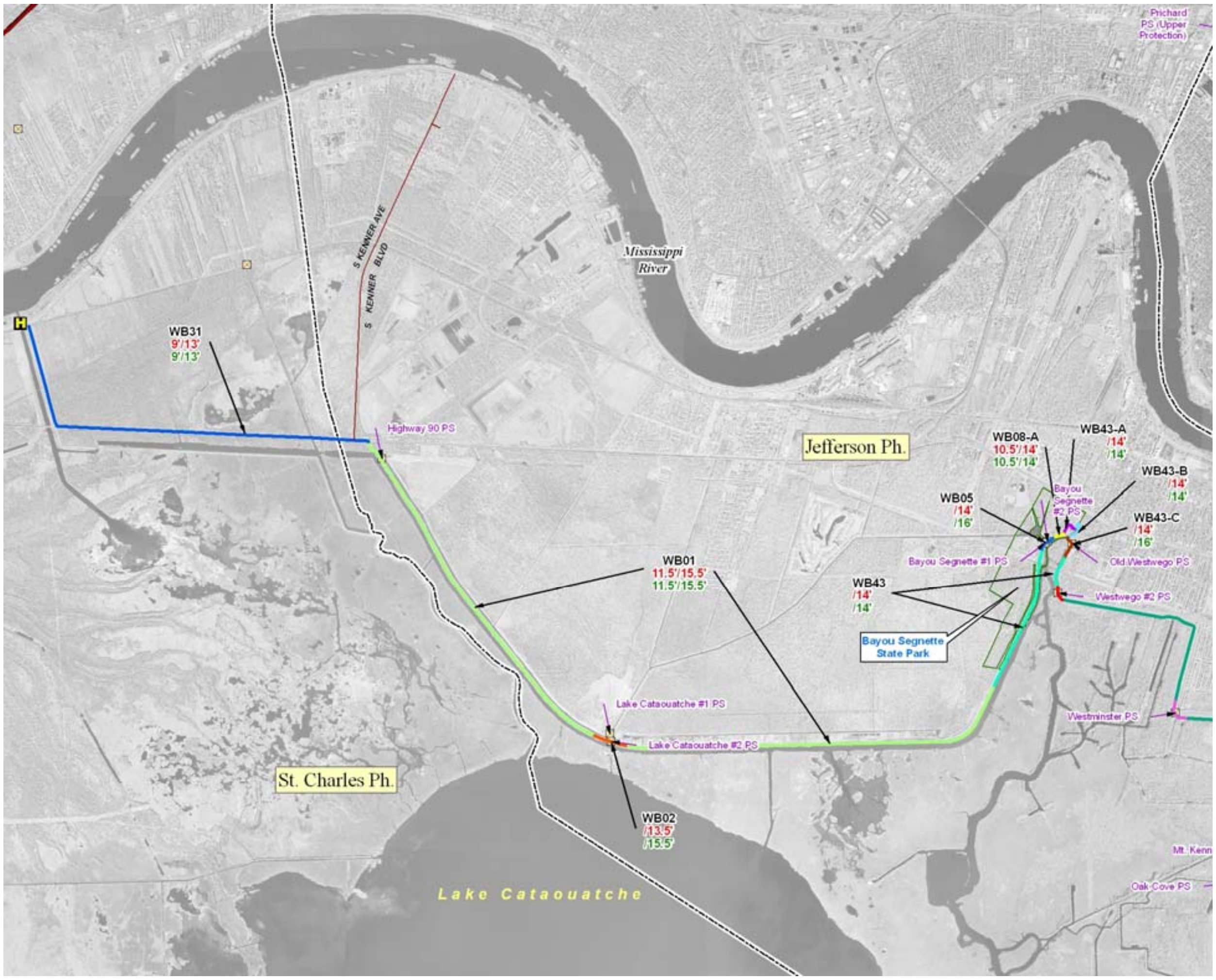


Figure 63 – Levees, Floodwalls and Pump Stations in the West Bank (Lake Cataouatche Reach)

There are currently no constructed levees or floodwalls in segment WB31, from the Mississippi River to US90, but three different flood protection alternatives that are currently being evaluated fall within, or partially within this reach. The alternatives follow different alignments, but all three would extend from to near Kenner to the northern end of the existing Lake Cataouatche levees at US90. The current levee alignment that has been evaluated in this report is shown in Figure 63. Other alternative alignments will be discussed in a separate report.

The existing levee segment WB01 extends from US90 to Bayou Segnette State Park. It trends to the southeast from US90, then due east, and then bends northward (Figure 63). The pump station outlet consists of pipes over the existing levee. Future plans are to abandon this station and reroute drainage. The pump stations (Lake Cataouatche Pump Station 1 and 2) will require a vertical wall at the outlet (WB02).

After the northward bend in segment WB01, this flood protection system changes into a floodwall (Figure 63). This so-called segment WB43 extends from Bayou Segnette State Park to the Bayou Segnette Pump Station, and consists of an existing floodwall. It ends at the Bayou Segnette Pump Station which is considered as a separate floodwall section (WB05).

4.2.2 Hydraulic Boundary Conditions

The hydraulic design characteristics used for the Lake Cataouatche reach are listed in Table 53. The existing conditions are based on the JPM-OS method using the results from ADCIRC and STWAVE. Output points with the highest values for the segments were selected, but the variation in the hydraulic conditions is small. The future condition design criteria were derived by adding 2.0 ft to the surge elevations, and adding 1.0 ft to the significant wave height. The wave period is increased in such a way that the wave steepness remains constant. For more information, see Chapter 2.

An average bottom elevation of +1.0 ft. was assumed for ground elevations in front of the levees to determine if the wave heights would be depth limited. A wave height of 40 percent of the design water depth was used as the depth-limiting criteria. The design wave heights for this reach were all less than 40 percent of the design water depth, therefore they were not reduced.

Westbank Sections (Lake Cataouache Reach) 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
WB31	Mississippi River to US90 Levees	Levee	Existing	6.5	0.7	1.6	0.2	5.4	1.1
WB31	Mississippi River to US90 Levees	Levee	Future	8.5	0.7	2.6	0.2	6.9	1.1
WB01	US Highway 90 to the Bayou Segnette State Park	Levee	Existing	6.5	0.7	2.1	0.2	5.5	1.1
WB01	US Highway 90 to the Bayou Segnette State Park	Levee	Future	8.5	0.7	3.1	0.2	6.7	1.1
WB02	Lake Cataouatche Pump Station 1 and 2	Structure/Wall	Future	8.5	0.7	3.1	0.2	6.7	1.1
WB43	Bayou Segnette State Park Floodwall	Structure/Wall	Future	8.5	0.7	2.4	0.1	5.6	0.9
WB05	Bayou Segnette Pump Station 1 and 2	Structure/Wall	Future	8.5	0.7	2.4	0.1	5.6	0.9

Table 53 – Lake Cataouatche Segments - 1% Hydraulic Boundary Conditions

4.2.3 Project Design Heights

The resulting design elevations are shown in Table 54. The levee sections are designed for both existing and future conditions. Note that the floodwalls and pump stations are only evaluated for future conditions, because these are hard structures. Both floodwalls at the pump stations (WB02 and WB05) include structural superiority of 2ft.

Westbank Sections (Lake Cataouache Reach) 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
WB31	Mississippi River to US90 Levees	Levee	Existing	5.5	9.0	0.002	0.044
WB31	Mississippi River to US90 Levees	Levee	Future	7.5	13.0	0.003	0.030
WB01	US Highway 90 to the Bayou Segnette State Park	Levee	Existing	6.5	11.5	0.003	0.024
WB01	US Highway 90 to the Bayou Segnette State Park	Levee	Future	8.5	15.5	0.006	0.034
WB02	Lake Cataouatche Pump Station 1 and 2	Structure/Wall	Future	8.5	15.5	0.001	0.003
WB43	Bayou Segnette State Park Floodwall	Structure/Wall	Future	8.5	14.0	0.000	0.002
WB05	Bayou Segnette Pump Station 1 and 2	Structure/Wall	Future	8.5	16.0	0.000	0.000

Table 54 – Lake Cataouatche Segments – 1% Design Information

Proposed designs for the two reaches are shown in Figure 64 and Figure 65 below for existing and future conditions for both levee sections WB01 and WB31. The design for levees in the Lake Cataouatche area has steep slopes near the crest. A wave berm was included to reduce the wave overtopping. The wave berm and the crest must be elevated to meet the design criteria for future conditions.

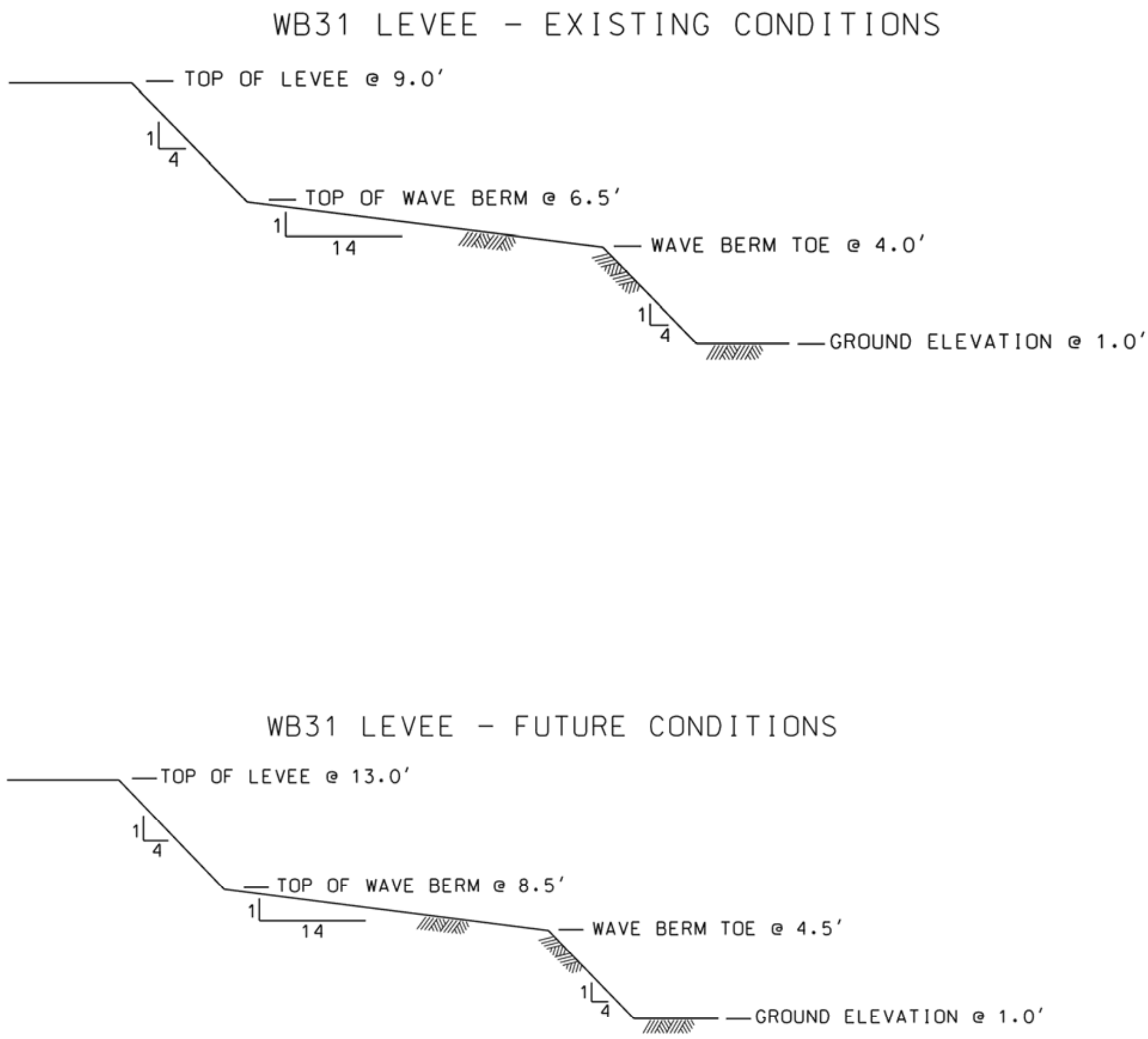


Figure 64 – Typical Design Cross Section for WB31 – Mississippi River to US90 Levees for existing (upper panel) and future conditions (lower panel)

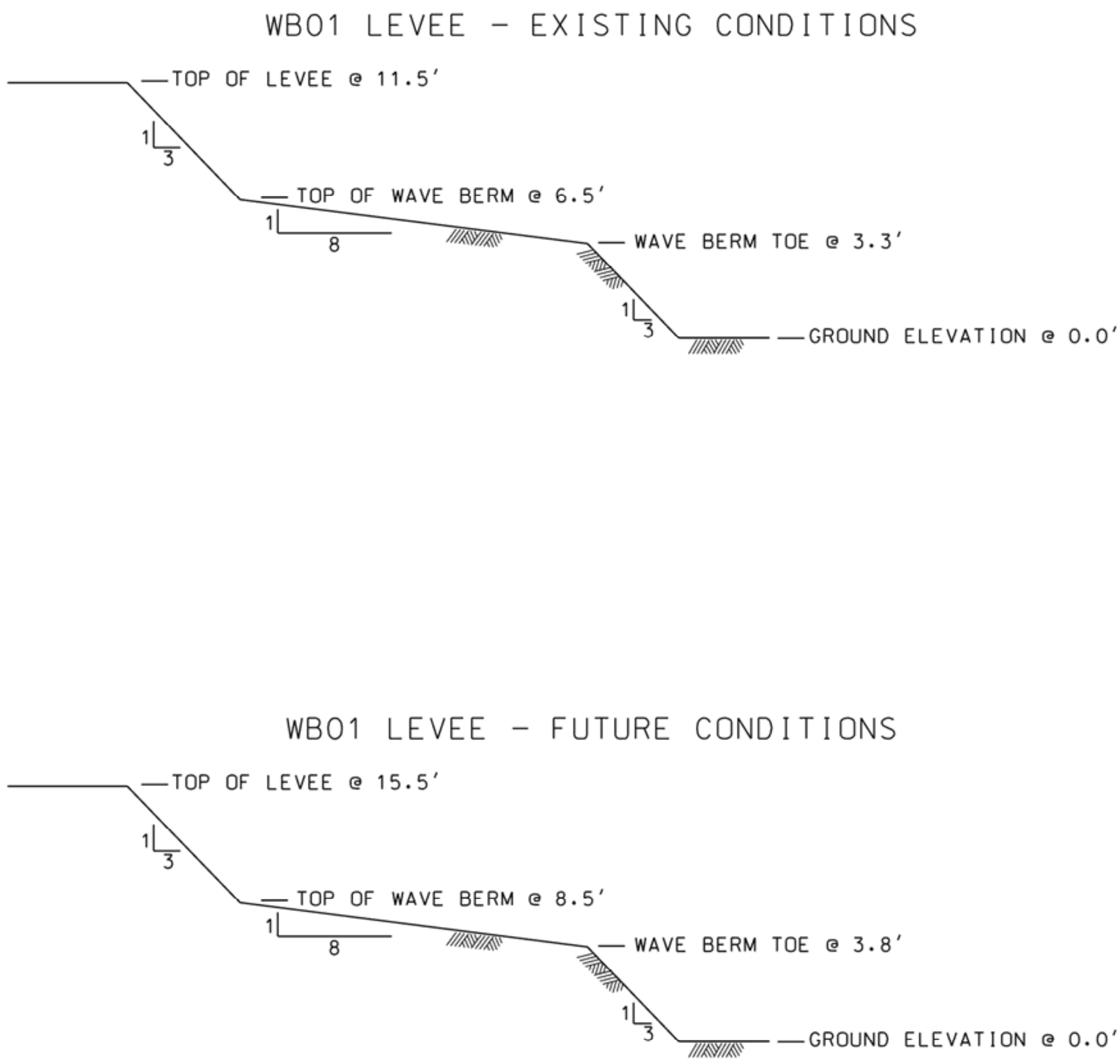


Figure 65 – Typical Design Cross Section for WB01 – US90 to Bayou Segnette State Park Levees for existing (upper panel) and future conditions (lower panel)

4.2.4 Wave Forces

Wave forces were computed for the structures along the Lake Cataouatche Reach with the Goda method, using future conditions. The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics. The following tables summarize the resulting wave forces. Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

West Bank - Lake Cataouache Sections Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
WB02	Lake Cataouatche Pump Station 1 and 2	5.4	40.9	6.6	5.4	40.9	6.6
WB05	Bayou Segnette Pump Station 1 and 2	2.9	20.1	6.9	4.1	28.4	6.9
WB43	Bayou Segnette State Park Floodwall	2.2	7.8	9.5	2.2	7.8	9.5

Table 55 – Waves Forces for Lake Cataouatche Segments (50% values)

West Bank - Lake Cataouache Sections Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
WB02	Lake Cataouatche Pump Station 1 and 2	4.4	31.8	6.3	4.4	32.2	6.3
WB05	Bayou Segnette Pump Station 1 and 2	2.4	16.1	6.7	4.1	27.4	6.7
WB43	Bayou Segnette State Park Floodwall	2.1	6.7	9.2	2.1	6.7	9.2

Table 56 – Waves Forces for Lake Cataouatche Segments (90% values)

4.2.5 Resiliency

The designs for the Lake Cataouatche Reach were examined for resiliency by also computing the overtopping rate for the 0.2 percent event for each design. The water level and overtopping rate was determined for the 50% assurance during the 0.2% event. The results are presented in Table 57. For all sections, the 0.2% surge elevation remains below the top of the flood defense.

However, the overtopping rate can be quite significant over some of the levee sections during a 0.2% event (e.g. WB31).

Westbank Sections (Lake Cataouache Reach) Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
WB31	Mississippi River to US90 Levees	Levee	Existing	9.0	8.9	1.803
WB31	Mississippi River to US90 Levees	Levee	Future	13.0	10.9	0.584
WB01	US Highway 90 to the Bayou Segnette State Park	Levee	Existing	11.5	9.0	0.575
WB01	US Highway 90 to the Bayou Segnette State Park	Levee	Future	15.5	11.0	0.343
WB02	Lake Cataouatche Pump Station 1 and 2	Structure/Wall	Future	15.5	11.0	0.072
WB43	Bayou Segnette State Park Floodwall	Structure/Wall	Future	14.0	11.1	0.141
WB05	Bayou Segnette Pump Station 1 and 2	Structure/Wall	Future	16.0	11.1	0.017

Table 57 – Resiliency for Lake Cataouatche Segments

4.3 Westwego to Harvey Canal Reach

4.3.1 General

This portion of the West Bank and Vicinity Hurricane Project extends from Bayou Segnette to the Harvey Canal. There are levee and floodwall segments and several pumping stations located within this reach. Figure 66 presents an overview of the various sections and the design elevations.

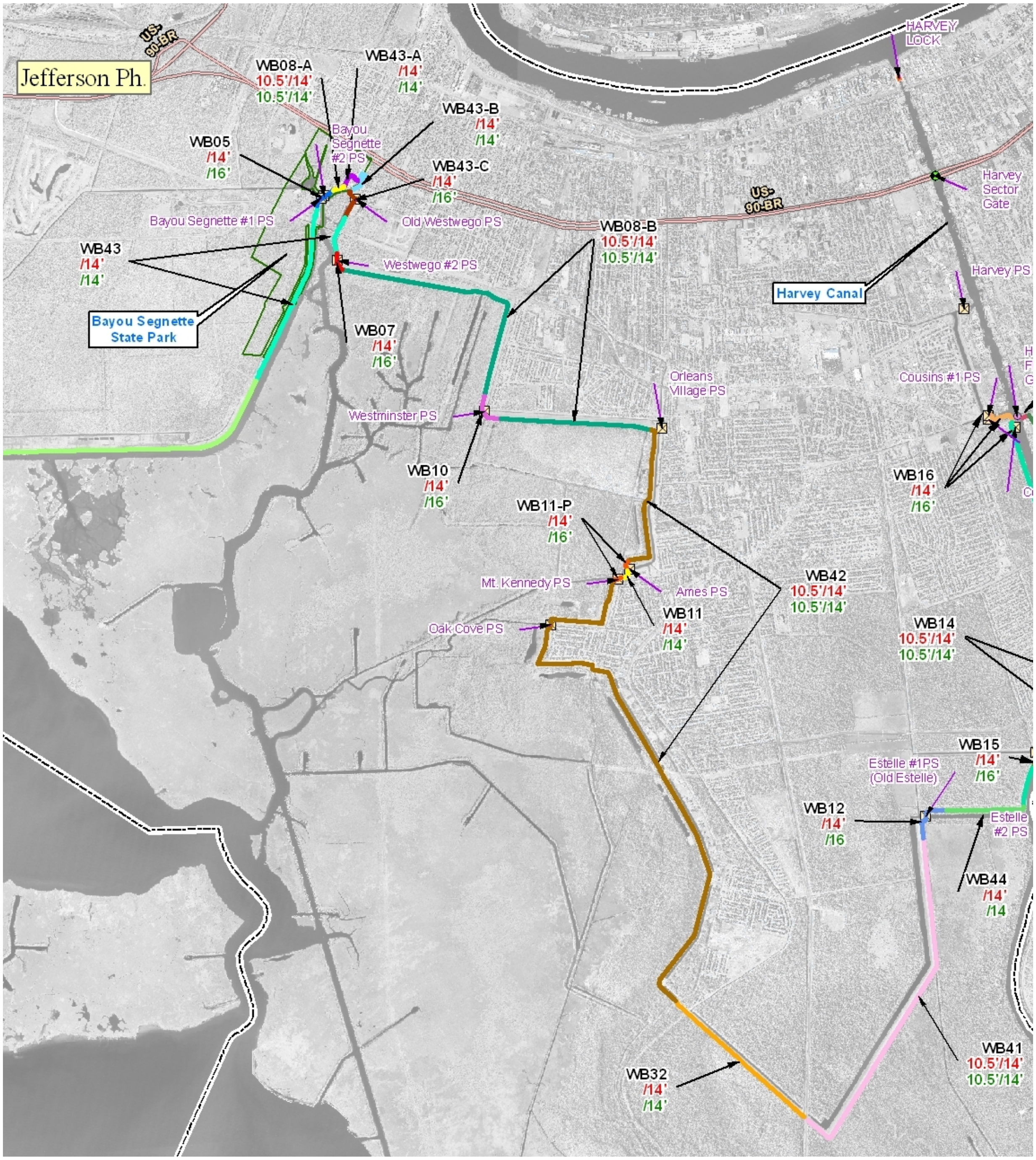


Figure 66 – Levees, Floodwalls and Pump Stations in the West Bank (Westwego to Harvey Canal Reach)

4.3.2 Hydraulic Boundary Conditions

The design characteristics of the sections between Westwego and Harvey Canal are listed in Table 58 below. The variation in hydraulic conditions was small throughout the reach. The future conditions were derived by adding 2.0 ft to the surge elevations, and adding 1.0 ft to the significant wave height. The wave period is increased in such a way that the wave steepness remains constant.

Westbank Sections (Westwego to Harvey Canal Reach) 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
WB08-a	Segnette Pump Station to Company Canal Floodwall	Levee	Existing	6.5	0.7	1.4	0.1	4.3	0.9
WB08-a	Segnette Pump Station to Company Canal Floodwall	Levee	Future	8.5	0.7	2.4	0.1	5.6	0.9
WB08-b	New Westwego Pump Station to Orleans Village Levee	Levee	Existing	6.5	0.7	1.4	0.1	4.3	0.9
WB08-b	New Westwego Pump Station to Orleans Village Levee	Levee	Future	8.5	0.7	2.4	0.1	5.6	0.9
WB43-a	Segnette Pump Station to Company Canal Floodwall	Structure/Wall	Future	8.5	0.7	2.4	0.1	5.6	0.9
WB43-b	Company Canal & Westwego Floodwall	Structure/Wall	Future	8.5	0.7	2.4	0.1	5.6	0.9
WB43-c	Old Westwego Pump Station	Structure/Wall	Future	8.5	0.7	2.4	0.1	5.6	0.9
WB07	New Westwego Pump Station	Structure/Wall	Future	8.5	0.7	2.4	0.1	5.6	0.9
WB10	Westminster Pump Station	Structure/Wall	Future	8.5	0.7	2.4	0.1	5.6	0.9
WB11	Ames to Kennedy Floodwall	Structure/Wall	Future	9.3	0.9	2.3	0.1	4.9	0.7
WB11-P	Ames & Kennedy Pump Station	Structure/Wall	Future	9.3	0.9	2.3	0.1	4.9	0.7
WB42	Orleans Village to Ames Pump Station Levee	Levee	Existing	7.3	0.9	1.3	0.1	3.7	0.7
WB42	Orleans Village to Ames Pump Station Levee	Levee	Future	9.3	0.9	2.3	0.1	4.9	0.7
WB32	Highway 45 to Highway 3134	Structure/Wall	Future	9.3	0.9	2.3	0.1	4.9	0.7
WB41	Highway 3134 to Old Estelle Pump Station Levee	Levee	Existing	7.3	0.9	1.3	0.1	3.7	0.7
WB41	Highway 3134 to Old Estelle Pump Station Levee	Levee	Future	9.3	0.9	2.3	0.1	4.9	0.7
WB12	Old Estelle Pump Station	Structure/Wall	Future	9.3	0.9	2.3	0.1	4.9	0.7
WB44	Old Estelle to Robinson Point	Structure/Wall	Future	9.3	0.9	2.3	0.1	4.9	0.7

Table 58 – Westwego to Harvey Segments - 1% Hydraulic Boundary Conditions

An average bottom elevation of +1.0 ft. was assumed for ground elevations in front of the levees to determine if the wave heights would be depth limited. A wave height of 40 percent of the design water depth was used as the depth-limiting criteria. The design wave heights were not reduced for any of the segments within this reach, based on this criteria.

