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Alabama Barrier Island Restoration Assessment Life-Cycle Structure Response Modeling

Victor M. Gonzalez, Fabian A. Garcia-Moreno, Jeffrey A. Melby,
Norberto C. Nadal-Caraballo, and Elizabeth S. Godsey

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Abstract

Dauphin Island, a barrier island off the coast of Alabama, plays an important role in the protection of the state's coastal natural resources. In 2011, the State of Alabama constructed a rubble mound berm across a 2 km breach in the western end of the barrier island to prevent oil from the Deep Water Horizon oil spill from migrating into the Mississippi Sound. The breach, referred to as the Katrina Cut, was caused by Hurricane Katrina in 2005. The US Army Corps of Engineers and the US Geological Survey (USGS) performed the Alabama Barrier Island Restoration Assessment study to assess the current and future function of Dauphin Island and evaluate potential restoration measures. A Monte Carlo life-cycle structure response assessment of the Katrina Cut rubble mound structure was performed as part of this study by the US Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory. Damage, wave transmission, and reliability were computed within the context of the decadal barrier evolution analysis performed by the USGS for various storminess and relative sea level change scenarios. The presence of a beach in front of the structure plays an important role in its protection. The breaching potential for measures was evaluated.

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Preface

This study was conducted for the US Army Corps of Engineers, Mobile District, under Project No. 452808. The Program Manager for the study was Mr. Justin S. McDonald.

The work was performed by the Harbors, Entrances, and Structures Branch of the Navigation Division, US Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory (ERDC-CHL). At the time of publication of this report, Mr. James D. Gutshall was Chief, Harbors, Entrances, and Structures Branch; Dr. Jacqueline S. Pettway was Chief, Navigation Division; and Mr. Charles E. Wiggins was the Technical Director for Navigation. The Deputy Director of ERDC-CHL was Mr. Jeffrey R. Eckstein, and the Director was Dr. Ty V. Wamsley.

Many of the methods and tools applied in this study were developed as part of the Civil Works Monitoring Completed Navigation Projects (MCNP) Work Unit Reliability of Rubble Mound Structures, of which Dr. Lyndell Z. Hales is Program Manager.

The Commander of ERDC was COL Teresa A. Schlosser, and the Director was Dr. David W. Pittman.

1 Introduction

Dauphin Island, a barrier island off the coast of Alabama, plays an important role in the protection of the state's coastal natural resources. During an 8-year span from 2004 to 2012, the island experienced significant tropical storm events and a man-made disaster that affected the morphology and ecology of the island. These included Hurricanes Ivan, Katrina, and Isaac, as well as the Deep Water Horizon oil spill. One of the most significant impacts was a 2 km^{1,2} breach in the barrier island created by Hurricane Katrina in 2005. As a response to the 2010 Deep Water Horizon oil spill, a temporary rubble mound structure was constructed across the cut to prevent oil moving into the Mississippi Sound. This technical report presents the results of the Katrina Cut rubble mound structure response modeling that was part of the wider Alabama Barrier Island Restoration Assessment Study sponsored by the National Fish and Wildlife Foundation and the State of Alabama. The structure response modeling was performed by the Coastal and Hydraulics Laboratory, US Army Engineer Research and Development Center. Structure damage and reliability estimates were computed for different storminess and sea level change scenarios for select restoration measures. Wave transmission due to overtopping was also evaluated.

1.1 Background

The passage of Hurricane Katrina west of Dauphin Island, AL produced a 2 km wide breach in the unpopulated western end of the barrier island. This breach is referred to as the Katrina Cut. Breaching and natural closing of the barrier island have been documented in the past, with closures of large breaches occurring over decadal time scales (USGS et al. 2017). Nevertheless, to protect the Mississippi Sound following the 2010 Deep Water Horizon oil spill, the Katrina Cut was closed with a rubble mound structure. Figure 1-1 shows the changes at the Katrina Cut area from the

¹ For a full list of the spelled-out forms of the units of measure used in this document, please refer to US Government Publishing Office Style Manual, 31st ed. (Washington, DC: US Government Publishing Office 2016), 248-52, <https://www.govinfo.gov/content/pkg/GPO-STYLEMANUAL-2016/pdf/GPO-STYLEMANUAL-2016.pdf>.