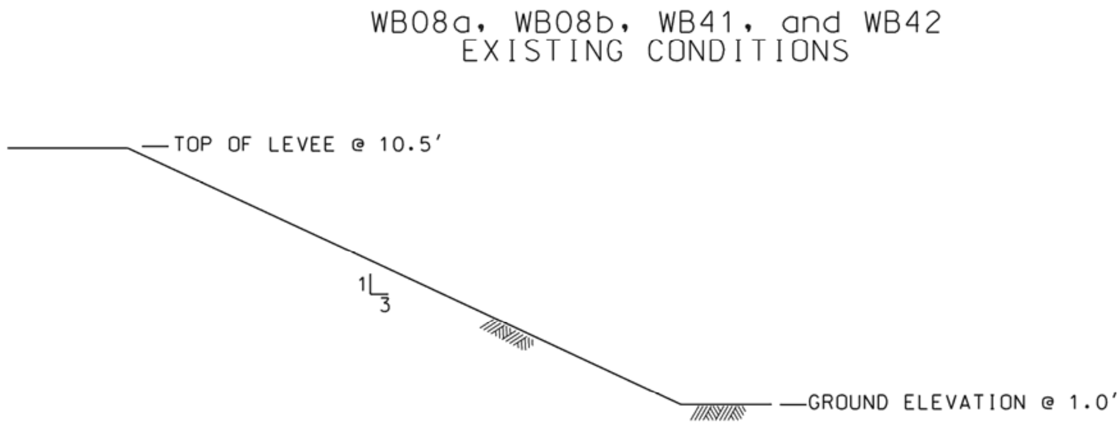
4.3.3 Project Design Heights

The design characteristics along the Westwego to Harvey Reach are summarized in Table 59. The levee sections are designed for both existing and future conditions. Note that the floodwalls and pump stations are only evaluated for future conditions, because these are hard structures. The Old and New Westwego Pump Station (WB43C and WB07), Ames and Kennedy Pump Station (WB11-P), Old Estelle Pump Station (WB12) and Westminster Pump Station (WB10) include structural superiority of 2ft.

Westbank Sections (Westwego to Harvey Canal Reach) 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
WB08-a	Segnette Pump Station to Company Canal Floodwall	Levee	Existing	5.5	10.5	0.001	0.010
WB08-a	Segnette Pump Station to Company Canal Floodwall	Levee	Future	7.5	14.0	0.008	0.037
WB08-b	New Westwego Pump Station to Orleans Village Levee	Levee	Existing	5.5	10.5	0.001	0.010
WB08-b	New Westwego Pump Station to Orleans Village Levee	Levee	Future	7.5	14.0	0.008	0.035
WB43-a	Segnette Pump Station to Company Canal Floodwall	Structure/Wall	Future	8.5	14.0	0.000	0.003
WB43-b	Company Canal & Westwego Floodwall	Structure/Wall	Future	8.5	14.0	0.000	0.002
WB43-c	Old Westwego Pump Station	Structure/Wall	Future	8.5	16.0	0.000	0.000
WB07	New Westwego Pump Station	Structure/Wall	Future	8.5	16.0	0.000	0.000
WB10	Westminster Pump Station	Structure/Wall	Future	8.5	16.0	0.000	0.000
WB11	Ames to Kennedy Floodwall	Structure/Wall	Future	9.3	14.0	0.001	0.008
WB11-P	Ames & Kennedy Pump Station	Structure/Wall	Future	9.3	16.0	0.000	0.000
WB42	Orleans Village to Ames Pump Station Levee	Levee	Existing	7.3	10.5	0.003	0.035
WB42	Orleans Village to Ames Pump Station Levee	Levee	Future	9.3	14.0	0.010	0.063
WB32	Highway 45 to Highway 3134	Structure/Wall	Future	9.3	14.0	0.001	0.008
WB41	Highway 3134 to Old Estelle Pump Station Levee	Levee	Existing	7.3	10.5	0.003	0.034
WB41	Highway 3134 to Old Estelle Pump Station Levee	Levee	Future	9.3	14.0	0.010	0.061
WB12	Old Estelle Pump Station	Structure/Wall	Future	9.3	16.0	0.000	0.000
WB44	Old Estelle to Robinson Point	Structure/Wall	Future	9.3	14.0	0.001	0.008

Table 59 – Westwego to Harvey Segments – 1% Design Information

The levee designs are all simple levees with straight slopes and no wave berms. Figure 67 shows typical proposed design sections for the Westwego to Harvey levees. The levee crest elevation for existing conditions is 10.5ft with a 1:3 slope. The levee crest elevation and the slope must be adapted for future conditions to meet the design criteria. The design elevation for future conditions is 14ft with a 1:4 slope.



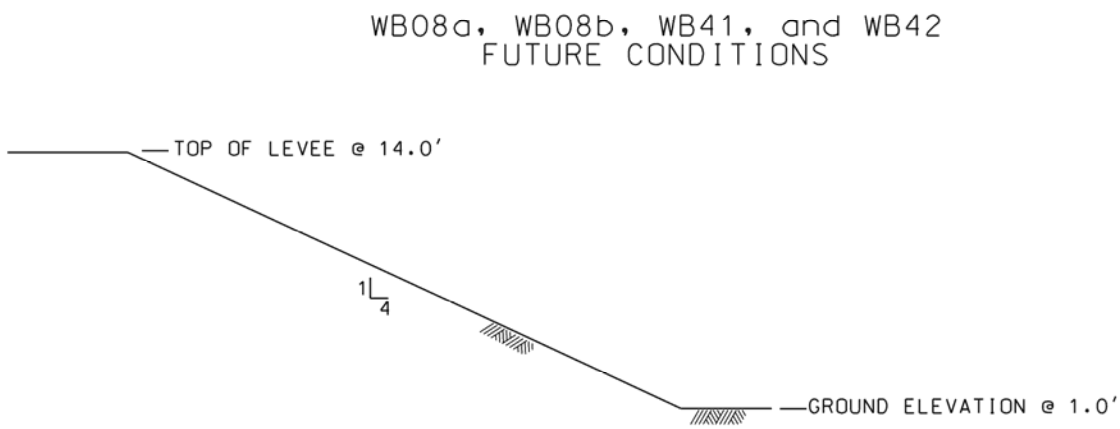


Figure 67 – Typical Design Cross Section for Westwego to Harvey Levees (WB08A, WB08B, WB41 and WB42) for existing (upper panel) and future conditions (lower panel)

There are several floodwall segments and pump stations within this reach. Fronting walls were designed for the Old and New Westwego Pump Station, the Westminster Pump Station, the

Ames and Kennedy Pump Stations, the Old Estelle Pump Station and the New Estelle Pump Station. For all of them, the design elevation includes 2ft of structural superiority.

4.3.4 Wave Forces

Wave forces were computed for the floodwalls, pump station fronting walls and navigation gates within the Westwego to Harvey Canal segment with the Goda method, using future conditions. The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics. The following tables summarize the resulting wave forces. Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

West Bank - Westwego to Harvey Canal Sections Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
WB07	New Westwego Pump Station	2.4	16.2	6.5	2.4	16.2	6.5
WB08-a	Segnette Pump Station to Company Canal Floodwall	2.1	6.7	9.2	2.1	6.7	9.2
WB08-b	New Westwego Pump Station to Orleans Village Levee	2.1	6.7	9.2	2.1	6.7	9.2
WB10	Westminster Pump Station	2.4	16.2	6.5	2.4	16.2	6.5
WB11	Ames to Kennedy Floodwall	1.7	5.6	9.4	1.7	5.6	9.4
WB11-P1	Ames Pump Station	2.4	16.4	6.9	3.4	23.9	6.9
WB11-P2	Kennedy Pump Station	2.3	14.5	7.4	3.8	25.0	7.4
WB12	Old Estelle Pump Station	3.4	56.9	0.7	3.4	56.9	0.7
WB32	Highway 45 to Highway 3134	1.8	10.9	7.1	1.8	11.1	7.1
WB43-a	Segnette Pump Station to Company Canal Floodwall	2.1	6.7	9.2	2.1	6.7	9.2
WB43-b	Company Canal & Westwego Floodwall	2.1	6.7	9.2	2.1	6.7	9.2
WB43-c	Old Westwego Pump Station	2.7	23.2	4.5	2.7	23.2	4.5

Table 60 – Waves Forces for Westwego to Harvey Canal Segments (50% values)

West Bank - Westwego to Harvey Canal Sections Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
WB07	New Westwego Pump Station	3.0	20.7	6.7	3.0	20.7	6.7
WB08-a	Segnette Pump Station to Company Canal Floodwall	2.2	7.8	9.5	2.2	7.8	9.5
WB08-b	New Westwego Pump Station to Orleans Village Levee	2.2	7.8	9.5	2.2	7.8	9.5
WB10	Westminster Pump Station	2.9	20.6	6.7	2.9	20.6	6.7
WB11	Ames to Kennedy Floodwall	1.9	7.0	9.7	1.9	7.0	9.7
WB11-P1	Ames Pump Station	3.0	22.0	7.4	3.7	25.6	7.4
WB11-P2	Kennedy Pump Station	2.8	19.4	7.9	4.0	25.8	7.9
WB12	Old Estelle Pump Station	4.5	73.7	1.5	4.5	73.7	1.5
WB32	Highway 45 to Highway 3134	2.2	14.2	7.4	2.2	14.4	7.4
WB43-a	Segnette Pump Station to Company Canal Floodwall	2.2	7.8	9.5	2.2	7.8	9.5
WB43-b	Company Canal & Westwego Floodwall	2.2	7.8	9.5	2.2	7.8	9.5
WB43-c	Old Westwego Pump Station	3.5	30.3	4.7	3.5	30.3	4.7

Table 61 – Waves Forces for Westwego to Harvey Canal Segments (90% values)

4.3.5 Resiliency

The designs for Westwego to Harvey Canal Reach were examined for resiliency by also computing the overtopping rate for the 0.2 percent event for each design. The water level and overtopping rate was determined for the 50% assurance during the 0.2% event. The results are presented in Table 62. For all sections, the 0.2% surge elevation remains below the top of the flood defense. The overtopping rate can be quite significant over the levee during a 0.2% event. The levee sections WB41 and WB42 have an overtopping rate of 1 - 2 cfs/ft per ft (best estimates).

Westbank Sections (Westwego to Harvey Canal Reach) Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
WB08-a	Segnette Pump Station to Company Canal Floodwall	Levee	Existing	10.5	9.1	0.945
WB08-a	Segnette Pump Station to Company Canal Floodwall	Levee	Future	14.0	11.1	0.727
WB08-b	New Westwego Pump Station to Orleans Village Levee	Levee	Existing	10.5	9.1	0.955
WB08-b	New Westwego Pump Station to Orleans Village Levee	Levee	Future	14.0	11.1	0.729
WB43-a	Segnette Pump Station to Company Canal Floodwall	Structure/Wall	Future	14.0	11.1	0.138
WB43-b	Company Canal & Westwego Floodwall	Structure/Wall	Future	14.0	11.1	0.140
WB43-c	Old Westwego Pump Station	Structure/Wall	Future	16.0	11.1	0.016
WB07	New Westwego Pump Station	Structure/Wall	Future	16.0	11.1	0.017
WB10	Westminster Pump Station	Structure/Wall	Future	16.0	11.1	0.016
WB11	Ames to Kennedy Floodwall	Structure/Wall	Future	14.0	12.4	0.434
WB11-P	Ames & Kennedy Pump Station	Structure/Wall	Future	16.0	12.4	0.047
WB42	Orleans Village to Ames Pump Station Levee	Levee	Existing	10.5	10.4	2.076
WB42	Orleans Village to Ames Pump Station Levee	Levee	Future	14.0	12.4	1.237
WB32	Highway 45 to Highway 3134	Structure/Wall	Future	14.0	12.4	0.447
WB41	Highway 3134 to Old Estelle Pump Station Levee	Levee	Existing	10.5	10.4	2.075
WB41	Highway 3134 to Old Estelle Pump Station Levee	Levee	Future	14.0	12.4	1.265
WB12	Old Estelle Pump Station	Structure/Wall	Future	16.0	12.4	0.047
WB44	Old Estelle to Robinson Point	Structure/Wall	Future	14.0	12.4	0.429

Table 62 – Resiliency for Westwego to Harvey Canal Segments

4.4 East of Harvey Canal

4.4.1 General

This portion of the West Bank and Vicinity Hurricane Project is the area east of Harvey Canal. The sections lie along Harvey Canal, Hero Canal and Algiers Canal. There are levee and floodwall segments and several pumping stations located within this reach. Figure 68 presents an overview of the various sections and the design elevations. The hydraulic boundary conditions, the design elevations, the wave forces and the resiliency analysis are discussed below.

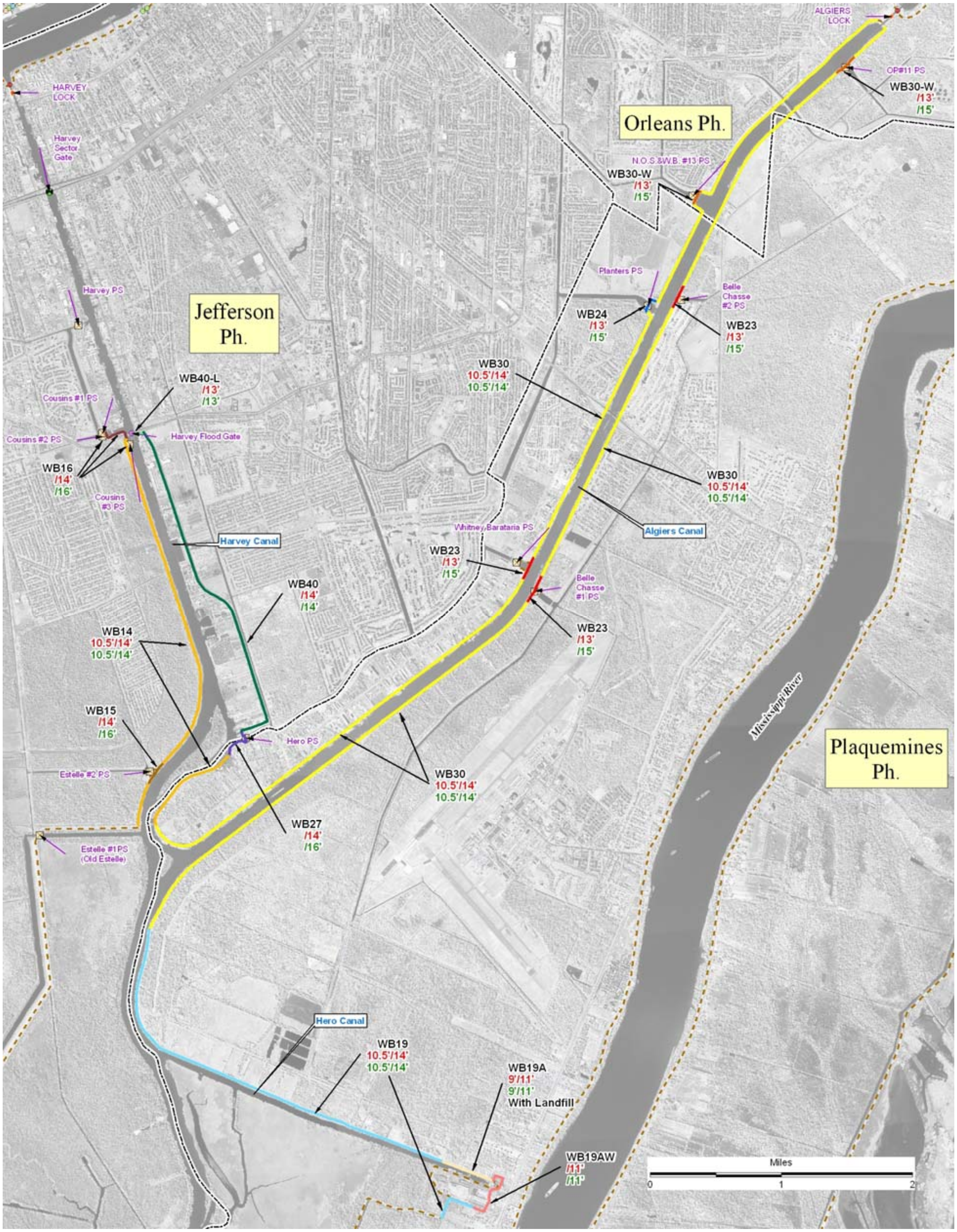


Figure 68 – Levees, Floodwalls and Pump Stations in the West Bank (East of Harvey Canal)

4.4.2 Hydraulic Boundary Conditions

The design characteristics of the sections east of Harvey Canal are listed in Table 63 below. The future conditions were derived by adding 2.0 ft to the surge elevations, and adding 1.0 ft to the significant wave height. The wave period is increased in such a way that the wave steepness remains constant.

Westbank Sections (East of Harvey Canal Reach) 1% Hydraulic boundary conditions									
				Surge level (ft)		Significant wave height (ft)		Peak period (s)	
Segment	Name	Type	Condition	Mean	Std	Mean	Std	Mean	Std
WB14	Robinson Pt. to Harvey Canal W. Levee	Levee	Existing	7.8	0.9	1.3	0.1	3.7	0.7
WB14	Robinson Pt. to Harvey Canal W. Levee	Levee	Future	9.8	0.9	2.3	0.1	4.9	0.7
WB15	New Estelle Pump Station	Structure/Wall	Future	9.8	0.9	2.3	0.1	4.9	0.7
WB40	Harvey Canal Floodwall	Structure/Wall	Future	9.8	0.9	2.3	0.1	4.9	0.7
WB16	Cousins Pump Station 1, 2 and 3 (on Harvey Canal)	Structure/Wall	Future	9.8	0.9	2.3	0.1	4.9	0.7
WB40-L	Sector Gate at Lapalco Overpass on Harvey Canal	Structure/Wall	Future	9.8	0.9	2.3	0.1	4.9	0.7
WB27	Hero Pump Station (on Harvey Canal)	Structure/Wall	Future	9.8	0.9	2.3	0.1	4.9	0.7
WB30	Algiers Canal - Hero Pump Station to Algiers Lock	Levee	Existing	7.8	0.9	1.3	0.1	3.7	0.7
WB30	Algiers Canal - Hero Pump Station to Algiers Lock	Levee	Future	9.8	0.9	2.3	0.1	4.9	0.7
WB23	Whitney Barataria and Belle Chase 1 and 2 Pump Stations	Structure/Wall	Future	9.8	0.9	2.3	0.1	4.9	0.7
WB24	Planters Pump Station	Structure/Wall	Future	9.8	0.9	2.3	0.1	4.9	0.7
WB30-W	NO SBW Pump Station 11 and 13	Structure/Wall	Future	9.8	0.9	2.3	0.1	4.9	0.7
WB19	Transition Point to Hero Canal to Oakville	Levee	Existing	7.3	0.9	1.3	0.1	3.7	0.7
WB19	Transition Point to Hero Canal to Oakville	Levee	Future	9.3	0.9	2.3	0.1	4.9	0.7
WB19-A	Hero Canal-Area Behind Landfill Berm	Levee	Existing	7.3	0.9	1.0	0.1	2.0	0.4
WB19-A	Hero Canal-Area Behind Landfill Berm	Levee	Future	9.3	0.9	1.0	0.1	2.0	0.4
WB19-AW	Hero Canal Floodwall behind Landfill Berm	Structure/Wall	Future	9.3	0.9	1.0	0.1	2.0	0.4

Table 63 – East of Harvey Canal Segments – 1% Hydraulic Boundary Conditions

The bottom elevation near the levees and floodwalls is generally around 0ft or less. Because the wave heights are small in these canals, the exact bottom elevation is not needed because the waves are not depth-limited. The design wave heights were not reduced for any of the segments within this reach, based on this criteria.

The specifics of the hydraulic boundary conditions for each canal reach are discussed below:

Harvey Canal
Along Harvey Canal, a levee was designed between Robinson Point to Harvey Canal (WB14). Furthermore, floodwalls along Harvey Canal were designed for the New Estelle Pump Station (WB15), Harvey Canal west and east side (WB40), Cousins Pump Stations 1,2 and 3 (WB16) and the Hero Pump Stations (WB27). Finally, a sector gate was designed for Harvey Canal, at the Lapalco Overpass (WB40-L). An average bottom elevation of -6.0 ft was assumed for the canal in front of the walls.

There are several pump stations that output into Harvey Canal. The impact of increased water volumes into these constricted areas on the surge elevations must be accounted for in the design heights of the protection system. An existing HEC-RAS model for an ongoing study, Donaldsonville to the Gulf feasibility study, was modified to include the pumping stations and the Harvey and Algiers Canals. The HEC-RAS model was run with a 100-year rainfall in the interior areas which are pumped into the Harvey and Algiers Canals. The 100-year surge elevation was used as a downstream boundary. Based on the HEC-RAS results, the surge elevations for the outpoint points used for the design of the structures within Harvey Canal were increased by 0.5 ft to account for the pumping into the canal, for both existing and future condition designs.

Algiers Canal
A levee is designed along Algiers Canal (WB30). This segment includes the following pump stations: N. O. Sewerage & Water Board (SWB) Pump Station #11 and #13, N.O. SWB Pump Station #13, Belle Chasse Pump Station #2, Belle Chasse Pump Station #1, and Planters Pump Station and Whitney-Barataria Pump Station (sections WB30-W, WB23, WB24).

There are several pump stations that output into the Algiers Canal. The impact of increased water volumes into these constricted areas on the surge elevations in the Canals must be accounted for in the design heights of the protection system. With a future design surge elevation of 9.3 ft and current pump efficiencies, stages in Algiers Canal increase by 0.5 ft. If the efficiencies increase such that the pumps can operate at full capacity, stages in the canal increase by 0.7 ft. The surge elevations for the outpoint points used for the design of the structures within Algiers Canal were increased by 0.5 ft to account for the pumping into the canal, for both existing and future condition designs.

Hero Canal
This segment (WB19) includes levees along the Hero Canal from a transition point approximately midway between Algiers Canal and Hero Canal easterly to the eastern end of the canal near Oakville, and a proposed floodwall near the end of the canal at Oakville (WB19W). A landfill with high perimeter berms is located south of the canal near the eastern end of the canal. The landfill berms would block waves for this area, so a levee and a floodwall design was provided (WB19A and WB19AW), assuming only a 1 ft and 2 s wave for the design criteria.

4.4.3 Project Design Heights

The designs for various sections along Harvey Canal, Algiers Canal and Hero Canal are summarized in Table 64 below.

Westbank Sections (East of Harvey Canal Reach) 1% Design heights							
						Overtopping rate	
Segment	Name	Type	Condition	Depth at toe (ft)	Height (ft)	q50 (cft/s per ft)	q90 (cft/s per ft)
WB14	Robinson Pt. to Harvey Canal W. Wall Levee	Levee	Existing	7.8	10.5	0.004	0.069
WB14	Robinson Pt. to Harvey Canal W. Wall Levee	Levee	Future	9.8	14.0	0.006	0.060
WB15	New Estelle Pump Station	Structure/Wall	Future	9.8	16.0	0.000	0.001
WB40	Harvey Canal Floodwall	Structure/Wall	Future	9.8	14.0	0.002	0.016
WB16	Cousins Pump Station 1, 2 and 3 (on Harvey Canal)	Structure/Wall	Future	9.8	16.0	0.000	0.001
WB40-L	Sector Gate at Lapalco Overpass on Harvey Canal	Structure/Wall	Future	9.8	13.0	0.011	0.073
WB27	Hero Pump Station (on Harvey Canal)	Structure/Wall	Future	9.8	16.0	0.000	0.001
WB30	Hero Pump Station to Algiers Canal Levee	Levee	Existing	7.8	10.5	0.004	0.068
WB30	Hero Pump Station to Algiers Canal Levee	Levee	Future	9.8	14.0	0.006	0.058
WB23	Whitney Barataria and Belle Chase Pump Stations	Structure/Wall	Future	9.8	15.0	0.000	0.004
WB24	Planters Pump Station	Structure/Wall	Future	9.8	15.0	0.000	0.004
WB30-W	NO SBW Pump Station 11	Structure/Wall	Future	9.8	15.0	0.000	0.004
WB19	Transition Point to Hero Canal to Oakville	Levee	Existing	7.3	10.5	0.001	0.024
WB19	Transition Point to Hero Canal to Oakville	Levee	Future	9.3	14.0	0.003	0.030
WB19-W	Hero Canal Floodwall	Structure/Wall	Future	9.3	13.0	0.005	0.033
WB19-A	Hero Canal-Area Behind Landfill Berm	Levee	Existing	7.3	9.0	0.000	0.078
WB19-A	Hero Canal-Area Behind Landfill Berm	Levee	Future	9.3	11.0	0.000	0.077
WB19-AW	Hero Canal Floodwall behind Landfill Berm	Structure/Wall	Future	9.3	11.0	0.001	0.067

Table 64 –East of Harvey Canal Segments – 1% Design Information

The levee designs along Harvey Canal (WB14), Algiers Canal (WB30) and Hero Canal (WB19) are all simple levees with 1V:4H slopes and no wave berms. Figure 69 shows a typical proposed design section for the levees. The levee crest elevation for existing conditions is 10.5ft with a 1:4 slope. The levee crest elevation and the slope must be adapted for future conditions to meet the design criteria. The design cross-section for future conditions is 14ft with a 1:5 slope.

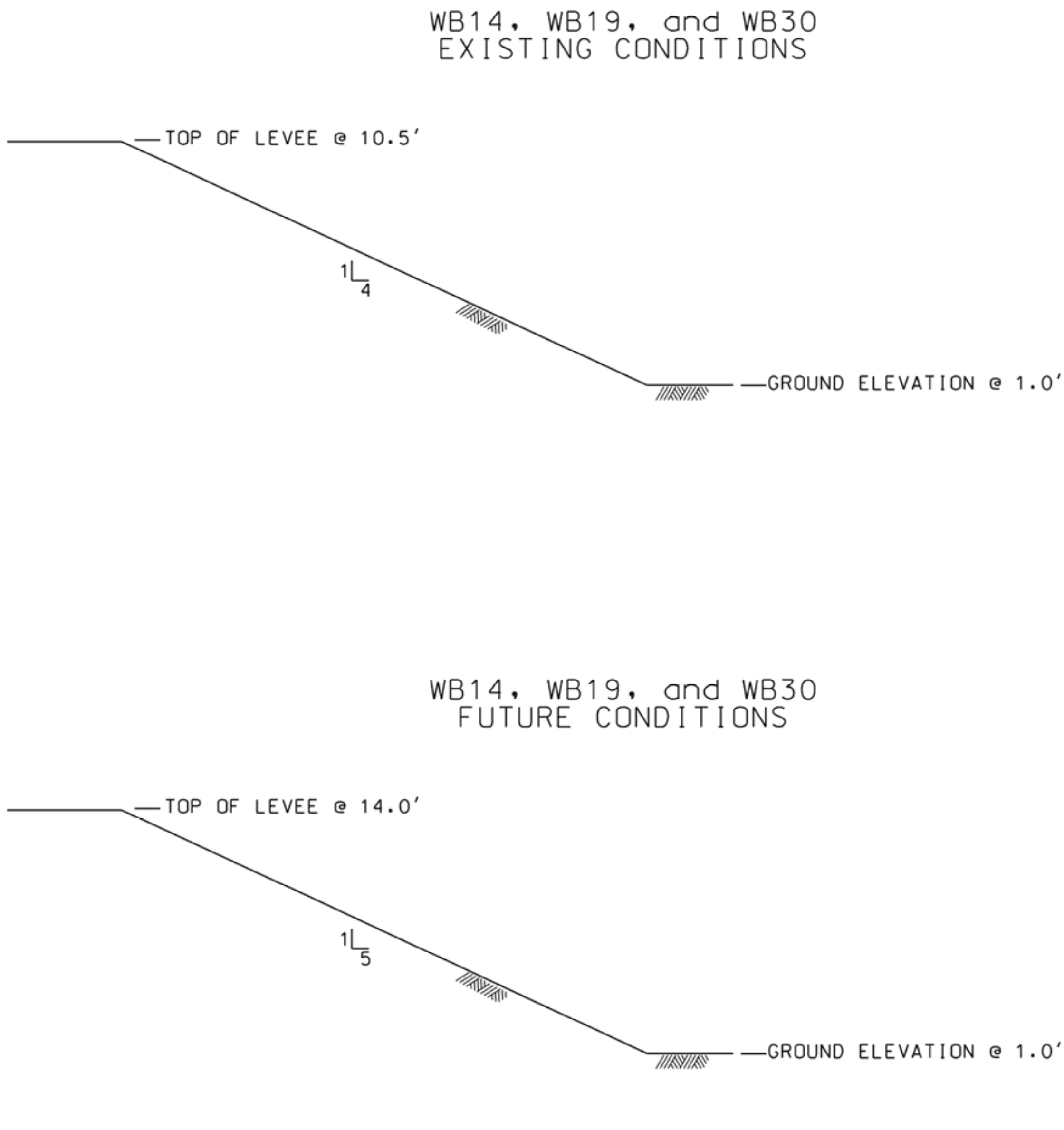


Figure 69 – Typical Design Cross Section for the levees along Harvey Canal (WB14), Algiers Canal (WB30) and Hero Canal (WB19) for existing conditions (upper panel) and future conditions (lower panel).

The levee design cross-section for the landfill (WB19-A) is a simple levee with 1V:4H slopes and no wave berms. The design elevation for existing conditions is 9ft. The crest must be elevated to 11ft to meet the design criteria under future conditions.

4.4.4 Wave Forces

Wave forces were computed for the structures along Harvey, Hero and Algiers Canal with the Goda method, using future conditions. The wave forces were evaluated for both irregular and breaking waves. The 50%-values and the 90%-values of the wave forces are both established based on the uncertainties in the hydraulic characteristics. The following tables summarize the resulting wave forces. Notice that the hydrostatic forces are not listed in these tables, but should be taken into account during design. A CD-ROM is available containing the diagrams of the wave and hydrostatic forces, and the hydraulic and structural input parameters.

West Bank - East of Harvey Canal Sections Wave forces on structures (50% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
WB14	Robinson Pt. to Harvey Canal W. Wall Levee	1.9	8.8	8.6	1.9	8.8	8.6
WB15	New Estelle Pump Station	3.4	57.5	0.9	3.4	57.5	0.9
WB16	Cousins Pump Station 1, 2 and 3 (on Harvey Canal)	3.1	40.2	3.1	3.1	40.2	3.1
WB19-AW	Hero Canal Floodwall behind Landfill Berm	0.4	2.4	7.0	0.4	2.4	7.0
WB19-W	Hero Canal Floodwall	2.2	12.6	6.8	2.2	12.6	6.8
WB23	Whitney Barataria and Belle Chase Pump Stations	3.2	45.3	2.2	3.2	45.3	2.2
WB24	Planters Pump Station	3.2	45.4	2.2	3.2	45.4	2.2
WB27	Hero Pump Station (on Harvey Canal)	3.1	40.4	3.1	3.1	40.4	3.1
WB30-W	NO SBW Pump Station 11 and 13	2.9	29.7	4.5	2.9	29.7	4.5
WB40	Harvey Canal Floodwall	1.8	11.3	7.3	1.8	11.5	7.3
WB40-L	Sector Gate at Lapalco Overpass on Harvey Canal	3.3	55.8	0.2	3.3	55.8	0.2

Table 65 – Waves Forces for East of Harvey Canal Segments (50% values)

West Bank - East of Harvey Canal Sections Wave forces on structures (90% values) associated with 1% design conditions							
		Irregular waves			Breaking waves		
Segment	Name	Force	Moment	Elevation	Force	Moment	Elevation
		x 1000 lb/ft	x1000 ft-lb/ft	ft	x 1000 lb/ft	x1000 ft-lb/ft	ft
WB14	Robinson Pt. to Harvey Canal W. Wall Levee	2.3	11.3	8.8	2.3	11.3	8.8
WB15	New Estelle Pump Station	4.5	75.5	1.7	4.5	75.5	1.7
WB16	Cousins Pump Station 1, 2 and 3 (on Harvey Canal)	4.1	53.8	3.4	4.1	53.8	3.4
WB19-AW	Hero Canal Floodwall behind Landfill Berm	0.5	3.0	7.1	0.5	3.0	7.1
WB19-W	Hero Canal Floodwall	2.7	16.0	7.0	2.7	16.0	7.0
WB23	Whitney Barataria and Belle Chase Pump Stations	4.2	59.3	2.5	4.2	59.3	2.5
WB24	Planters Pump Station	4.2	59.4	2.5	4.2	59.4	2.5
WB27	Hero Pump Station (on Harvey Canal)	4.1	53.9	3.4	4.1	53.9	3.4
WB30-W	NO SBW Pump Station 11 and 13	3.7	39.2	4.6	3.7	39.2	4.6
WB40	Harvey Canal Floodwall	2.2	14.7	7.5	2.3	14.7	7.5
WB40-L	Sector Gate at Lapalco Overpass on Harvey Canal	4.3	70.0	0.9	4.3	70.0	0.9

Table 66 – Waves Forces for East of Harvey Canal Segments (90% values)

4.4.5 Resiliency

The designs for West Bank sections along Harvey, Algiers and Hero Canal were examined for resiliency by also computing the overtopping rate for the 0.2 percent event for each design. The water level and overtopping rate was determined for the 50% assurance during the 0.2% event. The results are presented in Table 67. Apart from the landfill area (WB19-A and WB19-AW), the 0.2% surge elevation remains below the top of the flood defense. The overtopping rate can be quite significant during a 0.2% event for specific levees/floodwalls.

Westbank Sections (East of Harvey Canal Reach) Resiliency analysis (0.2% event)						
					Best estimates during 0.2% event	
Segment	Name	Type	Condition	Height (ft)	Surge level (ft)	Overtopping rate (cft/s per ft)
WB14	Robinson Pt. to Harvey Canal W. Wall Levee	Levee	Existing	10.5	10.9	2.947
WB14	Robinson Pt. to Harvey Canal W. Wall Levee	Levee	Future	14.0	12.9	1.343
WB15	New Estelle Pump Station	Structure/Wall	Future	16.0	12.9	0.084
WB40	Harvey Canal Floodwall	Structure/Wall	Future	14.0	12.9	0.803
WB16	Cousins Pump Station 1, 2 and 3 (on Harvey Canal)	Structure/Wall	Future	16.0	12.9	0.082
WB40-L	Sector Gate at Lapalco Overpass on Harvey Canal	Structure/Wall	Future	13.0	12.9	1.671
WB27	Hero Pump Station (on Harvey Canal)	Structure/Wall	Future	16.0	12.9	0.084
WB30	Hero Pump Station to Algiers Canal Levee	Levee	Existing	10.5	10.9	3.001
WB30	Hero Pump Station to Algiers Canal Levee	Levee	Future	14.0	12.9	1.405
WB23	Whitney Barataria Pump Station	Structure/Wall	Future	15.0	12.9	0.260
WB24	Planters Pump Station	Structure/Wall	Future	15.0	12.9	0.251
WB30-W	NO SBW Pump Station 11	Structure/Wall	Future	15.0	12.9	0.259
WB19	Transition Point to Hero Canal to Oakville	Levee	Existing	10.5	10.4	1.934
WB19	Transition Point to Hero Canal to Oakville	Levee	Future	14.0	12.4	0.901
WB19-W	Hero Canal Floodwall	Structure/Wall	Future	13.0	12.4	1.202
WB19-A	Hero Canal-Area Behind Landfill Berm	Levee	Existing	9.0	10.4	8.151
WB19-A	Hero Canal-Area Behind Landfill Berm	Levee	Future	11.0	12.4	8.218
WB19-AW	Hero Canal Floodwall behind Landfill Berm	Structure/Wall	Future	11.0	12.4	8.114

Table 67 – Resiliency for East of Harvey Canal Segments

5 Summary of Design Elevations

This chapter summarizes the design elevations of the various levee/floodwall sections. The hydraulic elevations and the design elevations are given for existing conditions and future conditions. Where there is a difference between the hydraulic and design elevations, structural superiority has been included. Notice that only the future conditions elevations are given for the structures (pump stations, walls, and gates), whereas both the existing and future conditions elevations are listed for levee sections.

St. Charles Parish

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
SC08	Bayou Trepagnier PS	Pump Station	N/A	16.5	N/A	18.5
SC11	Bonnet Carre Tie in Floodwall	Floodwall	N/A	16.5	N/A	18.5
SC05	Good Hope Floodwall	Gate	N/A	17.0	N/A	17.0
SC02-A	St.Charles Parish Levee west of I-310	Levee	15.5	18.0	15.5	18.0
SC07	Cross Bayou Canal T-Wall	Drainage	N/A	17.0	N/A	17.0
SC06	Gulf South Pipeline T-Wall	Pipeline	N/A	17.0	N/A	17.0
SC04	St. Rose Canal Drainage Structure T-Wall	Drainage	N/A	16.5	N/A	16.5
SC12	I-310 Floodwall	Floodwall	N/A	15.5	N/A	15.5
SC02-B	St.Charles Parish Levee east of I-310	Levee	14.0	16.0	14.0	16.0
SC09	Almedia Drainage Structure	Drainage	N/A	15.5	N/A	15.5
SC10	Walker Drainage Structure	Drainage	N/A	15.5	N/A	15.5
SC13	Armstrong Airport Floodwall	Floodwall	N/A	15.5	N/A	15.5
SC14	ICRR Floodgate	Gate	N/A	15.5	N/A	15.5
SC30	Transition	Floodwall	N/A	16.5	N/A	16.5
SC01-A	St. Charles Return Levee/Wall Lakeward	Floodwall	N/A	17.5	N/A	17.5
SC15	Shell Pipeline Crossing	Pipeline	N/A	17.0	N/A	17.0

Table 68 - Design Elevations St. Charles Parish

Jefferson Parish

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
JL09	Return wall	Floodwall	N/A	17.5	N/A	17.5
JL05	Pump Station 4	Pump Station	N/A	14.5	N/A	16.5
JL07	Williams Blvd Floodgate	Gate	N/A	14.5	N/A	16.5
JL04	Pump Station 3	Pump Station	N/A	17.0	N/A	19.0
JL01	Jefferson Lakefront Levees Reach 1-5	Levee	15.0	17.5	15.0	17.5
JL03	Pump Station 2	Pump Station	N/A	14.5	N/A	16.5
JL06	Causeway Crib Wall	Floodwall	N/A	20.5	N/A	20.5
JL02	Pump Station 1	Pump Station	N/A	14.5	N/A	16.5
JL08	Bonnabel Boat Launch Floodgate	Gate	N/A	14.5	N/A	16.5

Table 69 - Design Elevations Jefferson Parish Lakefront

New Orleans Metro Lakefront

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
NO06	NO Marina	Floodwall	N/A	16.0	N/A	16.0
NO10	Topaz St. Levee	Levee	15.0	17.5	15.0	17.5
NO15	Type II Floodgate/Similar to Canal Blvd	Floodwall	N/A	16.0	N/A	16.0
NO13	17th St. Outfall Canal Closure	Closure	N/A	16.0	N/A	16.0
NO12	Orleans Ave Outfall Canal Closure	Closure	N/A	16.0	N/A	16.0
NO14	Type I Floodgate Similar to Marconi Dr.	Floodwall	N/A	16.0	N/A	16.0
NO16	Lakeshore Dr. Near Rail St FG	Floodwall	N/A	16.0	N/A	16.0
NO07	Bayou St. John	Floodwall	N/A	16.0	N/A	16.0
NO11	London Ave Outfall Canal Closures	Closure	N/A	16.0	N/A	16.0
NO08	Pontchartrain	Floodwall	N/A	16.0	N/A	16.0
NO09	American Std FW	Floodwall	N/A	16.0	N/A	16.0
NO01	New Orleans Lakefront Levee	Levee	16.0	19.0	16.0	19.0
NO17	Leroy Johnson	Floodwall	N/A	16.5	N/A	16.5

Table 70 - Design Elevations New Orleans Metro Lakefront

New Orleans East Lakefront

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
NE01	Citrus Lakefront Levee	Levee	13.0	15.5	13.0	15.5
NE03	NO Lakefront Airport East FW	Floodwall	N/A	15.5	N/A	15.5
NE04	NO Lakefront Airport West FW	Floodwall	N/A	15.5	N/A	15.5
NE05	Lincoln Beach FW	Floodwall	N/A	15.5	N/A	15.5
NE07	Citrus PS FW	Pump Station	N/A	15.5	N/A	15.5
NE08	Jahncke PS FW	Pump Station	N/A	15.5	N/A	15.5
NE09	St Charles PS FW	Pump Station	N/A	15.5	N/A	15.5
NE30	Transition Reach from NE01 to NE02	Levee	14.5	16.5	14.5	16.5
NE02	NO East Lakefront Levee	Levee	15.5	17.5	15.5	17.5
NE06	NO East Lakefront Collins Pipeline Crossing	Floodwall	N/A	15.5	N/A	17.5
NE31	Southpoint Transition Reach	Levee	16.5	18.5	16.5	18.5

Table 71 - Design Elevations New Orleans East Lakefront

GIWW outside MRGO/GIWW Gates, including South Point to GIWW

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
NE10	South Point to Hwy 90 Levee	Levee	17.0	19.0	17.0	19.0
NE11A	Highway 90 to CSX RR Levee	Levee	22.0	25.0	22.0	25.0
NE11B	CSX RR to GIWW Levee	Levee	25.0	28.0	25.0	28.0
NE13	Highway 11 Floodgate	Gate	N/A	18.5	N/A	18.5
NE14	Highway 90 Floodgate	Gate	N/A	20.0	N/A	22.0
NE15	CSX RR Gate	Gate	N/A	28.0	N/A	30.0
NE32	Transition	Levee	28.0	31.0	28.0	31.0
NE12A	NO East Back Levee from PS15 East Along GIWW	Levee	28.0	31.0	28.0	31.0
NE12B	NO East Back Levee from Gate to PS15	Levee	29.0	31.5	29.0	31.5
NE16	NO East Pump Station 15	Pump Station	N/A	32.0	N/A	34.0

Table 72 - Design Elevations GIWW outside MRGO/GIWW Gates

IHNC and GIWW with MRGO/GIWW and without Seabrook Closure Structure

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
NO20	NS Railroad Gates Near Seabrook West	Gate	N/A	16.0	N/A	18.0
NE20	NS Railroad Gates Near Seabrook East	Gate	N/A	16.0	N/A	18.0
IH01-W	IHNC. South of I-10	Floodwall	N/A	13.5	N/A	13.5
IH02-W	IHNC. North of I-10	Floodwall	N/A	13.5	N/A	13.5
IH03	IHNC Levee South of I-10	Levee	12.0	13.5	12.0	13.5
IH01-W	IHNC lock to PS #5	Floodwall	N/A	13.5	N/A	13.5
IH05-W	Dwyer Pump Station	Pump Station	N/A	13.5	N/A	13.5
IH10	Orleans PS #5 and PS #19	Pump Station	N/A	13.5	N/A	15.5
IH30	Transition reach.	Levee	14.5	15.5	14.5	15.5
GI01	GI02 to IHNC	Levee	12.0	13.5	12.0	13.5
GI02	Paris Road to GI01	Levee	12.0	13.5	12.0	13.5
GI03	Bayou Bienvenue to GI03w	Levee	12.0	13.5	12.0	13.5
GI03	Michoud Canal to Michoud Slip	Levee	12.0	13.5	12.0	13.5
GI03W	Floodwall under Paris Rd Bridge	Floodwall	N/A	13.5	N/A	13.5
GI04	Michoud Canal and Slip	Floodwall	N/A	13.5	N/A	13.5
GI05	Amid Pump Station (PS#20)	Pump Station	N/A	13.5	N/A	13.5
GI06	Elaine Pump Station	Pump Station	N/A	13.5	N/A	13.5
GI07	Grant Pump Station	Pump Station	N/A	13.5	N/A	13.5
GI08	Bienvenue Floodgate	Gate	N/A	13.5	N/A	15.5

Table 73 - Design Elevations IHNC/GIWW with MRGO/GIWW Gates

IHNC with MRGO/GIWW and Seabrook Closure Structure

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
NO20	NS Railroad Gates Near Seabrook West	Gate	N/A	9.5	N/A	11.5
NE20	NS Railroad Gates Near Seabrook East	Gate	N/A	9.5	N/A	11.5
IH01-W	IHNC. South of I-10	Floodwall	N/A	9.5	N/A	9.5
IH02-W	IHNC. North of I-10	Floodwall	N/A	9.5	N/A	9.5
IH03	IHNC Levee South of I-10	Levee	10.5	10.5	10.5	10.5
IH01-W	IHNC lock to PS #5	Floodwall	N/A	9.5	N/A	9.5
IH05-W	Dwyer Pump Station	Pump Station	N/A	9.5	N/A	9.5
IH10	Orleans PS #5 and PS #19	Pump Station	N/A	9.5	N/A	11.5
IH30	Transition reach. Averages need to refine	Levee	10.5	10.5	10.5	10.5
GI01	GI02 to IHNC	Levee	10.5	10.5	10.5	10.5
GI02	Paris Road to GI01	Levee	10.5	10.5	10.5	10.5
GI03	Bayou Bienvenue to GI03w	Levee	10.5	10.5	10.5	10.5
GI03	Michoud Canal to Michoud Slip	Levee	10.5	10.5	10.5	10.5
GI03W	Floodwall under Paris Rd Bridge	Floodwall	N/A	9.5	N/A	9.5
GI04	Michoud Canal and Slip	Floodwall	N/A	9.5	N/A	9.5
GI05	Amid Pump Station	Pump Station	N/A	9.5	N/A	9.5
GI06	Elaine Pump Station	Pump Station	N/A	9.5	N/A	9.5
GI07	Grant Pump Station	Pump Station	N/A	9.5	N/A	9.5
GI08	Bienvenue Floodgate	Gate	N/A	9.5	N/A	11.5

Table 74 - Design Elevations IHNC/GIWW with MRGO/GIWW and Seabrook Gates

Seabrook and MRGO/GIWW Closure Structure

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
LEVEE A1	Closure Levee for MRGO and GIWW	Levee	29.0	31.5	29.0	31.5
GATE A1	Closure Gate for MRGO and GIWW	Gate	N/A	32.0	N/A	34.0
GATE A2	Seabrook Gate	Gate	N/A	16.0	N/A	18.0

Table 75 - Design Elevations MRGO/GIWW and Seabrook Gates and Levee

St. Bernard Parish

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
SB11	MRGO Reach A	Levee	29.0	31.5	29.0	31.5
SB12	MRGO Reach B	Levee	27.5	30.0	27.5	30.0
SB13	MRGO Reach C	Levee	26.5	29.0	26.5	29.0
SB15	MRGO Reach D	Levee	26.5	29.0	26.5	29.0
SB16	Verret to Caernarvon Reach A	Levee	26.5	29.0	26.5	29.0
SB17	Verret to Caernarvon Reach B	Levee	26.5	29.0	26.5	29.0
SB19	Bayou Dupre Floodgate	Gate	N/A	29.0	N/A	31.0
SB20	St. Mary Pump Station	Pump Station	N/A	28.5	N/A	30.5

Table 76 - Design Elevations St. Bernard Parish

Lake Cataouatche

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
WB31	Mississippi River to US90 Levees	Levee	9.0	13.0	9.0	13.0
WB02	Lake Cataouatche PS 1 & 2	Pump Station	N/A	13.5	N/A	15.5
WB01	US90 to Bayou Segnette State Park	Levee	11.5	15.5	11.5	15.5
WB43	Bayou Segnette State Park Floodwall	Floodwall	N/A	14.0	N/A	14.0
WB05	Bayou Segnette Pump Station 1 & 2	Pump Station	N/A	14.0	N/A	16.0

Table 77 - Design Elevations Lake Cataouatche

Westwego to Harvey Canal

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
WB07	New Westwego Pump Station	Pump Station	N/A	14.0	N/A	16.0
WB08-A	Segnette PS to Company Canal Levee	Levee	10.5	14.0	10.5	14.0
WB08-B	New Westwego PS to Orleans Village Levee	Levee	10.5	14.0	10.5	14.0
WB10	Westminster PS	Pump Station	N/A	14.0	N/A	16.0
WB11	Ames to Kennedy Floodwall	Floodwall	N/A	14.0	N/A	14.0
WB11-P	Ames Pump Station	Pump Station	N/A	14.0	N/A	16.0
WB11-P	Kennedy Pump Station	Pump Station	N/A	14.0	N/A	16.0
WB12	Old Estelle Pump Station	Pump Station	N/A	14.0	N/A	16.0
WB14	Robinson Pt. to Harvey Canal W. Wall Levee	Levee	10.5	14.0	10.5	14.0
WB15	New Estelle Pump Station	Pump Station	N/A	14.0	N/A	16.0
WB16	Cousins Pump Station 1, 2, and 3 (on Harvey Canal)	Pump Station	N/A	14.0	N/A	16.0
WB27	Hero Pump Station (on Harvey Canal)	Pump Station	N/A	14.0	N/A	16.0
WB32	HWY 45 to HWY 3134	Floodwall	N/A	14.0	N/A	14.0
WB40	Harvey Canal Floodwall	Floodwall	N/A	14.0	N/A	14.0
WB40-L	Sector Gate at Lapalco Overpass on Harvey Canal	Gate	N/A	13.0	N/A	13.0
WB41	Highway 3134 to Old Estelle PS Levee	Levee	10.5	14.0	10.5	14.0
WB42	Orleans Village to Ames PS Levee	Levee	10.5	14.0	10.5	14.0
WB42	Kennedy PS to Hwy 45 Levee	Levee	10.5	14.0	10.5	14.0
WB43-A	Segnette PS to Company Canal Floodwall	Floodwall	N/A	14.0	N/A	14.0
WB43-B	Company Canal & Westwego Floodwall	Floodwall	N/A	14.0	N/A	14.0
WB43-C	Old Westwego Pump Station	Pump Station	N/A	14.0	N/A	16.0
WB44	Old Estelle to Robinson Point	Floodwall	N/A	14.0	N/A	14.0

Table 78 - Design Elevations Westwego to Harvey Canal

East of Harvey Canal

Segment	Description	Feature Type	Ex Cond Hydraulic Elevation	Fut Cond Hydraulic Elevation	Ex Cond Design Elevation	Fut Cond Design Elevation
WB19	Transition Point to Hero Canal to Oakville	Levee	10.5	14.0	10.5	14.0
WB19A	Hero Canal-Area Behind Landfill Berm w/sm waves	Levee	9.0	11.0	9.0	11.0
WB19AW	Hero Canal Floodwall Behind Landfill Berm	Wall	N/A	11.0	N/A	11.0
WB23	Belle Chasse Pump Station 2	Pump	N/A	13.0	N/A	15.0
WB23	Whitney Barataria PS	Pump	N/A	13.0	N/A	15.0
WB23	Belle Chasse Pump Station 1	Pump	N/A	13.0	N/A	15.0
WB24	Planters Pump Station	Pump	N/A	13.0	N/A	15.0
WB30	Hero PS to Algiers Canal Levee	Levee	10.5	14.5	10.5	14.5
WB30-W	NO SWB Pump Station 11	Pump	N/A	13.0	N/A	15.0
WB30-W	NO SWB Pump Station 13	Pump	N/A	13.0	N/A	15.0

Table 79 - Design Elevations East of Harvey Canal

6 Conclusions and Recommendations

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. The elevations are sufficient to provide protection from a hurricane event that would produce a 1% exceedence surge elevation and associated waves.