² For a full list of the unit conversions used in this document, please refer to US Government Publishing Office Style Manual, 31st ed. (Washington, DC: US Government Publishing Office 2016), 345-7, <https://www.govinfo.gov/content/pkg/GPO-STYLEMANUAL-2016/pdf/GPO-STYLEMANUAL-2016.pdf>.

year 2000 to the year 2019, including structure construction. The 2005 image predates Hurricane Katrina and shows a smaller breach caused by Hurricane Ivan the year prior. In the years following the closure of the Katrina Cut, a beach formed on the Gulf of Mexico (south) side of the structure. The State of Alabama engaged the US Geological Survey (USGS) and the US Army Corps of Engineers (USACE) to perform a study, through a grant from the National Fish and Wildlife Foundation, Gulf Environmental Benefit Fund, to investigate feasible alternatives for the sustainable restoration of Dauphin Island for habitat and species impacted by the 2010 Deepwater Horizon oil spill.

Figure 1-1. Aerial images of Katrina Cut area in Dauphin Island from 2000 to 2019.

1.2 Objectives

The Alabama Barrier Island Restoration Assessment project's main objective was to assess the current and future function of Dauphin Island, develop options for restoration, and evaluate the feasibility and cost associated with restoration actions. The focus of the restoration actions was the optimization of the island's resiliency to storm events, the enhancement of wildlife habitat, and increase of sustainability. The Katrina Cut rubble mound structure was intended to be a temporary measure to prevent oil migration into the Mississippi Sound from the Deep Water Horizon oil spill. The structure is still present and therefore evaluated as part of the study.

The structural response modeling of the Katrina Cut structure was performed to determine damage, wave transmission, and reliability within the context of the decadal modeling performed for the restoration measures. Specific objectives of this study included the following:

- Characterization of the wave and water level forcing for the morphologic and the rubble mound structure response modeling for selected restoration measures.
- Evaluation of the performance of the Katrina Cut rubble mound structure for restoration measures for varying storminess conditions and relative sea level change (RSLC) scenarios.

1.3 Approach

A Monte Carlo life-cycle analysis of the rubble mound structure response was applied using a 10-year simulation period consistent with the decadal modeling approach used by the USGS in its barrier island evolution forecasting (Mickey et al. 2020). Synthetic storms developed for a regional joint probability method coastal hazard storm surge study were sampled to generate the forcing for the 10-year simulation period. Rubble mound response was evaluated for the range of probable beach response scenarios. The outcome of the rubble mound structure assessment will inform the costing of restoration measures and aid decision makers and stakeholders in determining the fate of the temporary rubble mound structure at the Katrina Cut.

2 Dauphin Island Katrina Cut Structure and Restoration Measures

A small breach, formed through the western portion of Dauphin Island, AL, after hurricane Ivan, was significantly widened to a 2 km wide breach by Hurricane Katrina the following year. This breach was closed in 2011 with a rubble mound berm by the State of Alabama in response to the 2010 Deep Water Horizon oil spill. The original objective was to build a temporary berm that would prevent oil migration from the spill into the Mississippi Sound (USGS et al. 2017).

2.1 Katrina Cut structure

The Katrina Cut structure was designed as a temporary structure. The permit for the structure includes drawings and specifications as well as the performance criteria on which the design was based. The typical cross section is shown in Figure 2-1. It consists of a 2:1 side-slope rubble mound structure with a sand core surrounded by a geotextile filter fabric. The top of the core was designed to an elevation of 1.52 m North American Vertical Datum of 1988 (NAVD88), the estimated wave run-up elevation under normal conditions. The definition of *normal* was not found, but the use of the term seems consistent with the intent of it being a temporary structure. The design crest height was 2.44 m NAVD88, with a 0.91 m thick armor top layer. The armor design specified the use of Class 5 riprap as defined in specification 814 of the 2008 Alabama Standard Specifications for Highway Construction. Class 5 stone is composed of nearly rectangular quarry stone with the size distribution shown in Table 2-1. The median size for this stone was 450 kg, which, according to the design, was estimated to provide sufficient protection against 1.37 m wave heights. The design section also included a 12 in. #57 choke stone layer on top. A choke stone is a small-sized stone used to stabilize the surface of an open-graded material. Based on field assessments of the structure conducted in September 2016, it is unclear if the choke stone was placed. It may have been placed and washed away after installation. This study assumes that the choke stone does not have any effect on the performance of the structure. The structure had a crest width of 6.10 m, with a toe elevation of -2.44 m at the deepest part of the breach.

Figure 2-1. Typical section design drawing for Katrina Cut structure.
(Permit No. SAM-2010-00822-JBE)

Table 2-1. Class 5 riprap gradation and corresponding nominal stone size.