The design elevations and levee slopes presented in this report are the initial values determined from hydraulic analyses and will form the baseline for detailed design. The designers will work with the hydraulic engineers in an iterative process to prepare plans and specifications. 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 documented.

The design elevations and slopes presented in this report are based on a given alignment and the topographic and bathymetric conditions at the site. Detailed surveys were used where available, but use of Lidar and historic data were also utilized. During the design process, detailed survey data will be taken, and there will be the opportunity to reverify the values presented in this report.

Soil borings will also be taken during the design process, and stability calculations performed. Changes in the topographic conditions at a levee or structure may occur, necessitating the need to reverify the values presented in this report.

The designers may look at alternatives such as new alignments and changing a levee to a floodwall, and these alternatives can include measures to reduce wave overtopping. If wave overtopping is reduced, design elevations may be reduced, or levee slopes may be steepened. Typical levees slopes are grass covered and are therefore considered to be “smooth”. The placement of riprap on the slope roughens the surface and thereby reduces overtopping. Breakwaters can be used at levees, floodwalls and floodgates to alter the waves before they can break on the structure. Vegetation also alters the wave characteristics; adding roughness by planting trees appears to have merit in reducing wave overtopping.

Changes to the design elevations will be documented in addenda to this report. In addition, the addenda will also include the hydraulic analysis performed for the evaluation of alternatives.

This report documents the process followed by the New Orleans District hydraulic engineers to determine these protection system design elevations. Draft design guidance has been prepared that incorporates the procedures described in this report. Continued evaluation of the tools, processes, and procedures used in the development of the design elevations and slopes is an important goal. With continued research, design guidance can revise. The design guidance will be updated routinely.

Several areas have been identified for further investigation. They are:

1. Occurrence of the maxima of surge levels and wave characteristics.
2. Application of friction in wave modeling
3. Methods to calculate wave overtopping
4. Wave overtopping limits and damage thresholds
5. Armoring and Resiliency

Occurrence of maxima

The hydraulic designs have been calculated from the 1% surge levels, 1% wave heights and 1% wave periods near the toe of the levees and structures. In this approach, the correlation between the surge and the waves was not taken into account. Because the water depth is relatively shallow, one may expect that the surge level and the wave height are closely correlated. However, this approach is conservative and further research is recommended to analyze the magnitude of this effect.

Application of friction in wave modeling

The STWAVE results used in the 1% design elevations also do not consider friction. The STWAVE model runs used in IPET also did not include friction. Disagreement exists among internal and external experts as to the effects of friction on waves and which model results, friction or no friction, best represent the wave climate. Experts, such as Don Resio of ERDC, believe the model results without friction better represent the wave climate.

A key problem in this discussion is that the calibration of the STWAVE model was limited due to the lack of near shore wave data. ERDC has initiated data collection in Lake Borgne. In addition, the New Orleans District is formulating plans for the placement of wave gages west of the Mississippi River to collect wave data in the area between Grand Isle and the West Bank and Vicinity Hurricane Protection System. The collection of wave data is critical to developing wave models and assessing methods to compute wave overtopping.

An action plan is to be developed with the goal to find ways to reduce the uncertainty in wave characteristics and increase our confidence in our design parameters. Consultation with internal and external experts such as Dr Bob Dean of University of Florida, who was on the IPET team and the ASCE ERP for the 1% design elevations, will take place.

Methods to calculate wave overtopping

During the hydraulic design, it was recognized that empirical methods cannot cope with very complex geometries. Process-based methods are in the early stage of development and their application is not well suited to a detail design process. ERDC initiated development of tools to aid the hydraulic engineer during the hydraulic design, but as of August 2007, these tools are not complete.

There is a need for further research into developing design tools that can model the physics of wave runup and overtopping for levees and structures and also be practical and implementable. Wave overtopping field data will be useful in assessing the methodologies and their applicability to coastal Louisiana.

There is limited information on wave runup and overtopping for recurved walls. The 1984 Shore Protection Manual contained design parameters for one recurved wall design. The Coastal Engineering Manual does not have information on recurved walls. Recurved walls may provide a solution to areas where there limits to structural solutions. Additional research on recurved walls and other possible innovative design solutions is needed.

Wave overtopping limits and damage thresholds

Design criteria for the levees and structures elevations consider wave overtopping limits. Guidelines for establishing the overtopping rate threshold (i.e., the threshold associated with the onset of levee erosion and damage) for different types of embankments can be found in EM 1110-2-1100 (Part VI), Table VI-5-6. These threshold values are consistent with those that are adopted by the Technical Advisory Committee on Flood Defence in the Netherlands, (TAW 2002).

There is no clear field experience in coastal Louisiana to support overtopping rates higher than those presented in these documents. After consultation with the ASCE External Review Panel, the following wave overtopping rates have been established for the New Orleans District hurricane protection systems:

- 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.

During the coastal and hydraulic engineering analyses, it became apparent that additional analysis, research, and experimentation are needed in overtopping. Determining the allowable overtopping rates depends on an understanding of the erosion processes that occur during the overtopping event, and the quality of construction and maintenance of the levee system. Much of the current methodology is based on research on grass spillways and slopes for areas outside the New Orleans District. Little research is focused on MVN levees or on combination of floodwalls and levees.

Upcoming ERDC research for Homeland Security will fill a great need in advancing our understanding of erosion processes; however, it does not consider transitions or floodwall/levee combinations. Katrina showed that these features are the weak links in the hurricane protection system.

Because of the lack of analysis and experiments pertaining to the New Orleans hurricane protection system, present design elevations are based on conservative assumptions. Opportunity exists to save millions of dollars in initial construction costs and in future lifts with additional comprehensive research and analysis that focuses on the hurricane protection system within the New Orleans District.

Some of the critical research and analysis needed:

1. Large flume overtopping tests. Estimates of overtopping from the Boussinesq and the Empirical models diverge at low overtopping rates. Differences between their predictions could well translate into 1-3 ft of design height in hurricane protection levees; consequently, it is of critical importance that we resolve the source of these differences and apply the appropriate model in our final designs. Much of this problem arises from the fact that model testing in flumes has been focused on situations with much higher overtopping rates than the low values being considered for design (0.01 to 0.1 cfs/ft). Thus, the empirical models are forced to extrapolate from their region of experimentation into this low overtopping range. It is very possible that frictional effects could force low overtopping rates to deviate significantly from the scaling characteristics of higher overtopping situations. The Boussinesq model incorporates terms that should properly account for this difference, but due to the lack of good lab data for low overtopping rates has not been thoroughly validated for this situation. It is proposed that a set of near-prototype scale (1:3 – 1:1) tests be run with actual vegetation in place on top of typical levee sections in order to 1) resolve the differences between the empirical and Boussinesq model estimates 2) investigate the role of levee vegetation on reducing overtopping rates, and 3) provide a better foundation for making critical design decisions in the New Orleans area. It is estimated that these experiments would take about 2 months to conduct at a test facility.

2. Improved estimates of overtopping and breaching during Katrina. It is very important to continue to evaluate the IPET results and perform a detailed investigation of overtopping, breaching, and resulting flooding (with and without breaching) in the Southeast Louisiana during Katrina. These data will be critical to interpretations of levee fragility based on actual data and would also provide valuable insights relative to the contribution of levee failures to the timing and levels of flooding in areas affected by Katrina.

3. On site overtopping tests. The ASCE external review panel expressed concerns about construction and maintenance practices for levees and floodwalls. Overtopping thresholds were selected for the initial 1% design elevations based on good material, construction practices, and maintenance practices. On-site physical testing similar to recent tests the Dutch performed on their levees are critically needed to define the resiliency of our levees to differing overtopping rates. The Dutch tested grass, grass over geotextile, and bare levee conditions. Tests can be expanded to consider some of our local conditions and address some local construction concerns such as:

    - Local Levee Core Construction Materials
    - Local Vegetation
    - Saturated Levee Conditions
    - Drought Levee Conditions
    - Geotextile under Grass
    - Geotextile in place with additional lifts constructed on top

On-site testing of overtopping rates on levees within the New Orleans District hurricane protection system will increase public confidence in the levee system as well as the USACE.

On-site testing can become part of the levee certification process. On-site levee tests will also add to the body of knowledge on resiliency.

Armoring and Resiliency

P.L. 109-234 Title II, Chapter 3, Flood Control and Coastal Emergencies, page 38 (120 STAT. 455), hereinafter "4th Supplemental", provides : "For an additional amount for 'Flood Control and Coastal Emergencies', as authorized by section 5 of the Act of August 18, 1941 (33 U.S.C. 701n), for necessary expenses relating to the consequences of Hurricane Katrina and other hurricanes, $3,145,024,000, to remain available until expended: *Provided*, That the Secretary of the Army is directed to use the funds appropriated under this heading to modify, at full Federal expense, authorized projects in southeast Louisiana to provide hurricane and storm damage reduction and flood damage reduction in the greater New Orleans and surrounding areas; …$170,000,000 shall be used for armoring critical elements of the New Orleans hurricane and storm damage reduction system: . . . "

The Flood Control and Coastal Emergencies Section of Title II, Chapter 3 of the Joint Explanatory Statement of the Committee of Conference, Flood Control and Coastal Emergencies, page 115, states "Funds totaling $3,145,024,000 are recommended to continue repairs to flood and storm damage reduction projects. These projects are to be funded at full Federal expense. . . . Additionally, the Conferees include: . . .$170,000,000 for levee and floodwall armoring; . . ."

Armoring is defined as: A natural or artificial material placed on or around a levee, floodwall, or other structure to reduce damage and protect from catastrophic damage (damage that compromises or undermines the structural integrity and design intent) when confronted with overflow and overtopping from a storm in excess of the design event. The minimum armoring for levees shall be grass. Armoring is only one of the components of resilience and is integral to design.

IPET identified resilience as one of the "Overarching Lessons Learned" from Hurricane Katrina. Resiliency is generally defined as: The capacity of the levee / floodwall to resist, without catastrophic failure, overtopping (wave or surge) caused by a storm which is greater than the design event or the ability to withstand, without catastrophic failure, forces, and conditions, beyond those intended or estimated in the design. For our purposes, resilience refers to the ability to withstand higher than designed water levels and overtopping without breaching.

A Project Management Plan is under development to define and establish design criteria for armouring and resiliency. A key issue is identification of damage thresholds from wave overtopping. Equally important is the need to relate research and field testing results back to the methods for calculating overtopping.

The research and field testing for wave overtopping will provide valuable information to validate or refine existing damage threshold values. ERDC has recently developed two methods to estimate flow velocities associated with wave runup and overtopping: an empirical technique based on large-scale laboratory experiments and a numerical technique using a Boussinesq model. Additional testing is needed to acquire velocity measurements for validating the empirical technique and improve the accuracy and reliability of the numerical Boussinesq model.

This report includes the calculation of surge elevations, wave and overtopping flow from the 0.2% annual exceedence surge elevation. The 0.2% annual exceedence was selected as a starting point for assessing damage thresholds and establishing design criteria. For urban areas such as New Orleans, the 0.2% exceedence probability is considered an appropriate minimum level of evaluation of resiliency.

USACE experts, academia, and ASCE external review members attended a resiliency workshop held in New Orleans on 4-5 September, 2007. The participants strongly recommended a focused Resiliency Team be formed to develop concepts, methods, and tools for incorporating resiliency into the design. The draft resiliency workshop report, New Orleans Hurricane Protection System, Resiliency and Overtopping Workshop, outlines possible goals and charter for the Resiliency Team.

A small nucleus of dedicated full time staff, with expertise in several key areas, including geotechnical, hydraulic, structural, policy, risk management, construction, and maintenance would work over the next 6 months to develop concepts and methods, identify necessary research, and integrate the products into design guidance and policy. The Resiliency Team would also work with the Armoring Team, national Levee Assessment and Levee Certification Teams to share knowledge, leverage resources, integrate products, and standardize methods.

From a national perspective, other areas of the country are looking for guidance with levee design. This work could serve as a template or starting point for other levee systems throughout the nation. From a global perspective, it appears that we may be the first to explicitly consider resiliency in our design methodology. Pursuing resiliency research would allow the USACE to make a significant contribution in the global community of practice for levee design.

Examples of critical research and analysis needed:

1. Improved understanding of design problems in the vicinity of transitions from hard structures to earthen levees via physical model testing. Wave and currents interactions with a hard vertical structure produce very different forces on adjacent levees than are present in the absence of structures. It is essential that an improved understanding of these forces be obtained before these transitions are tested by the next major hurricane.

2. Continued detailed investigation of overtopping, breaching, and resulting flooding (with and without breaching) in the Southeast Louisiana during Katrina. These data will be critical to interpretations of levee fragility based on actual data and would also provide valuable insights relative to the contribution of levee failures to the timing and levels of flooding in areas affected by Katrina.

7 References

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8 Terminology and Abbreviations

A

ABFE-	Advisory Base Flood Elevation
ACES -	Automated Coastal Engineering System
ADCIRC -	Advanced Circulation Model
AMID -	Almonaster-Michoud Industrial District

B

Blvds -	Boulevards
BN&SF RR -	Burlington Northern and Santa Fe Railroad
bw -	Breakwater

C

C -	Caernarvon
CEDAS -	Coastal Engineering Design and Analysis System
CEM -	Coastal Engineering Manual
cfs or cft/s -	cubic feet per second
COULWAVE -	Cornell University Long and Intermediate Wave Modeling Package
CSX RR-	The Chessie System Railroad

D

DFIRM -	Digital Flood Insurance Rate Map

E

El. -	Elevation
EO -	East Orleans
ER -	US Army Corps of Engineers Engineer Regulation
ERDC -	US Army Corps of Engineers Engineering Research and Development Center
EST -	Empirical Simulation Technique
ETL-	US Army Corps of Engineers Engineering Technical Letter

F

F -	Force
FEMA -	Federal Emergency Management Authority
ft -	feet or foot
FUNWAVE -	Fully Nonlinear Boussinesq Wave Model

G

GIWW -	Gulf Intracoastal Waterway

H

H -	Horizontal
Hs -	Significant Wave Height

HURDAT -	Database with historical hurricane data
HURWIN -	
Hwy -	Highway

I

ICRR -	Illinois Central Railroad (Canadian National Railroad)
IHNC -	Inner Harbor Navigational Canal (Industrial Canal)
IPET -	Interagency Performance Evaluation Team

J

JPM-OS -	Joint Probability Method - Optimal Sampling

L

L0 -	Deep water wave length
LA -	Louisiana
LACPR -	Louisiana Coastal Protection and Restoration Study
LCA -	Louisiana Coastal Area Plan

M

M -	Moment
MATLAB -	A numerical package developed by The MathWorks
MCS -	Monte Carlo Simulation
MRGO -	Mississippi River Gulf Outlet
MVN -	New Orleans District, Mississippi Valley Division US Army Corps of Engineers

N

NAVD -	North American Vertical Datum
NFIP-	National Flood Insurance Program
NO -	New Orleans
NOAA -	National Oceanic and Atmospheric Administration
NSRR -	Norfolk Southern Railroad
NWB -	North West Bank

O

ORPT-	One Percent Review Team

P

P.S. -	Pump Station
PBL -	Planetary Boundary Layer
PC-Overslag -	Dutch Wave Run-up and Overtopping Software

R

REF/DIF -	Refraction/Diffraction Model

S

s -	second
SBN -	St. Bernard North
SBS -	St. Bernard South

SHORECIRC - A Quasi 3-D Nearshore Model
SLOSH - Sea, Lake, and Overland Surges from Hurricanes
SPH - Standard Project Hurricane
SPM - Shore Protection Manual
SSP - South Shore Lake Pontchartrain
std - Standard Deviation
STWAVE - Steady State Spectral Wave Model
SWAN - Simulating Waves Nearshore Model
SWB - South West Bank

T
TAW - Technical Advisory Committee on Flood Defense (The Netherlands)
Tp - Peak Wave Period

V
V - Vertical

W
WAM - Global Ocean Wave Prediction Model
WIFM WES - Implicit Flooding Model
WIFM - Waterways Experimental Station Implicit Flooding Model
WISWAVE - Wave Information Study Wave Model
WSE - Water Surface Elevation

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9 Appendices

9.1 Appendix A – Maps of 1% still water levels, wave heights, and wave periods

This appendix presents the 1% still water levels, 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 of the main report.

Figure A.1 1% still water levels at the Lakefront.

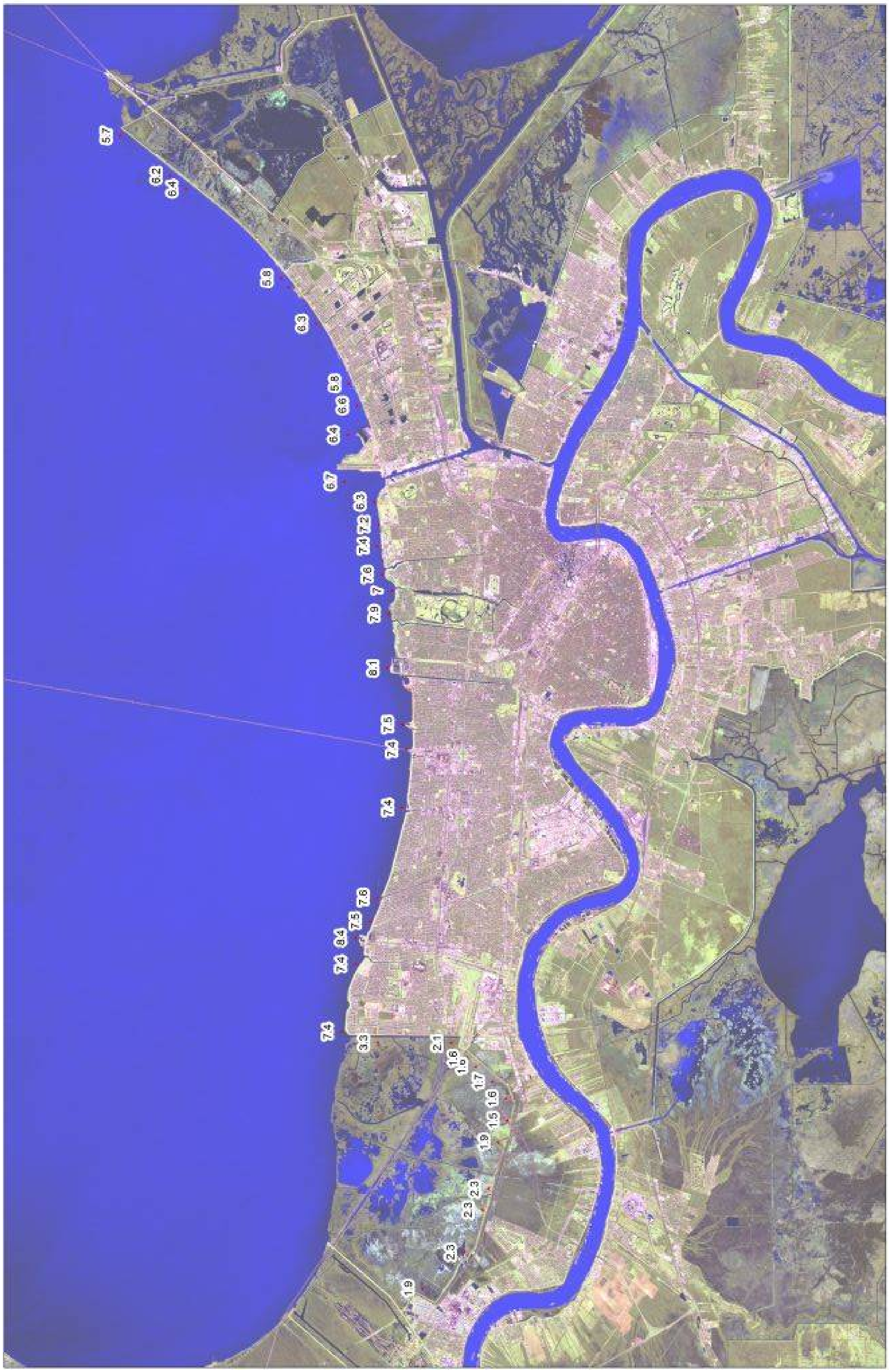

Figure A.2 1% significant wave heights at the Lakefront.

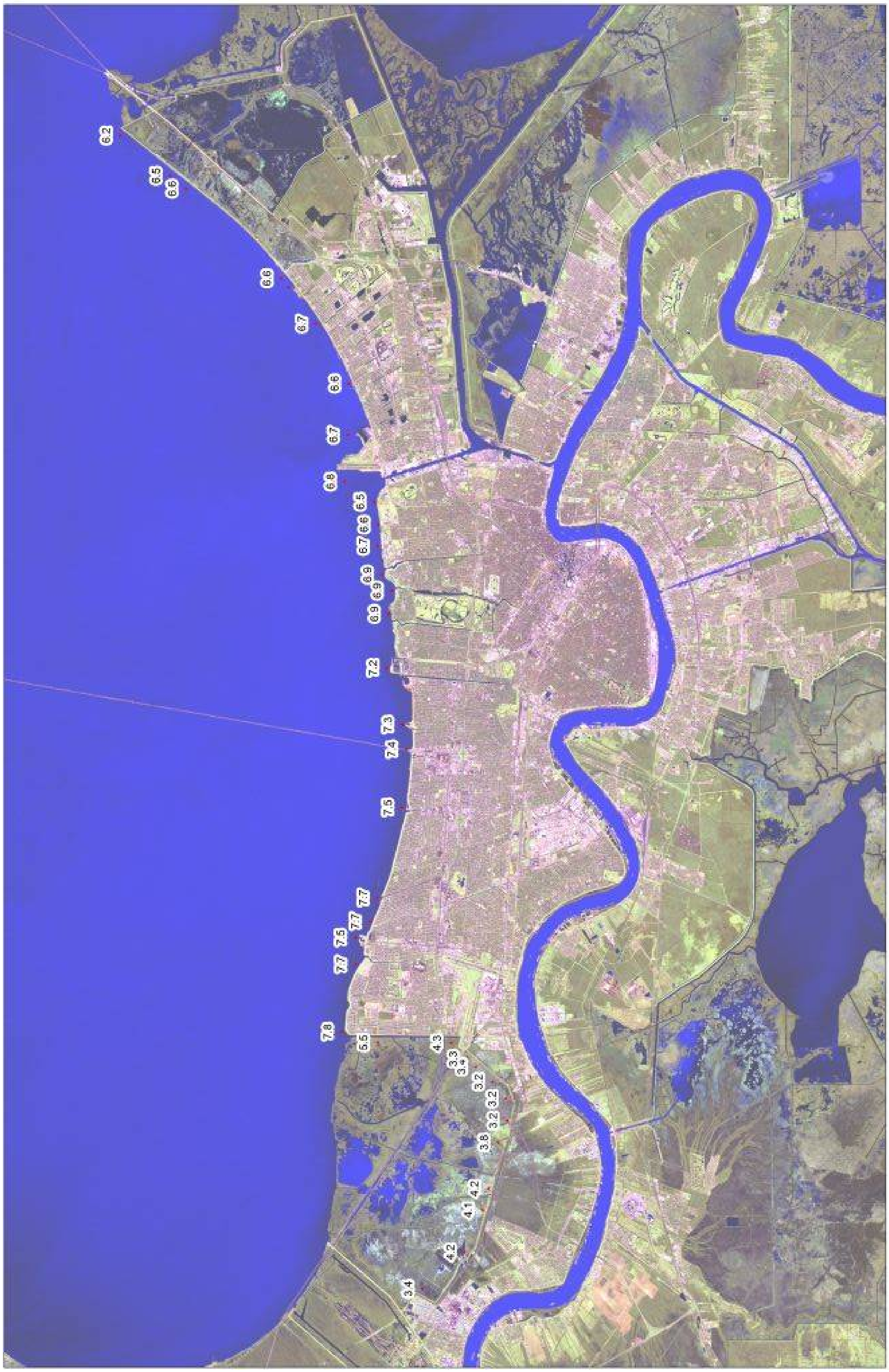

Figure A.3 1% peak period at the Lakefront.

Figure A.4 1% still water 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).

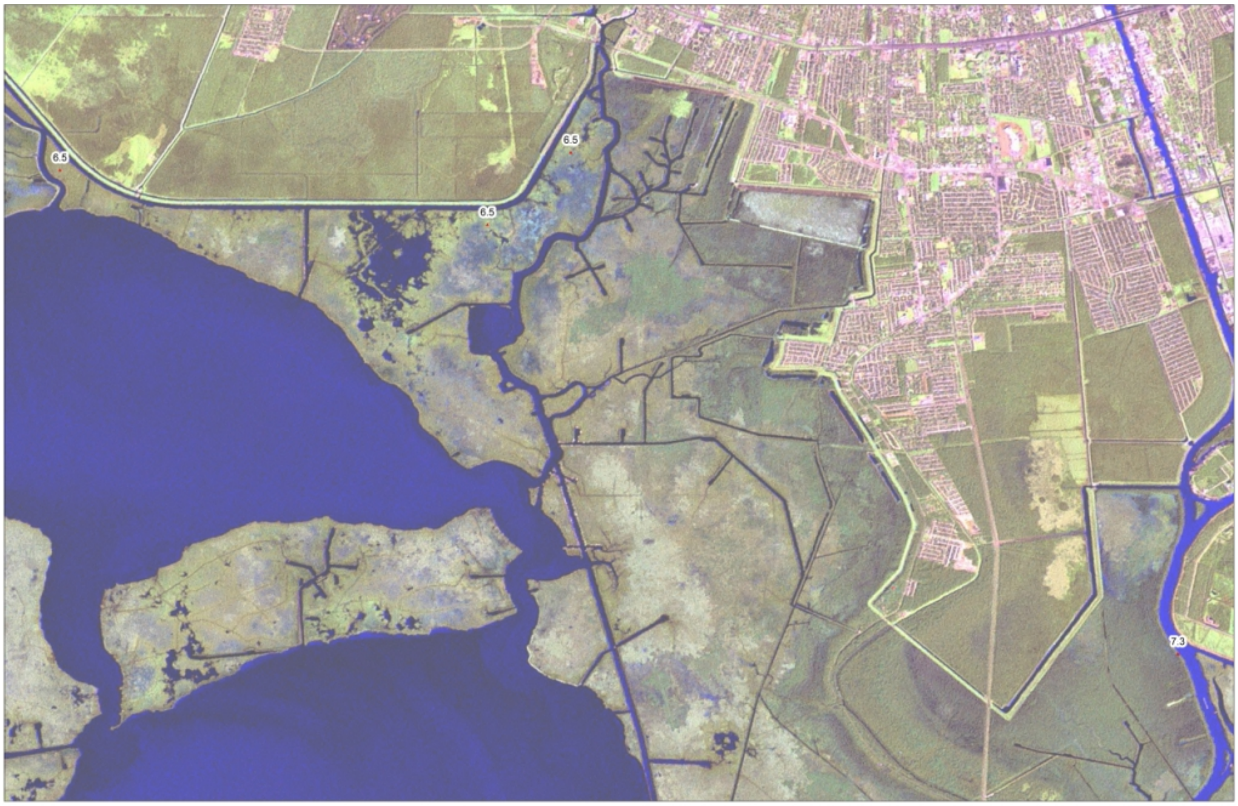

Figure A.7 1% still water levels at the West Bank.

Figure A.8 1% significant wave heights at the West Bank.

Figure A.9 1% peak period at the West Bank.

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9.2 *Appendix B - Boussinesq Modelling (Author: P. Lynett, Texas AM)*

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 dimensions (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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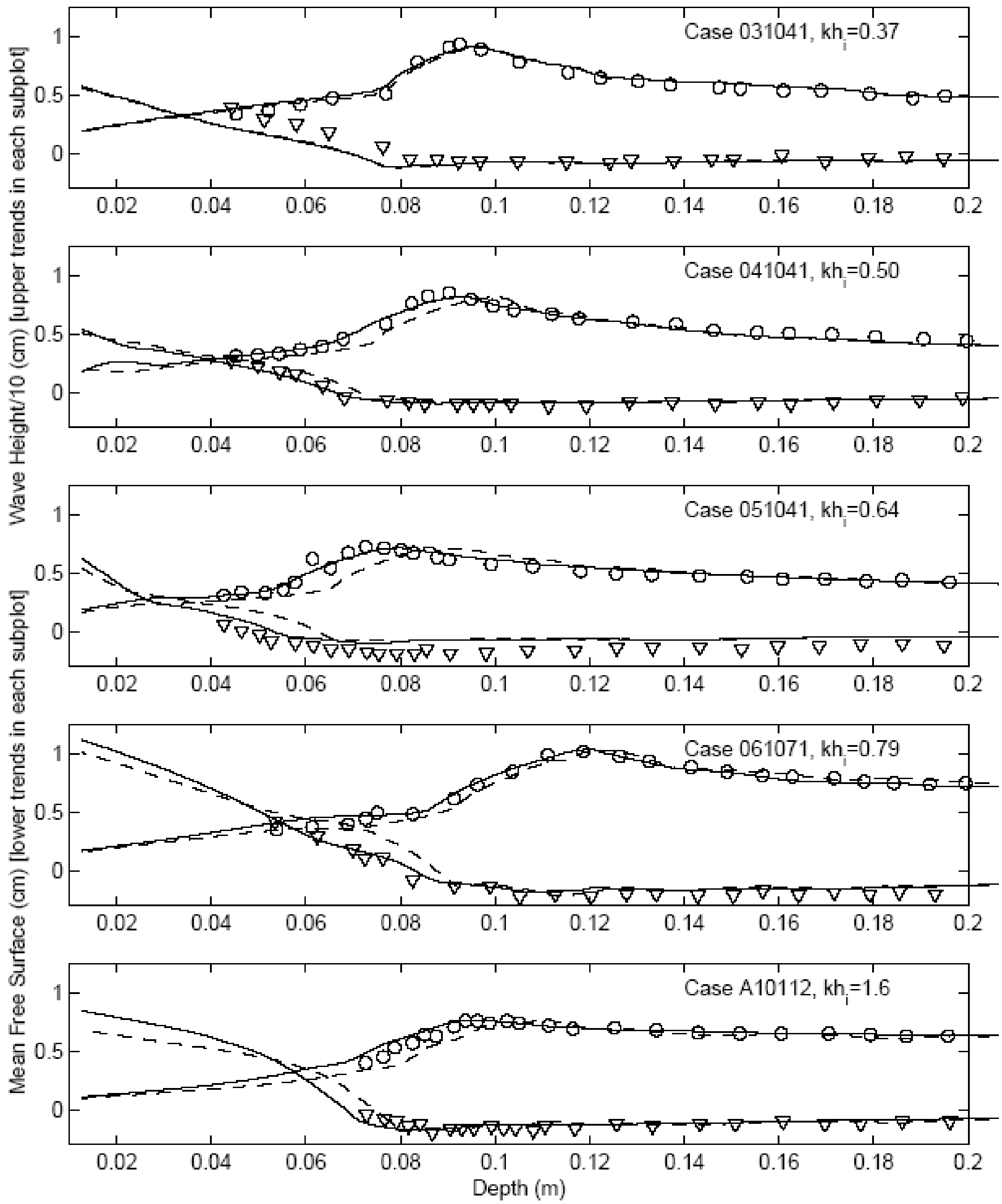

Figure 1. Wave height and mean free surface measurements from the experiments of Hansen and Svenson (1978) (symbols), from the traditional Boussinesq model (dashed-line), and from COULWAVE (solid line). Trials are for monochromatic waves breaking on a planar 1/20 slope.

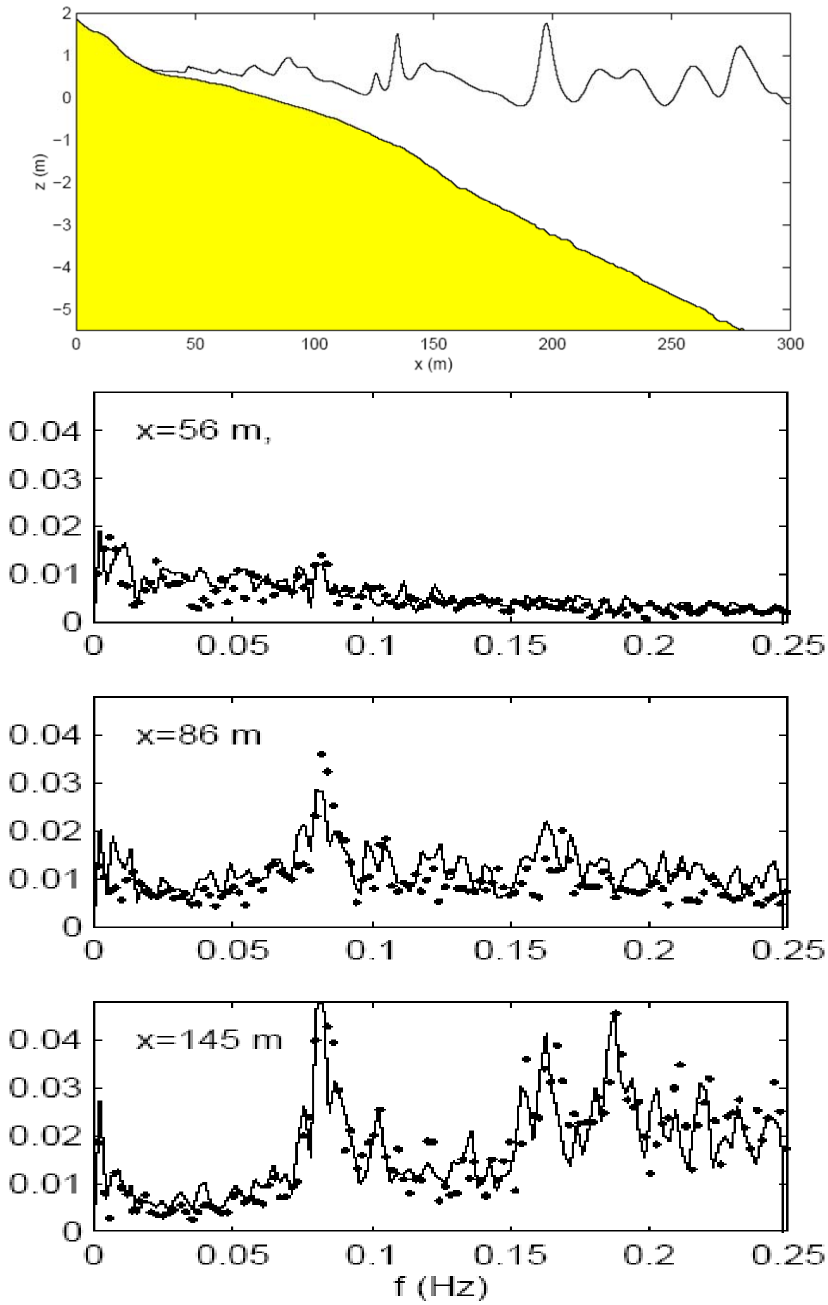

Figure 2. COULWAVE random wave comparison with field data. The lower subplots show the spectrum comparisons at three different locations, where the dots are the field data from Raubenhiemer (2002), and the solid lines are the COULWAVE results.

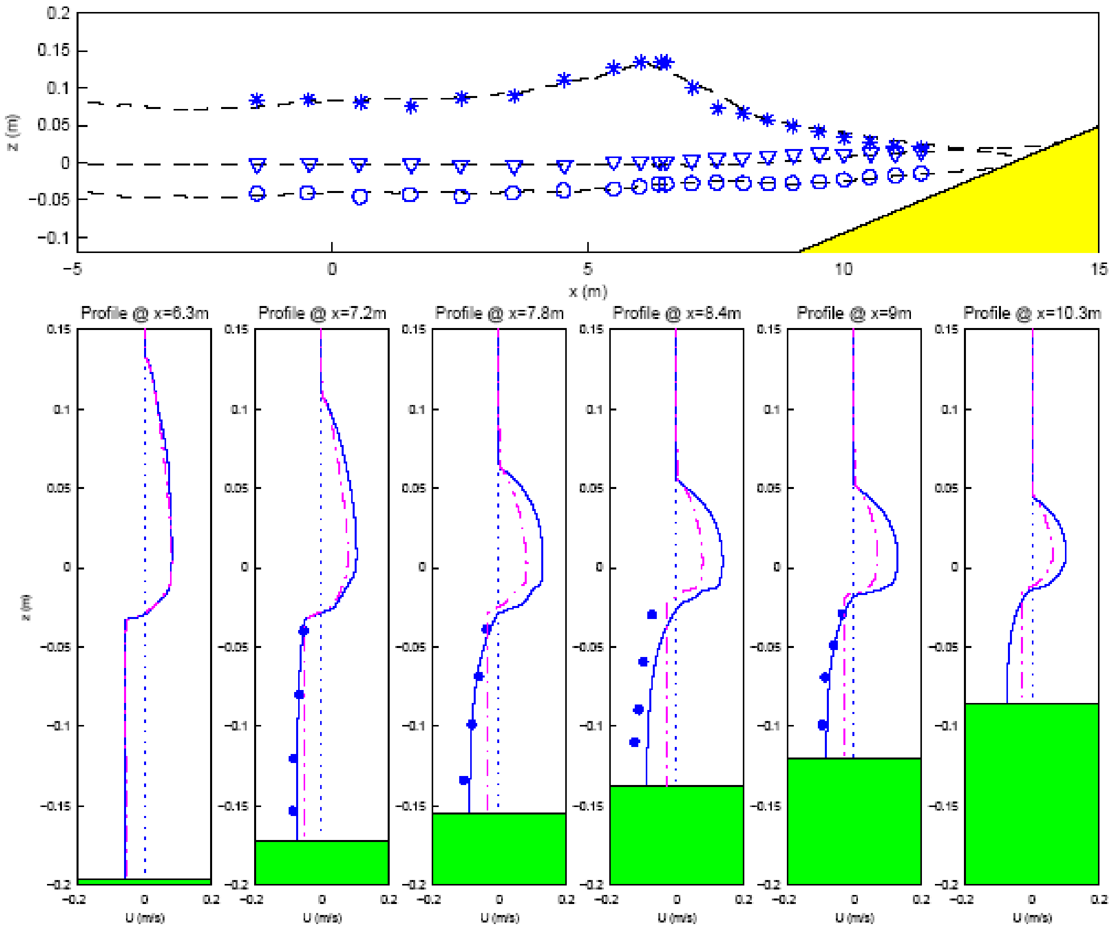

Figure 3. Comparison with the data of Ting and Kirby (1995) spiller. The top plot shows the mean crest level (stars), mean water level (triangles), and mean trough level (circles) for the experiment as well as the numerical simulation. The lower subplots are the time-averaged horizontal velocities, where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

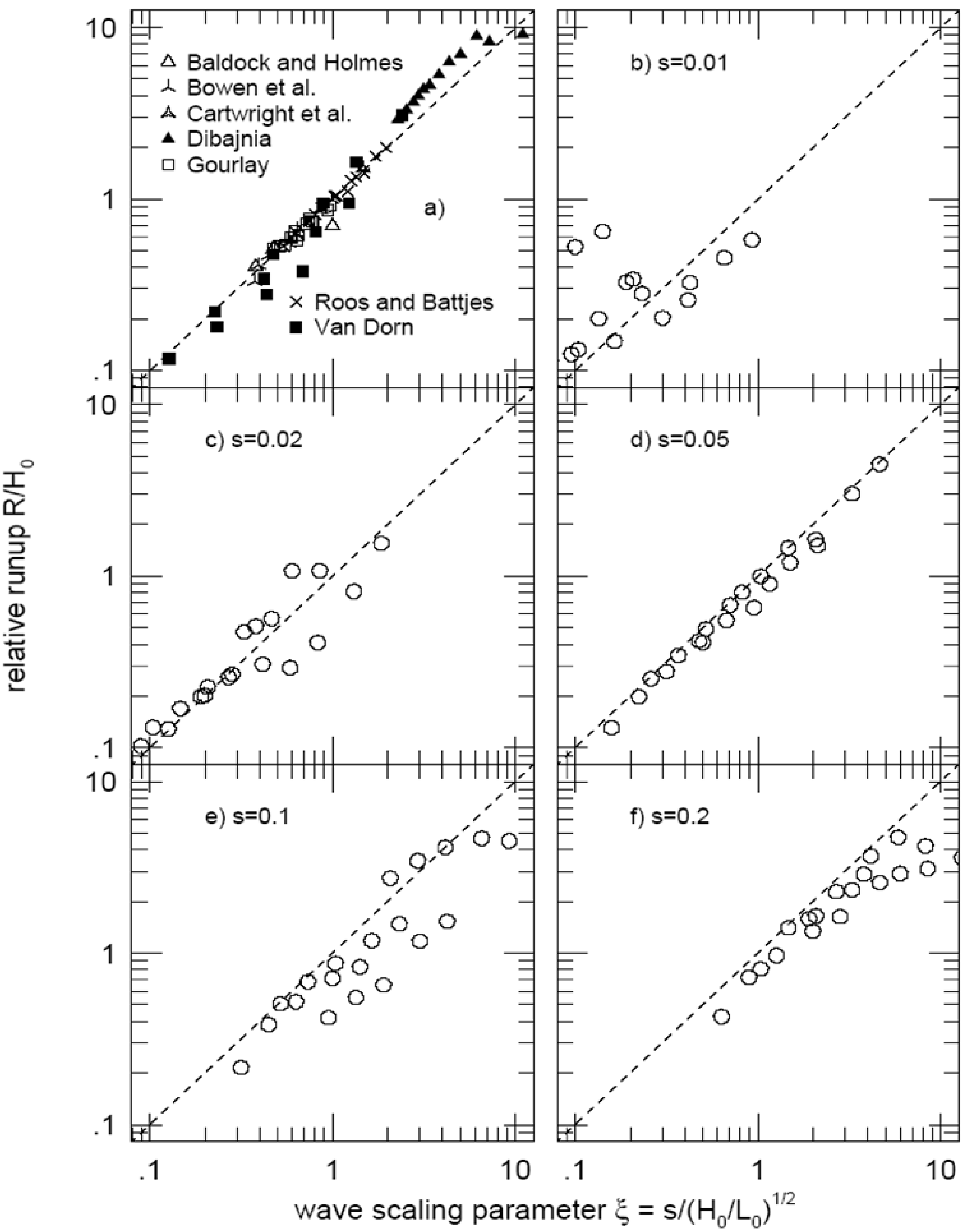

Figure 4. Wavetank experimental measurements of runup from the literature (Bowen *et al.*, 1968; Roos and Battjes, 1976; Van Dorn, 1976, 1978; Gourlay, 1992; Baldock and Holmes, 1999; Gourlay, 1992; Dijabnia, 2002) and COULWAVE runup results (open circles). The relative runup R/H_0 is plotted vs. the 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.

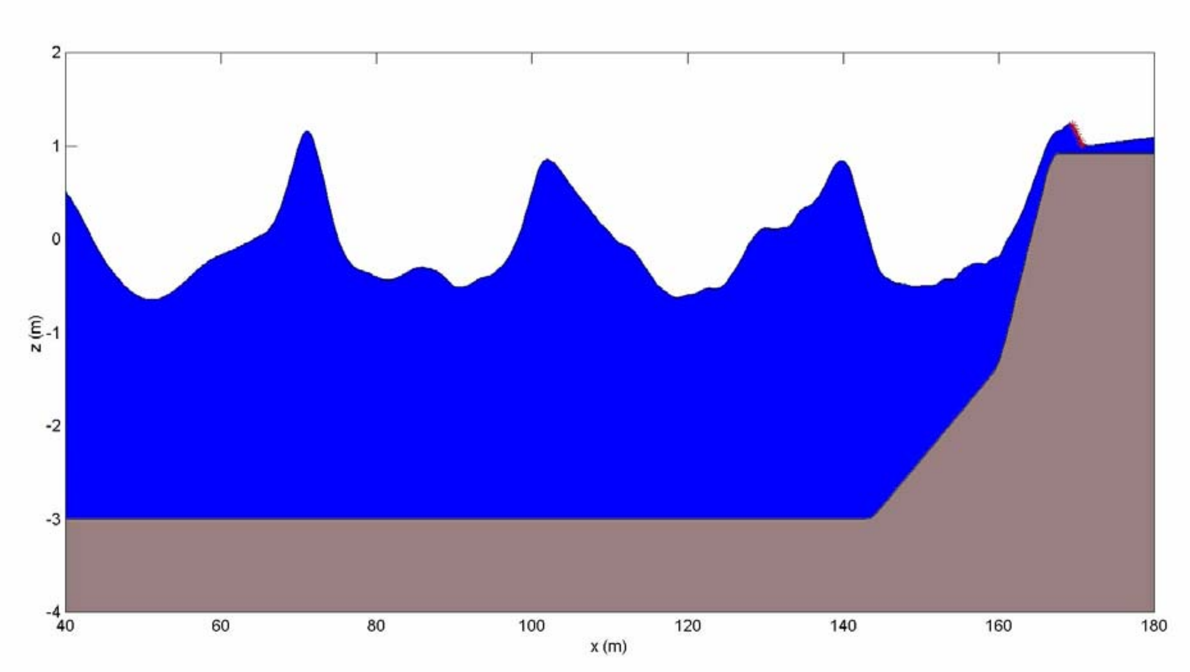

Figure 5. COULWAVE snapshot from a recreation of the Saville (1955) experiments. The general setup is a wavemaker depth ~3m, a flat portion leading up to a 1/10 slope, which connects to the "structure." In these experiments, the structure has either a 1/3 or 1/1.5 slope.

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 still water 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

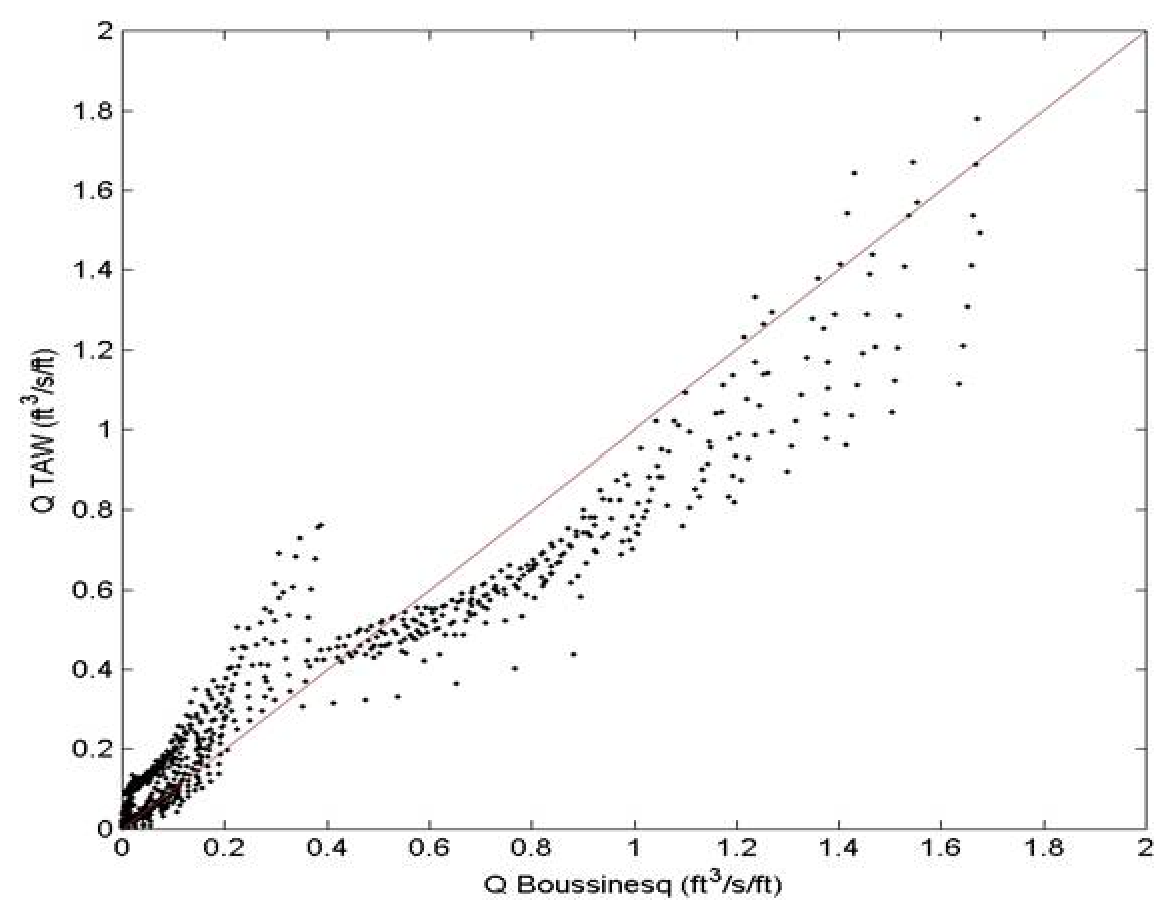

Figure 6. Comparison of Boussinesq overtopping rates with the formula given in the TAW design guidance.

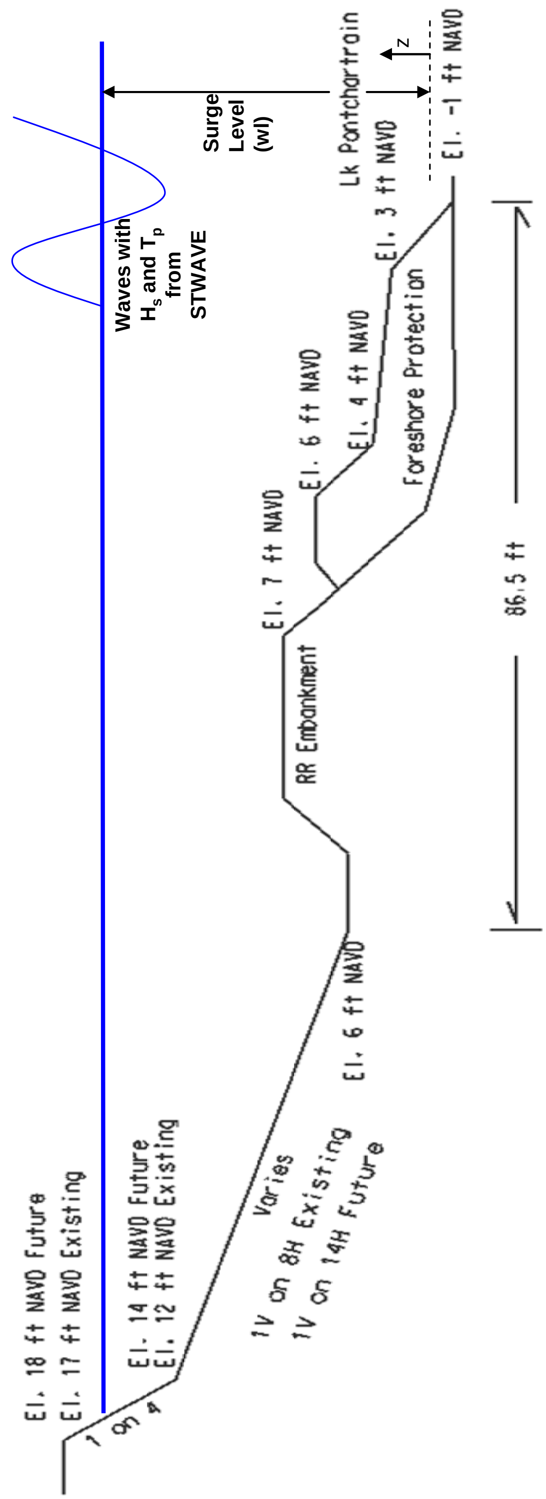

Figure 7. New Orleans East Lakefront Levee typical section

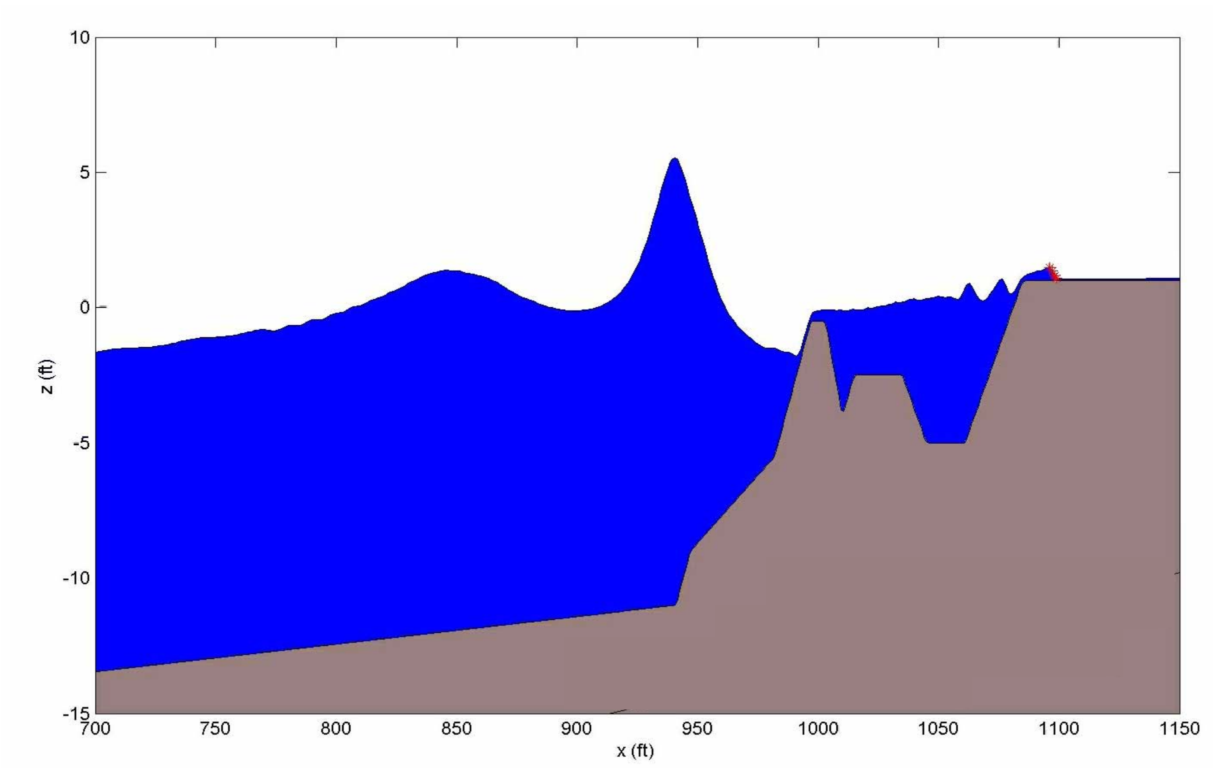

Figure 8. Snapshot from Boussinesq simulation of waves propagating across a reach with foreshore protection.

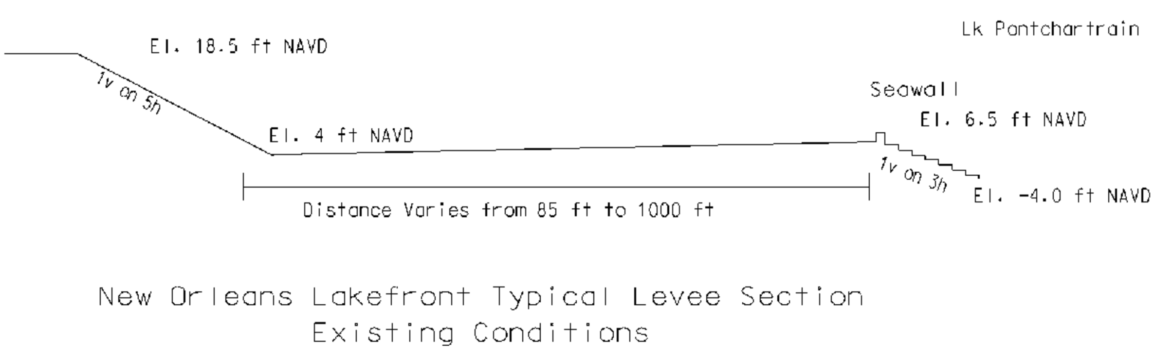

Figure 9. New Orleans Lakefront Levee typical section

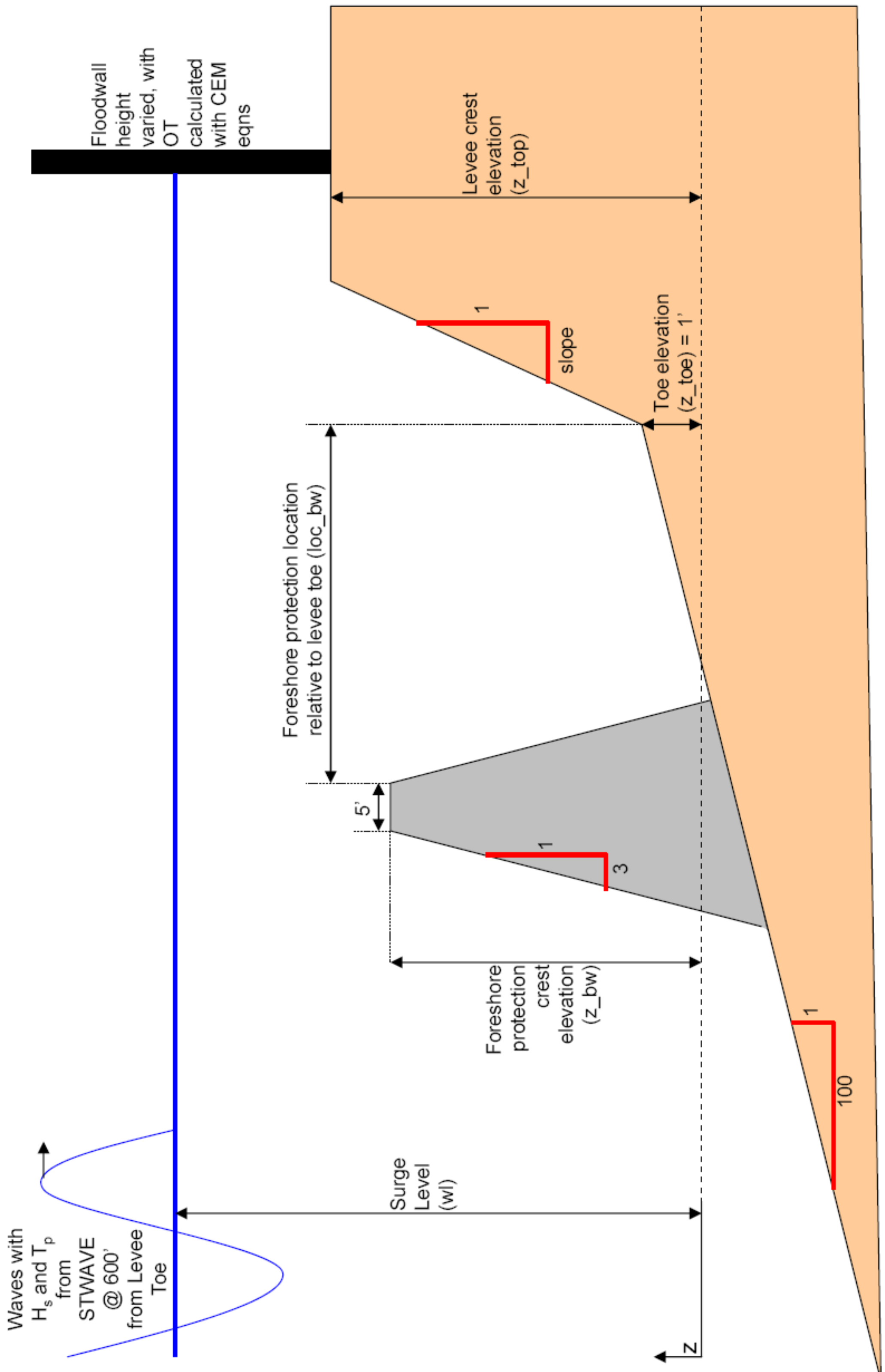

Figure 10. Schematic for generic reach lookup.

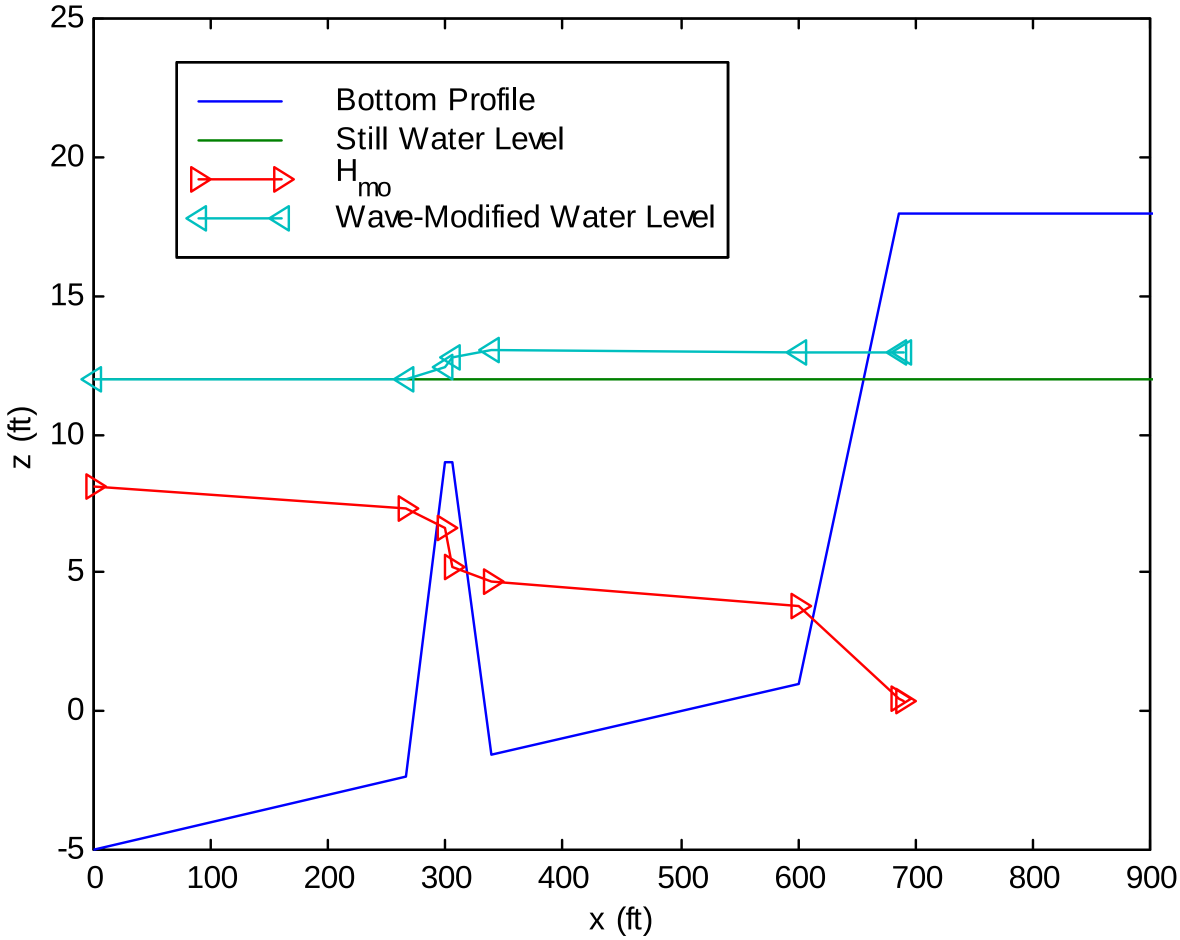

Figure 11. Example Matlab output plot from the generic lookup tool.

9.3 *Appendix C – 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 (e.g. Shore Protection Manual, 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' *Shore Protection Manual* (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 can not be predicted by extrapolating annual wind speed distributions. Batts, et. al. estimated hurricane winds indirectly from statistical distributions of hurricane climatological 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 speeds 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 E-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 *Shore Protection Manual*, 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: $1.277 + 0.296 \tanh (0.9 \log_{10} 45/t) = 1.30$

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.

9.4 Appendix D – 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).

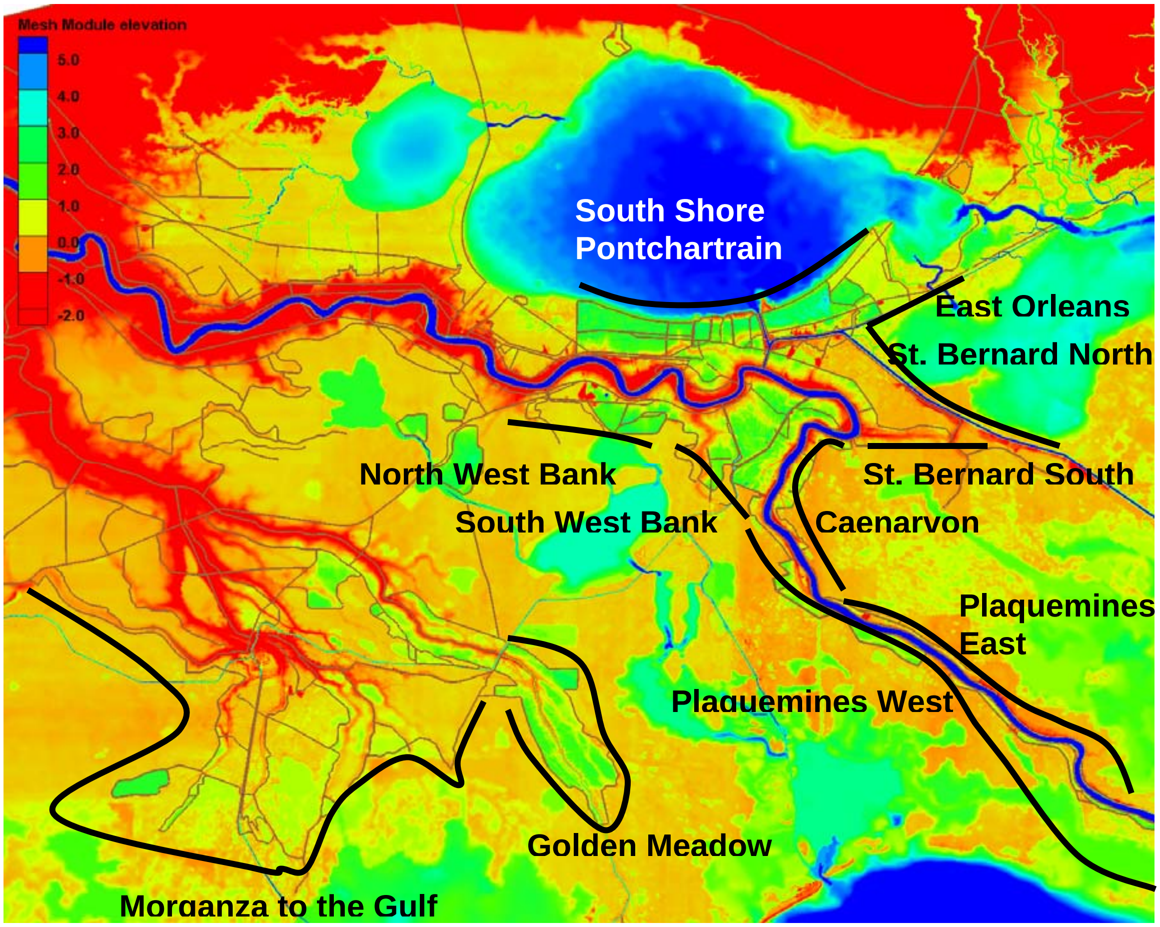

Figure 1. Reach Definitions overlaid on the ADCIRC gird (depths in meters).

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.

<u>Back Levees of East Orleans and St. Bernard North</u>. 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.

<u>St. Bernard South and Caenarvon</u>. 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.

<u>Plaquemines East and West</u>. 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.

<u>West Bank</u>. 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.

<u>Golden Meadow and Morganza to the Gulf</u>. 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.

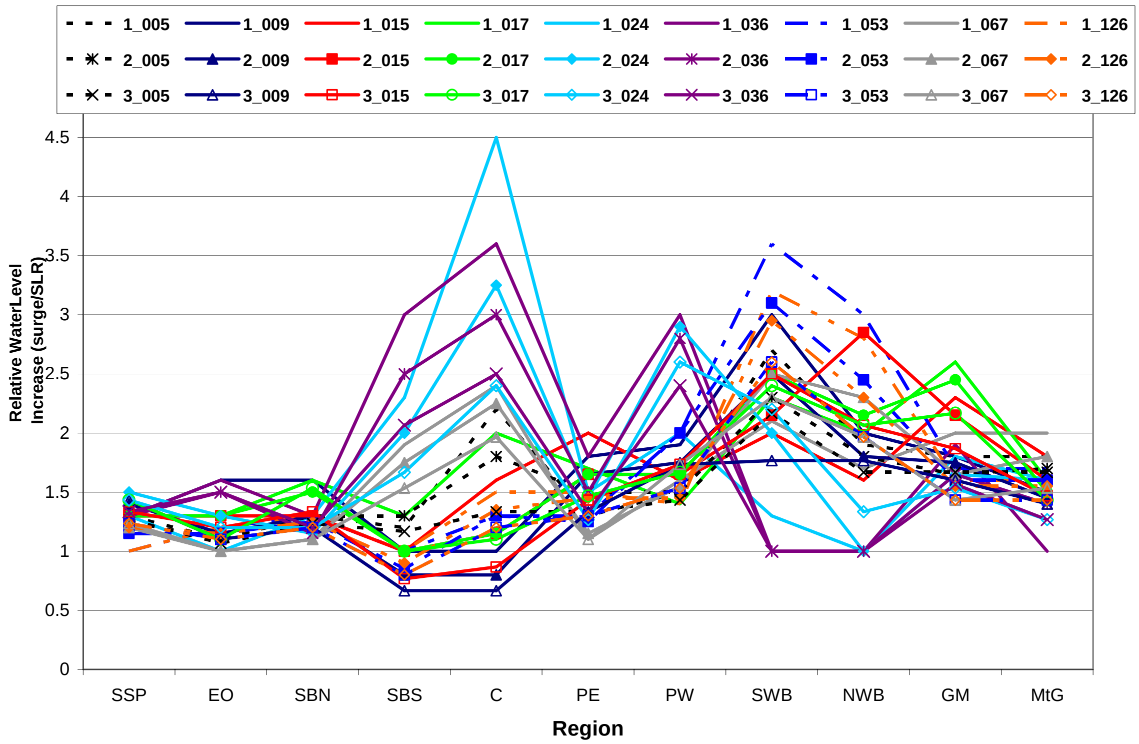

Figure 2. Relative Water Level Increases by Reach (legend provides the sea level rise (1, 2, and 3 ft) and storm number).

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.

<u>Plaquemines East and West</u>. 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.

<u>West Bank</u>. 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.

<u>Golden Meadow and Morganza to the Gulf</u>. 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

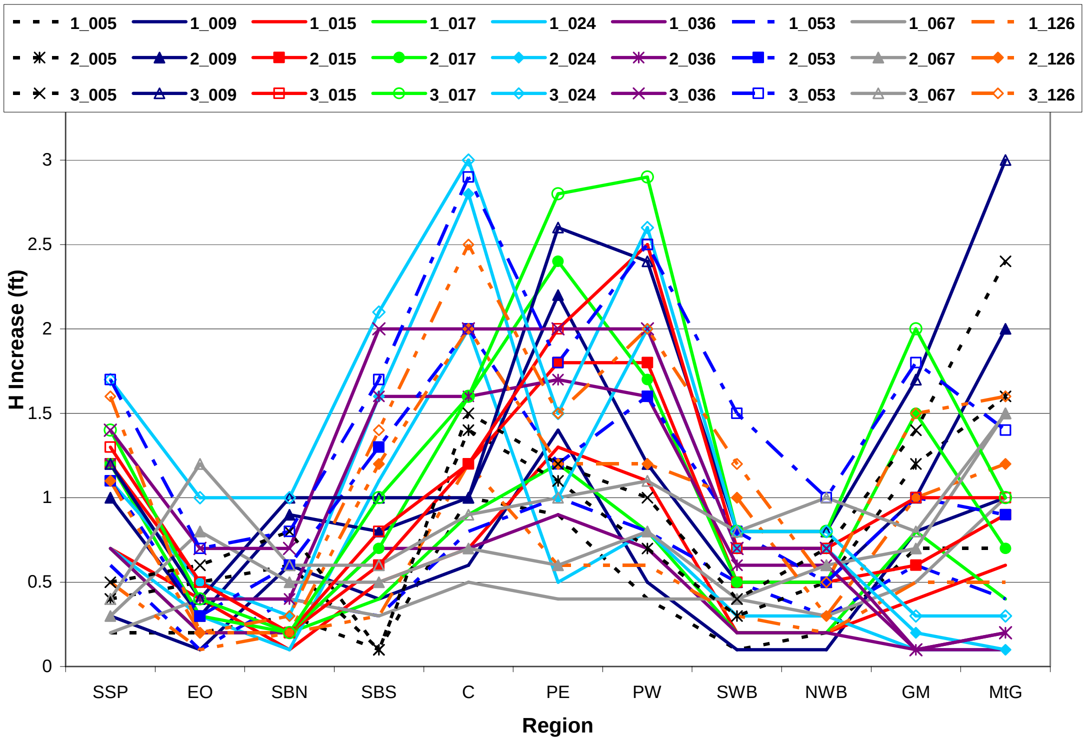

Figure 3. Wave height increase (feet).

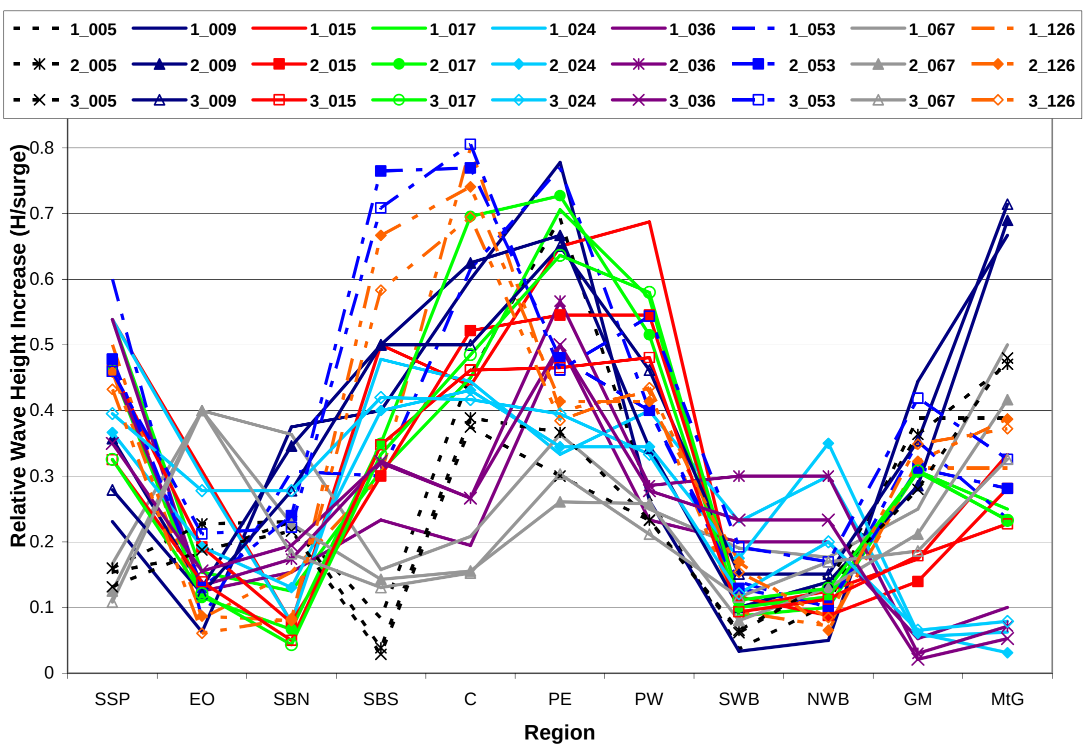

Figure 4. Normalized wave height increase (H increase/water level increase). Recommended Wave Height Increases.

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**

9.5 Appendix E – Overtopping criterion (Author: S.A. Hughes, ERDC)

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 still water 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.

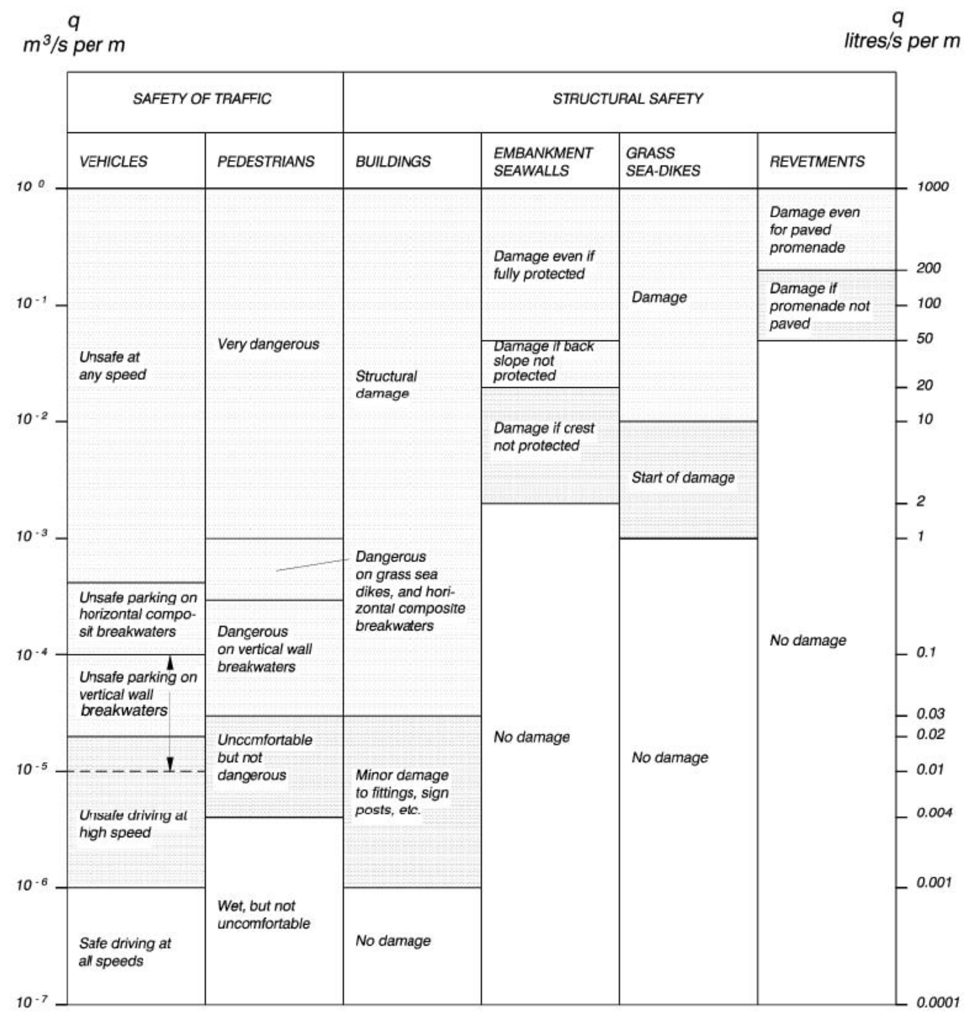

Figure 1. Table of permissible overtopping from the Coastal Engineering Manual

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.

<u>Dutch Criterion for Grass Sea Dikes</u>

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 judgment, 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 still water level (swl) is lower than the levee crest elevation, and where the swl 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 <u>engineering judgment</u>. 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).

<u>**Goda's Criteria for Embankments and Seawalls**</u>

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: over breakage 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 still water 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 still water 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.

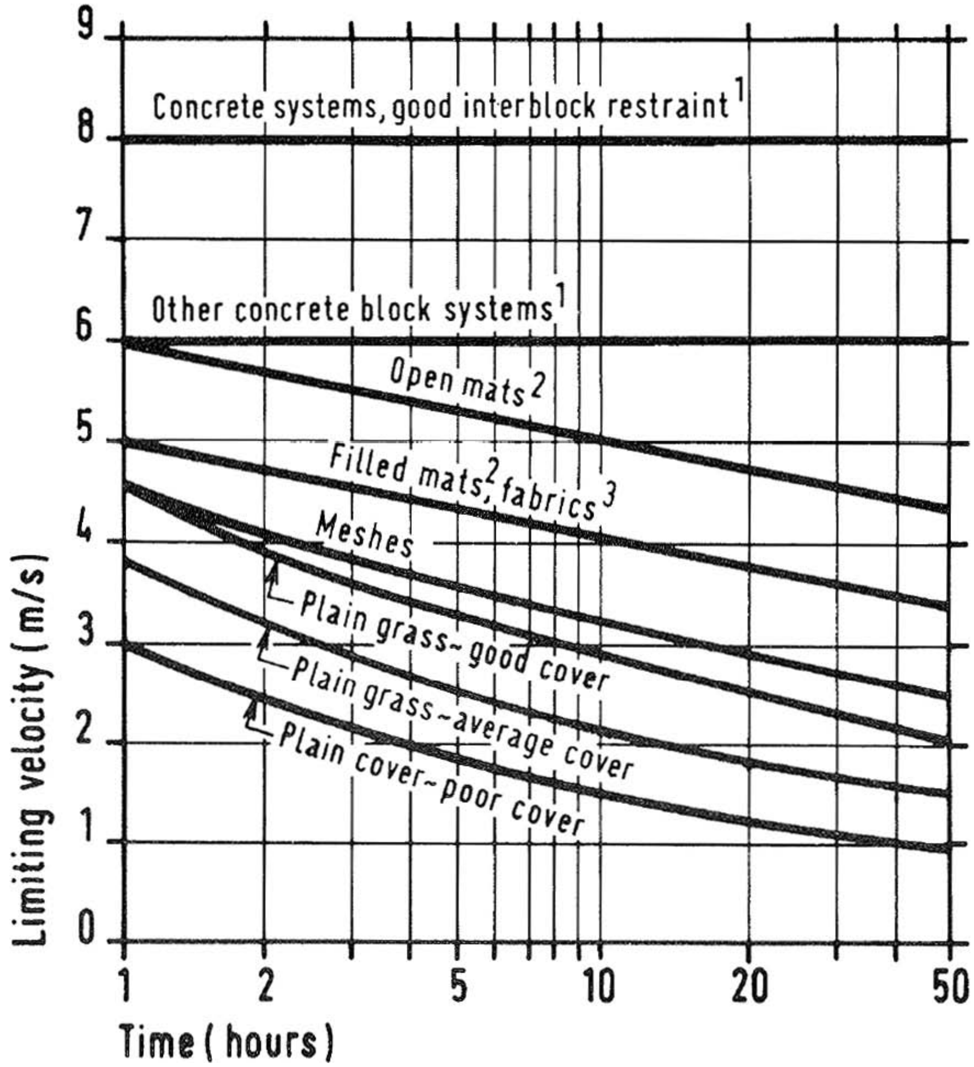

Figure 2. Erosion resistance of plain grass to steady overtopping (Hewlett, et al. 1987)

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).

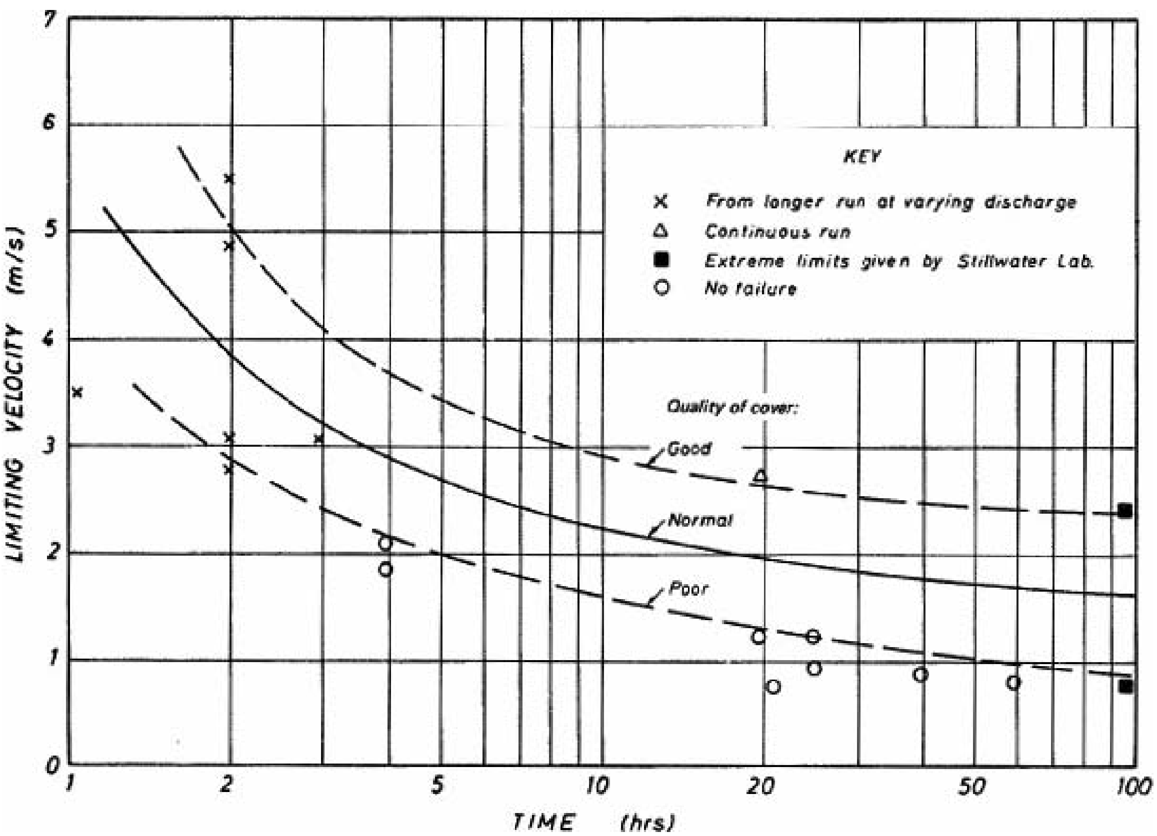

Figure 3. Erosion resistance of grass-lined spillways (Whitehead, et al. 1976)

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.

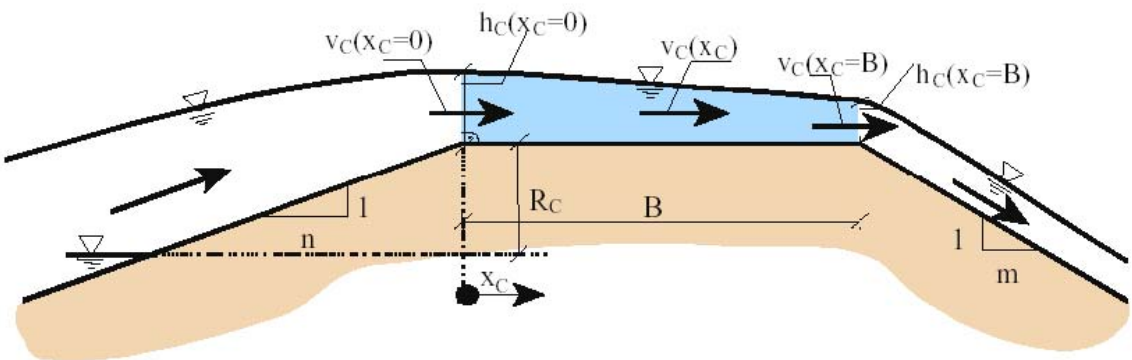

Figure 5. Wave overtopping definition sketch (from Schüttrumpf and Oumeraci 2005)

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

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

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 still water 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% exceedence 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

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

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% exceedence 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

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

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

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

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 still water 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.

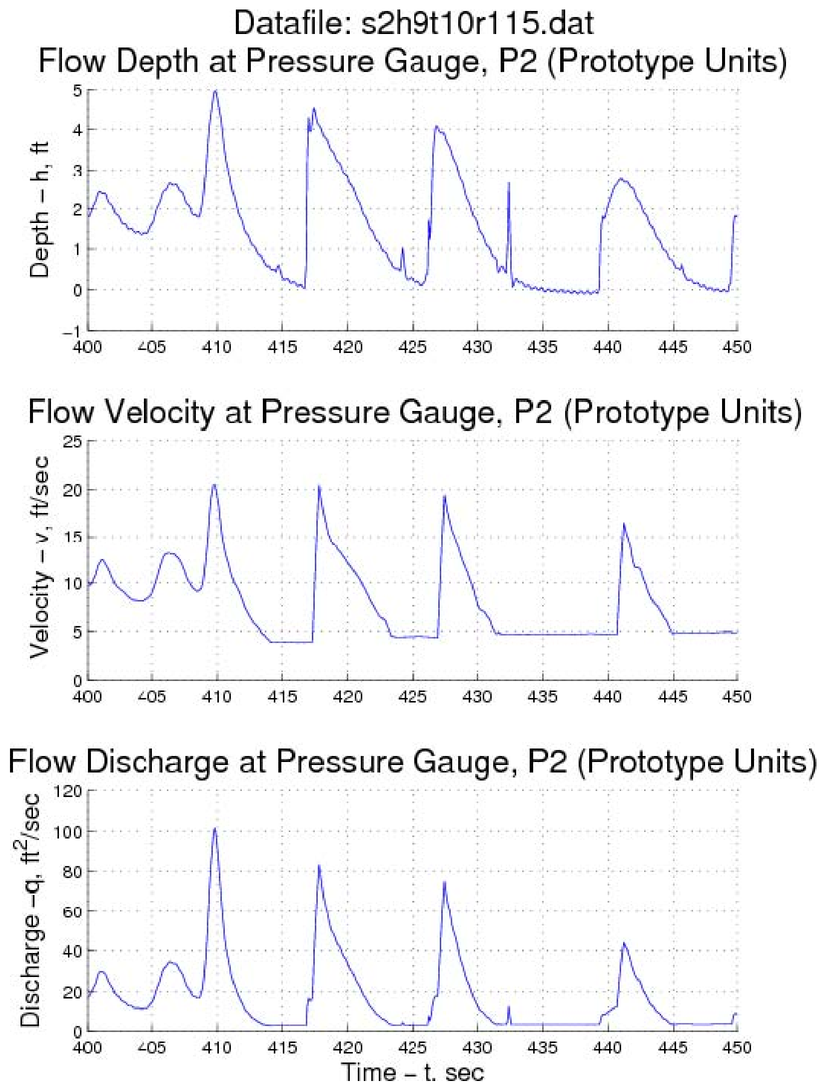

Figure 6. Laboratory measurements of waves overtopping a levee

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.

<u>Estimation of an Appropriate Friction Factor</u>

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.

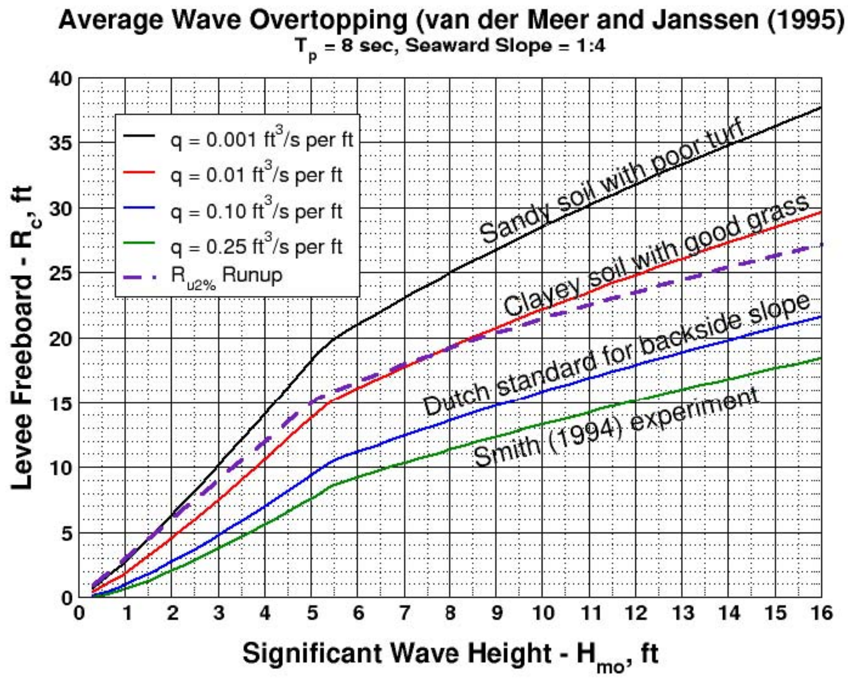

Figure 7. Average wave overtopping for 8-second peak period waves

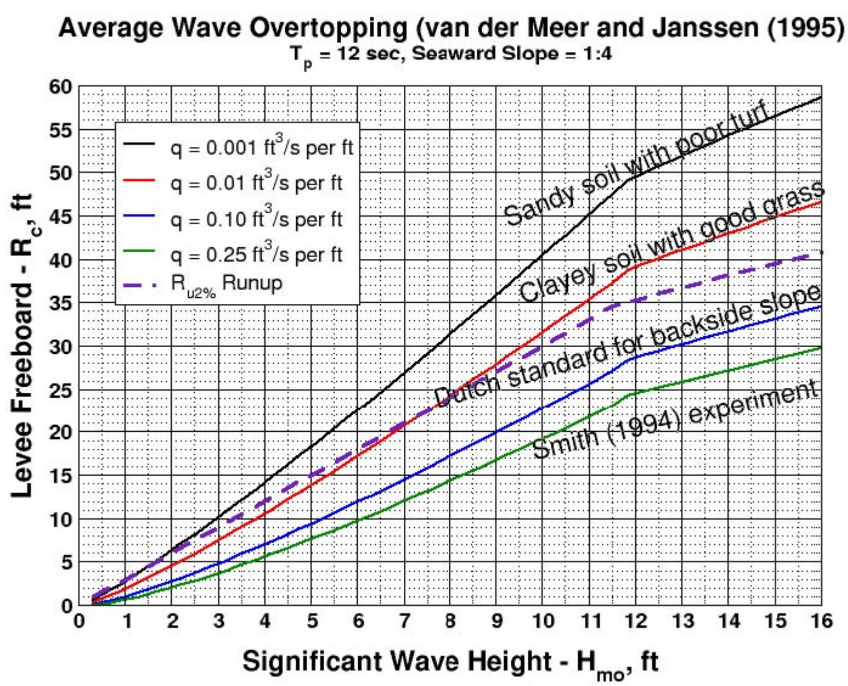

Figure 8. Average wave overtopping for 12-second peak period waves

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.

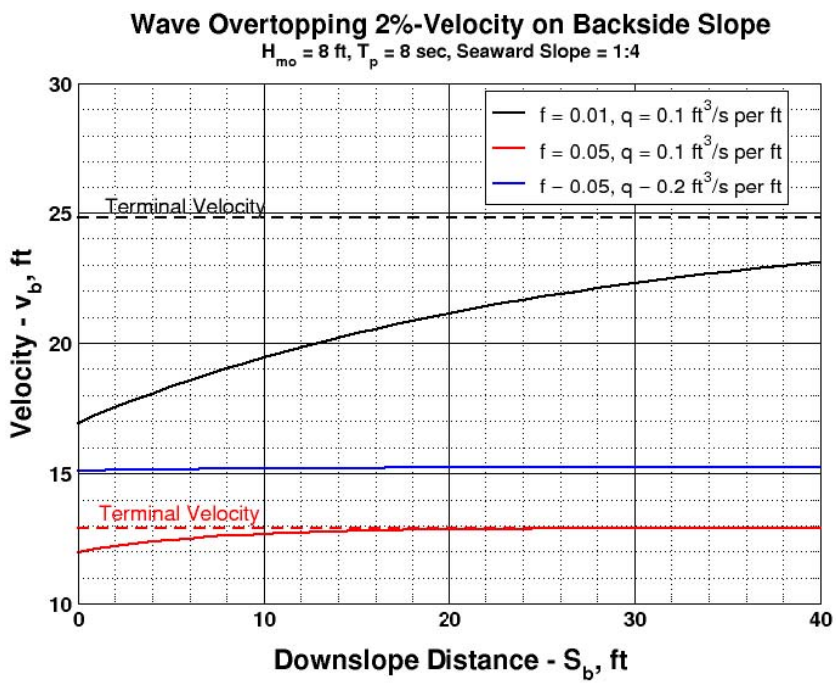

Figure 9. Peak velocity on levee protected-side slope exceeded by 2% of the waves

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 still water 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 exceedence 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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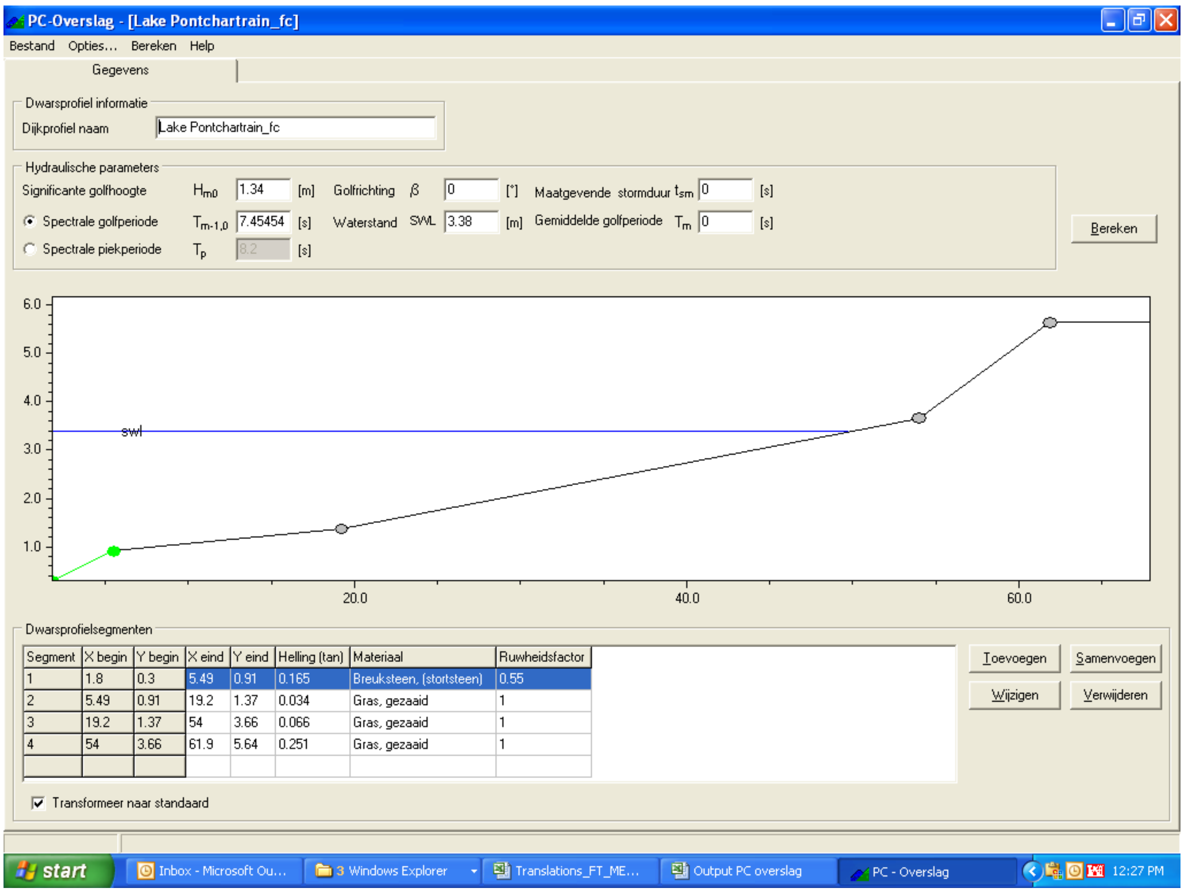
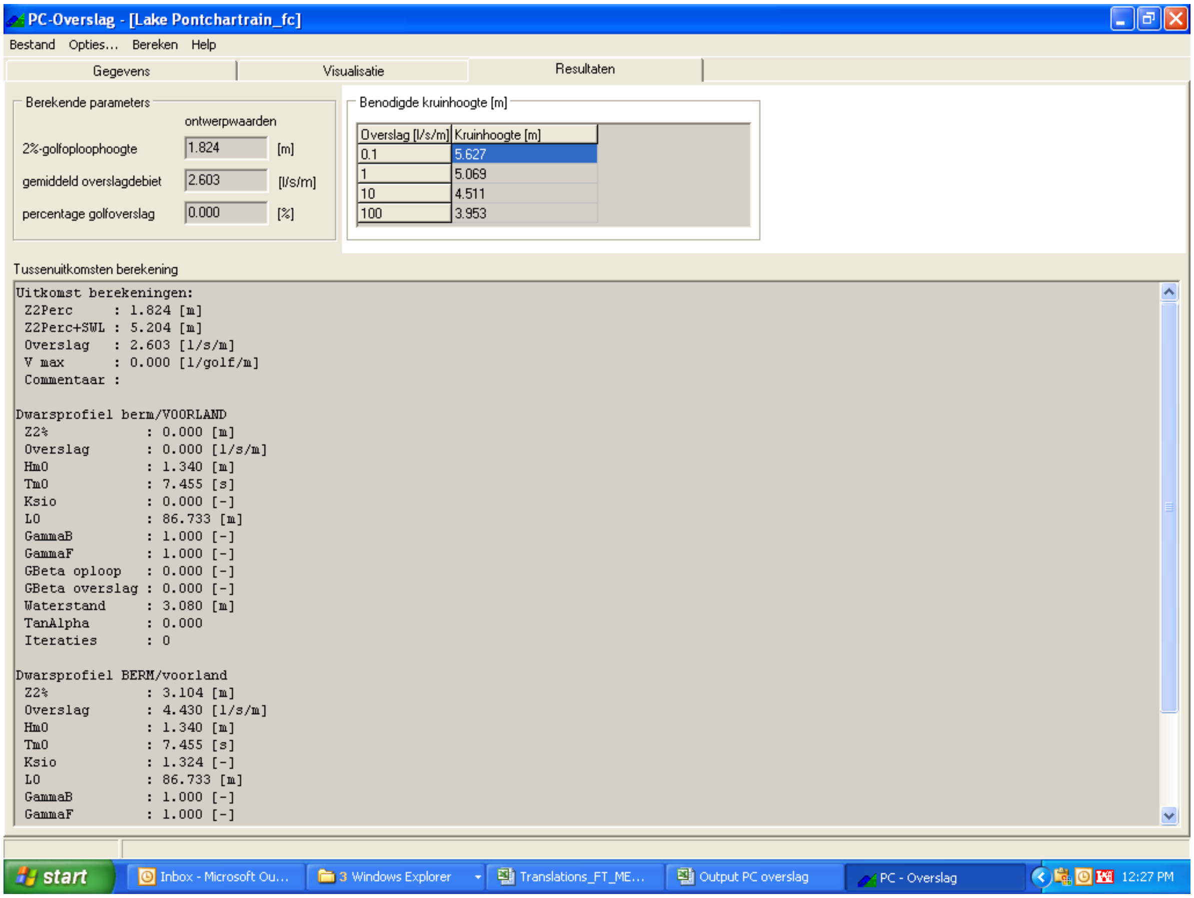
9.6 *Appendix F – Sample Design Calculations*

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, still water elevation and levee geometry. Note that units are metric and the language is Dutch.



The screen dump 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.



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/BW
Floodwall Elevations
Eq. Franco and Franco (1999)

g	32.19	cft/s2		Note: Add 2 feet to Wall height for uncertainties, so Top of Floodwall	
ztop	12.50	ft	Crest height		
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)		

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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.

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

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
$h_b/h_s = 0.21$

Disclaimer:
This message is not intended to provide construction, engineering or architectural advice. If such advice is required, it should be obtained in the form of complete plans and drawings. Unless complete drawings and plans are prepared and contracted for that enable construction, Haskoning Inc. does not guarantee the accuracy, completeness, efficacy, timeliness or correct sequencing of any information contained herein. Haskoning Inc.'s advice is subject to further review and this is not final until a written recommendation is rendered indicating final advice.

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9.7 Appendix G – 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 600ft 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-wall 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 J.2 to J.5. This appendix closes with a discussion of these results in Section J.6.

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 4ft 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.

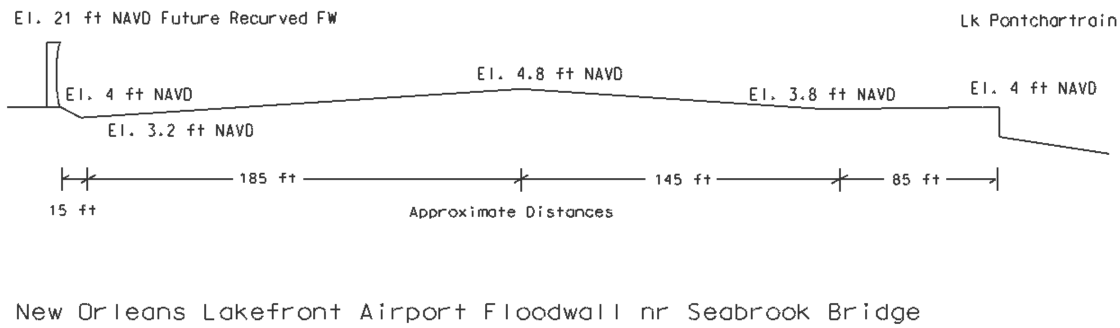


Figure 1 Cross-section Lakefront Airport Floodwall.

	Empirical approach	Boussinesq run
Still water 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	14ft	See Figure (flood wall 14ft)
Composite slope	-	
Berm coefficient	-	

Table 1 1% design values Lakefront Airport floodwall (mean values / standard deviation).

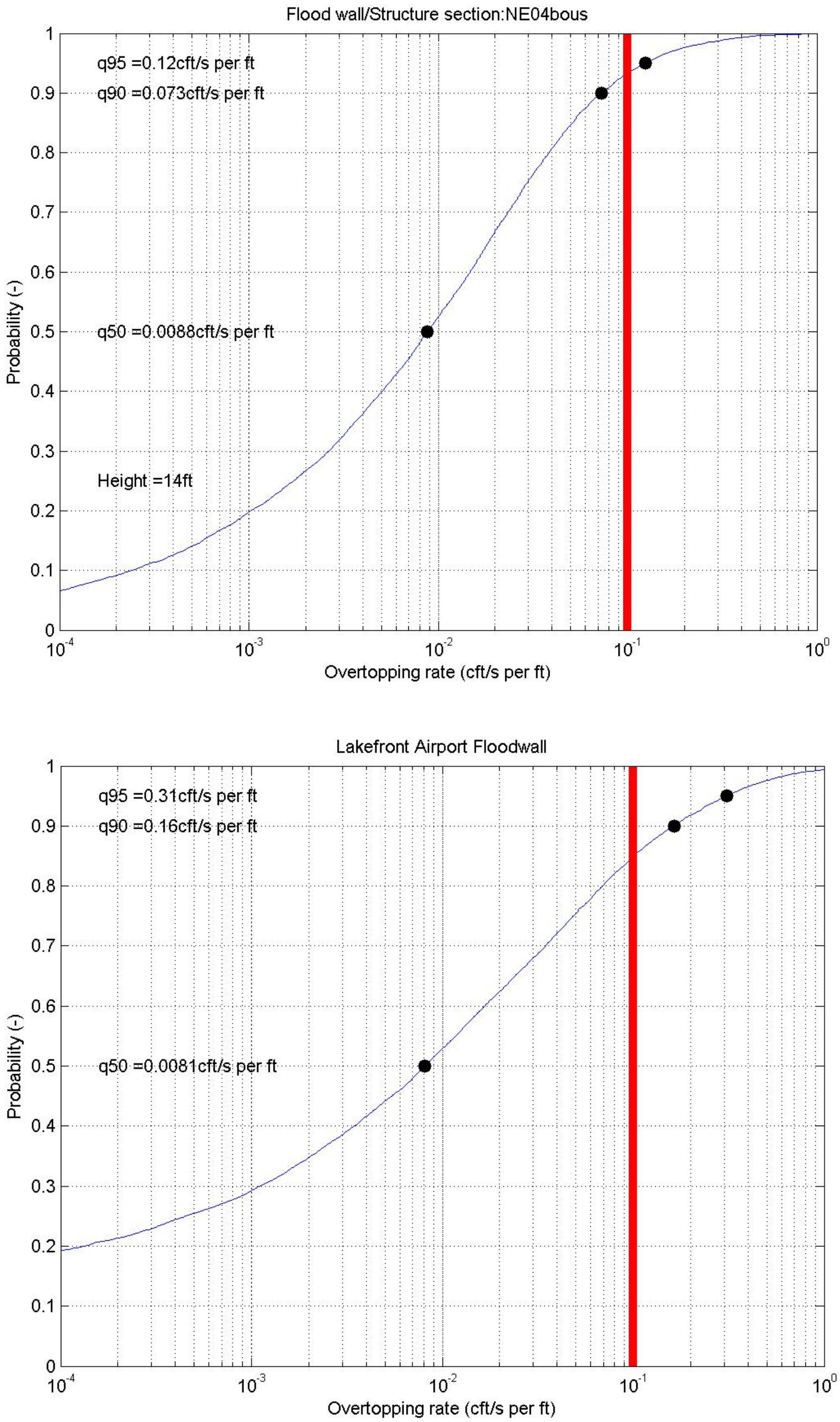


Figure 2 Result from MCS using the empirical formulations from the TAW manual (upper panel) and using the Boussinesq results (lower panel) for Lakefront Airport Floodwall.

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 ft. The new values used are summarized in Table 2. The results of the MCS are presented in Figure 4.

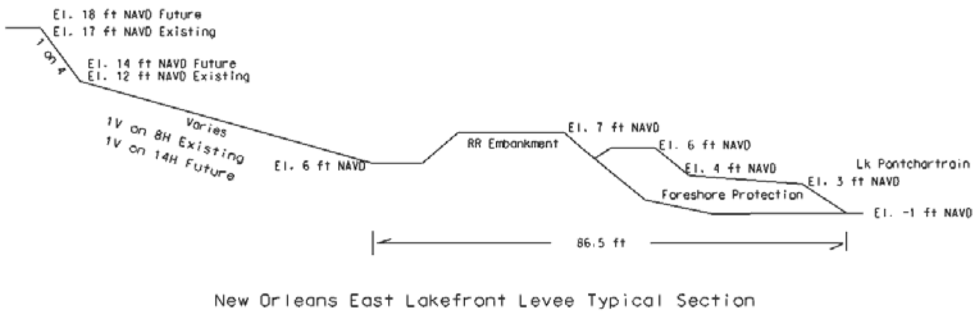
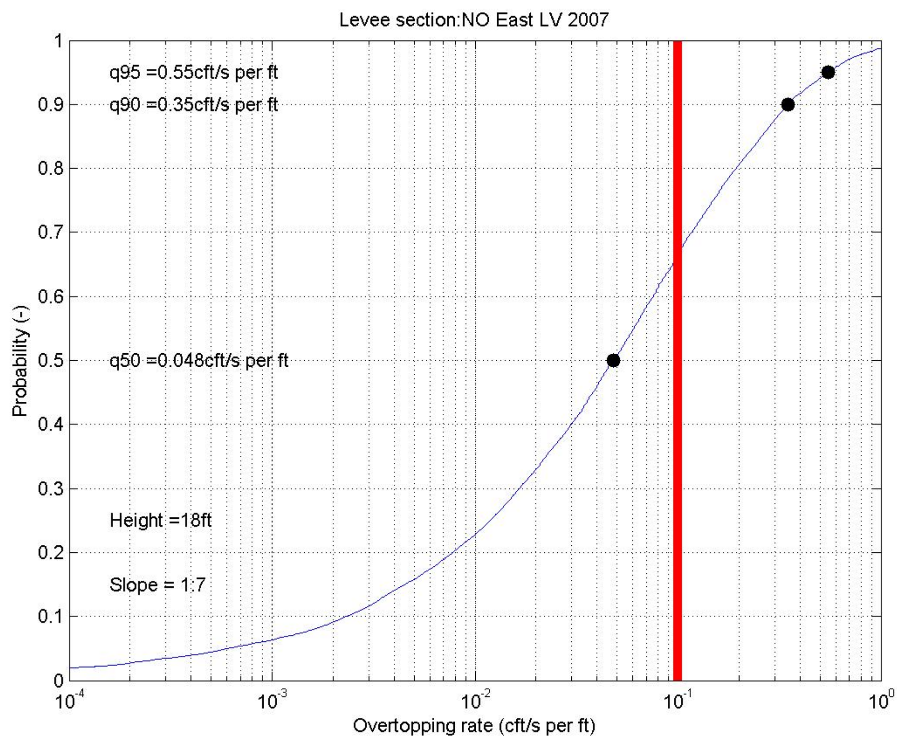


Figure 3 Cross-section New Orleans East Lakefront Levee.

	Empirical approach	Boussinesq run
Still water 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	18.0ft	See Figure (future conditions)
Composite slope	1/7	
Berm coefficient	0.7	

Table 2 1% design values New Orleans East Lakefront Levee.



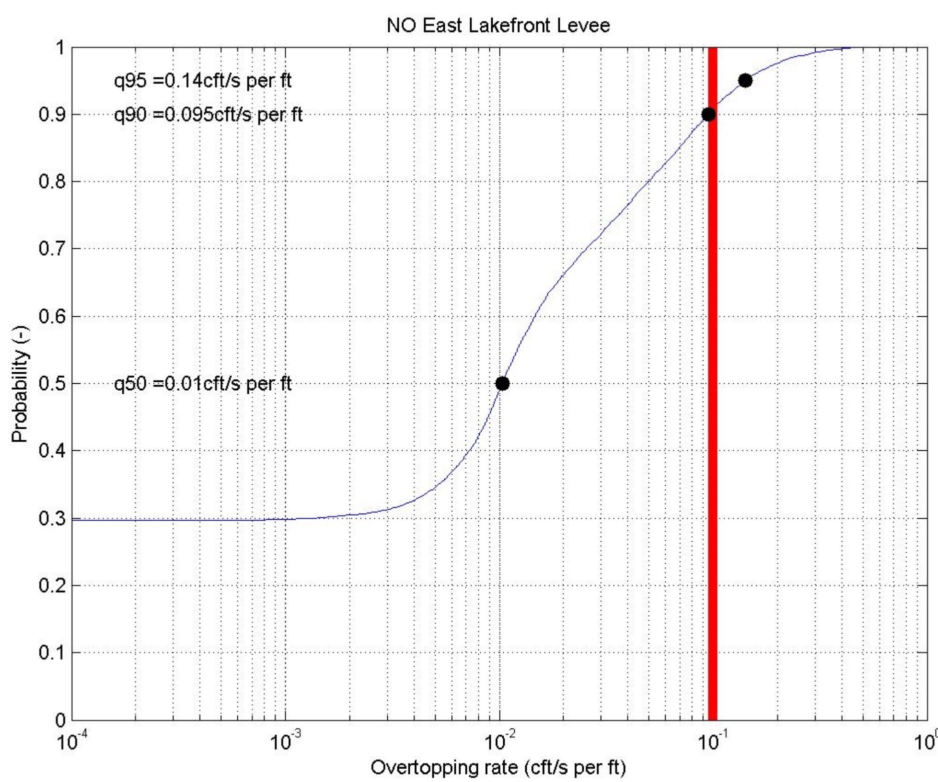


Figure 4 Result from MCS using the empirical formulations from the TAW manual (upper panel) and using the Boussinesq results (lower panel) for New Orleans East Lakefront Levee.

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.

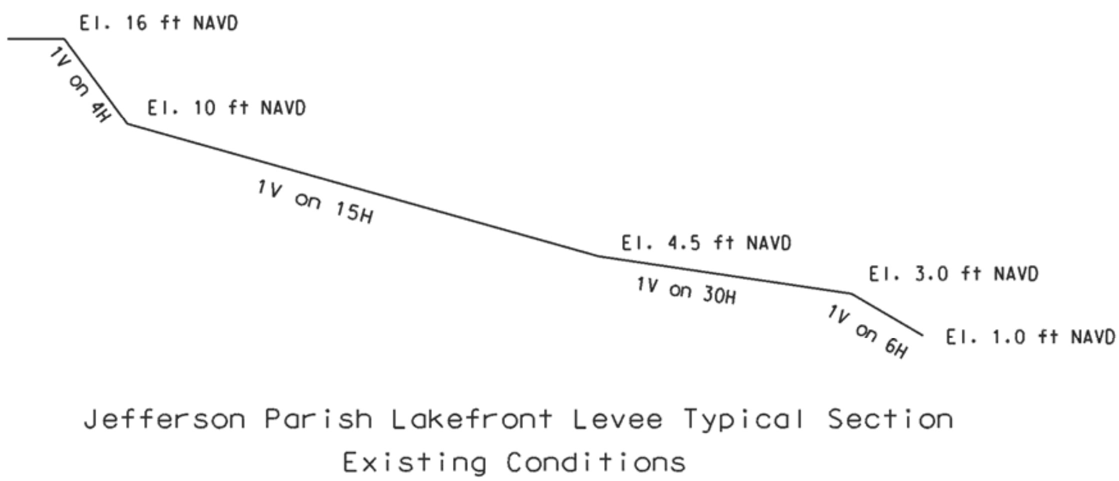


Figure 5 Cross-section Jefferson Parish Lakefront.

	Empirical approach	Boussinesq run
Still water 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	16ft	See Figure 5
Composite slope	1/4	
Berm coefficient	0.65	

Table 3 1% design values Jefferson Parish Lakefront

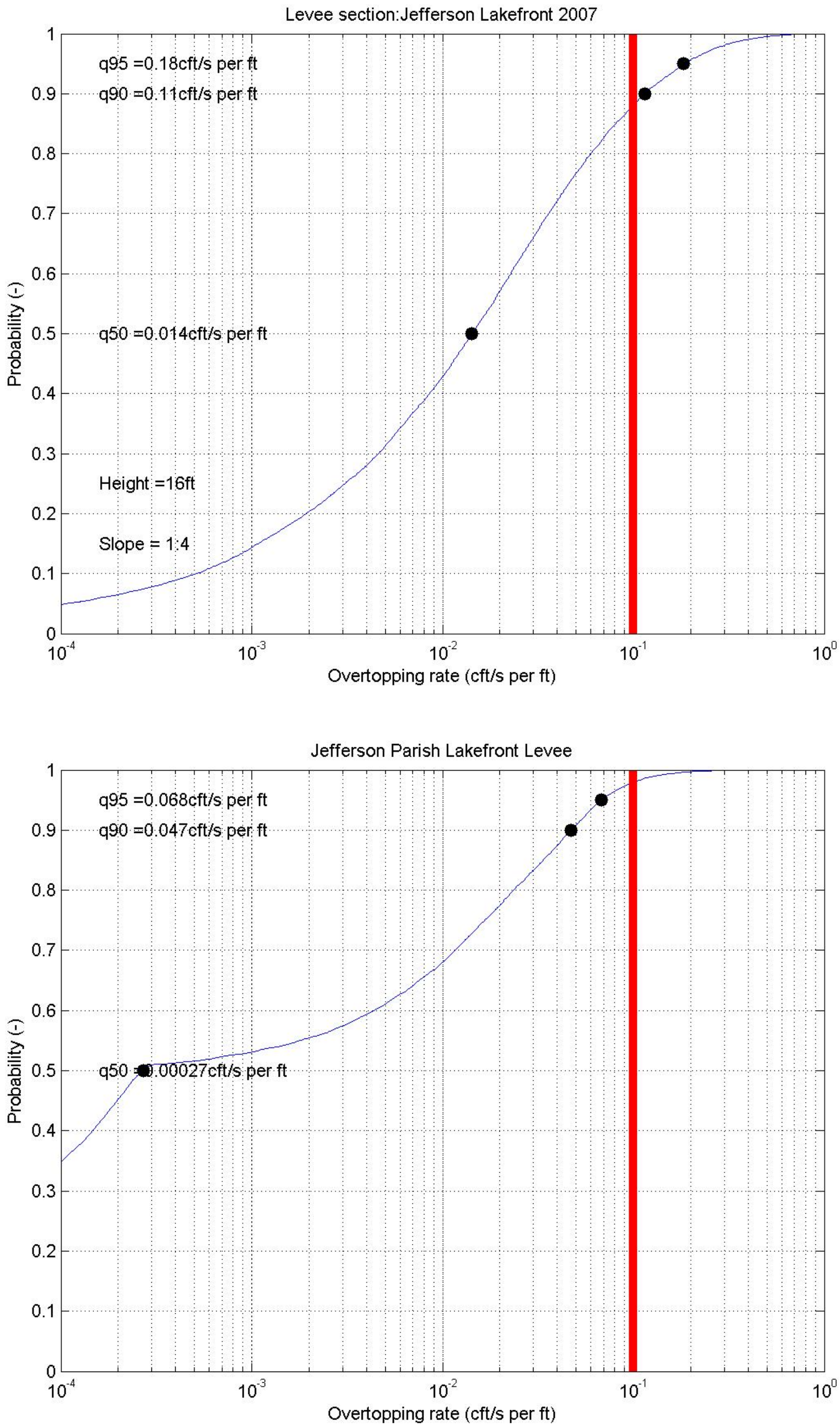


Figure 6 Result from MCS using the empirical formulations from the TAW manual (upper panel) and using the Boussinesq results (lower panel).

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 85ft. The 1% design values for the existing conditions (2007) are directly applied except for the still water level (Table). The still water 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.

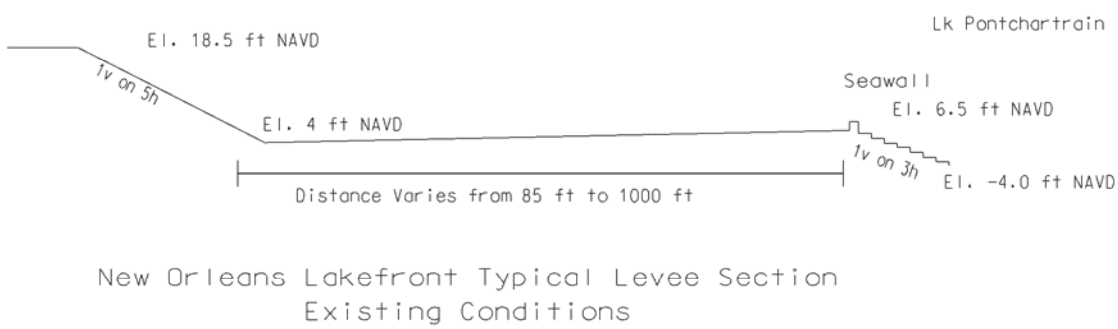
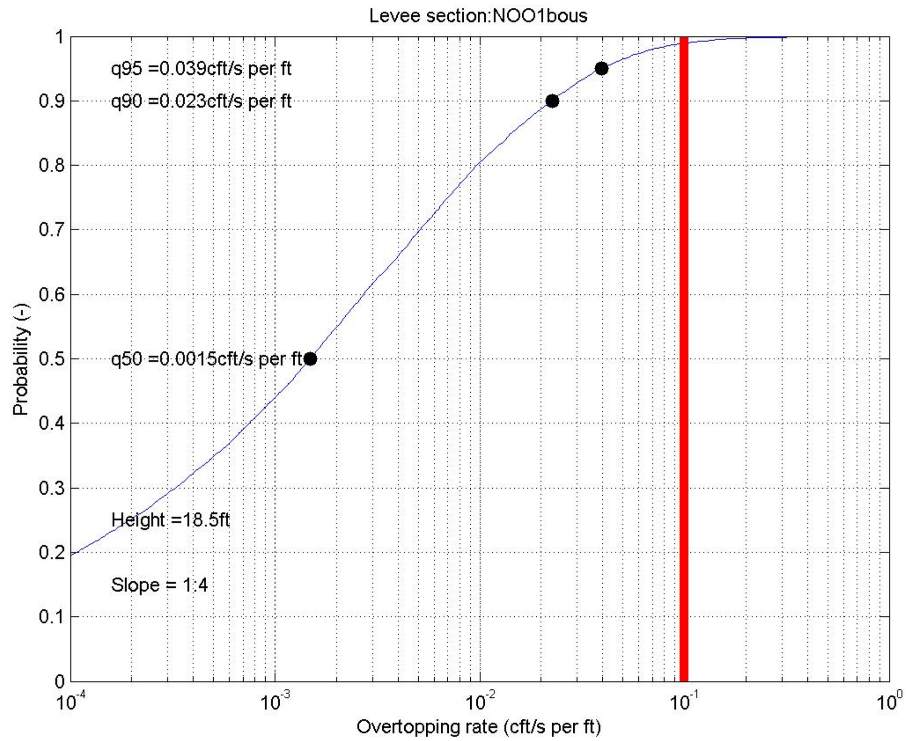


Figure 7 Cross-section New Orleans Lakefront (the applied berm length is 85ft).

	Empirical approach	Boussinesq run
Still water 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	18.5ft	See Figure
Composite slope	1/5	
Berm coefficient	0.6	

Table 4 1% design values New Orleans Lakefront Levee.



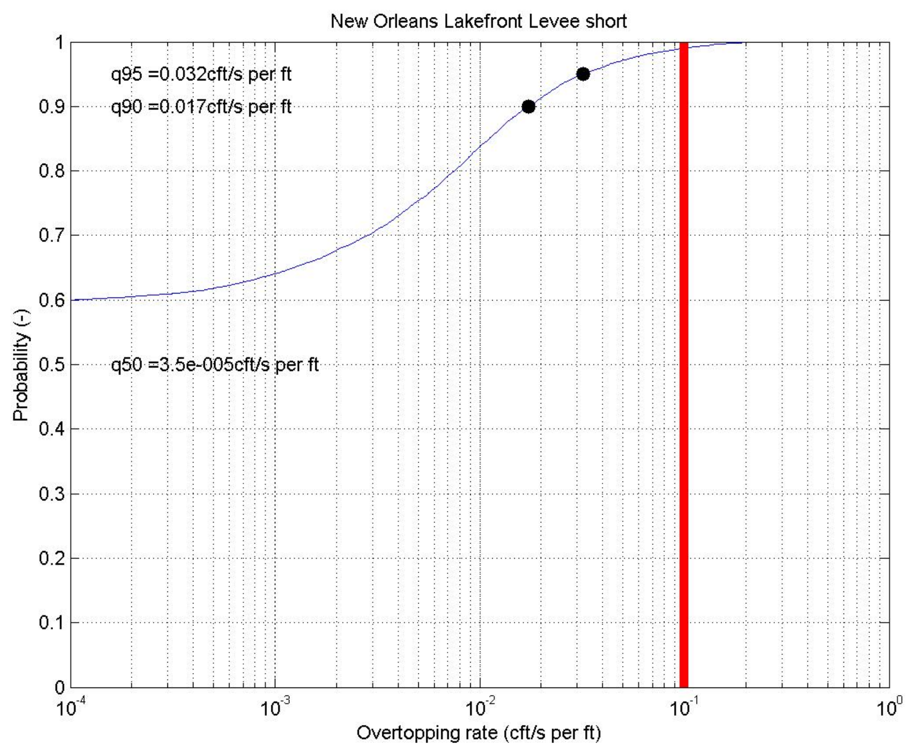


Figure 8 Result from MCS with empirical approach (upper panel) and Boussinesq approach (lower panel).

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:

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

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 - 7ft 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 1ft at maximum.

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

The equation governing average overtopping rate is:

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

which can be differentiated with respect to R_c and rearranged to

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

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

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

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).