Percent passing	Weight (kg)	D _n (m)
90%	900	0.7
50%	450	0.55
25%	100	0.33

The nominal median stone size (D_{n50}), also referred to as the equivalent cube length of the median stone, was computed assuming a specific armor weight, γ_s , of 2,643 kg/m³ using Equation 2-1. The computed median nominal stone size was 0.55 m.

$$D_{n50} = \left(\frac{W_{50}}{\gamma_s} \right)^{1/3} \quad (2-1)$$

An assessment of the aerial images shows that over time the structure has suffered damage at some locations since its construction. This was confirmed by a 2016 survey and inspection commissioned by the USACE Mobile District. The survey revealed multiple areas where the crest elevation had been lowered from the original design elevation. Figure 2-2 shows an aerial image with six damage areas identified in 2016.

Figure 2-3 presents a photograph of leeside damage at damage area 3. Gas and/or oil pipelines run underneath the west end of the Katrina Cut. Two of the alignments run underneath damage areas 1 and 2. The design of the structure contemplated the presence of pipelines and called for the installation of a structural geogrid under the berm at locations where they cross. Figure 2-4 shows a schematic of the location of the pipeline alignments, represented as yellow lines running perpendicular to the rubble mound structure.

Figure 2-2. Damage area in Katrina Cut rubble mound structure identified in 2016 survey.

Figure 2-3. Leaside damage at damage area 3.

Figure 2-4. Alignment of pipelines that cross underneath the Katrina Cut structure.

2.2 Restoration measures

The life-cycle structure response assessment was performed in the context of restoration measures proposed for Dauphin Island. Seven restoration measures and a no-action measure were implemented by the USGS for initialization of the barrier island response modeling over a 10-year timeframe. The description of the restoration measures is presented in Table 2-2. The information in this section summarizes the analysis of the restoration measures that were documented in Mickey et al. (2020).

Table 2-2. Description of restoration measures

Restoration Measure	Restoration Description
R0	No Action
R1	Katrina Cut – sand berm
R2	Pelican Island Southeast nourishment
R3	Sand Island platform nourishment and sand bypassing
R4	West and East End beach and dune nourishment
R5	Back Barrier tidal flats and marsh habitat restoration
R6	West End beach and dune nourishment
R7	West End and Katrina Cut beach and dune nourishment

Not all restoration measures impact the area of Katrina Cut. Measure R1 was a sensitivity run where the Katrina rubble mound structure was replaced with a sand berm. Measures R2 and R3 are implemented to the southeast of the Katrina Cut at Pelican Island and the Sand Island platform and do not produce an impact at the study area. Restoration measures R4, R5, R6, and R7 are implemented in the vicinity of the Katrina Cut area. Measure R4 includes nourishment of the beach east of the Katrina Cut and the shorefront area east of Pelican Island. Figure 2-5 shows the shoreline comparison between R0 (blue line) and R4 (orange line). Measure R5 refills borrow pits at the back of the Katrina Cut area and restores the back-bay marsh and island platforms there. The comparison of the no-action shoreline and this measure is shown in Figure 2-6. Measure R6 represents a modification of R4 that moves the restored dune area on the western side of Katrina Cut landward to better align with locations observed in 1950s photographs (Mickey et al. 2020). It does not include the beach nourishment in the east. Figure 2-7 shows a comparison of the shorelines for Measure R7 and R0. Measure R7 extends the beach restoration in R6 to the front of the Katrina Cut structure, and the shoreline is shown in Figure 2-8.

Figure 2-5. R4: West and East End beach and dune nourishment.

Figure 2-6. R5: Back Barrier tidal flats and marsh habitat restoration.

Figure 2-7. R6: West End beach and dune nourishment.

Figure 2-8. R7: West End and Katrina Cut beach and dune nourishment.

The modeling of the barrier island restoration measures was done for three levels of tropical cyclone (TC) frequency and intensity, which will be referred to herein as storminess levels. A Monte Carlo sampling method was used to sample 1,000 life cycles of TCs. This method is described in Chapter 3. The USGS (Mickey et al. 2020) developed a method to group the life-cycle storm sequences or realizations into storminess bins (ST) based on the corresponding island response using a one-dimensional (1D) proxy model framework (CA1D). Additional description of the ST approach can be found in section 3.3. A single sequence of storms was selected per ST to be modeled in the full modeling framework. The framework consists of Delft3D (Deltares 2019) and the empirical dune growth model (EDGR), along with numerical model XBeach (Roelvink et al. 2009), to simulate storm-induced erosion. Three 10-year ST were run, each adopting three RSLC scenarios. These were determined using a relative rate of sea level change (SLC) of 3.61 mm/year along with the USACE SLC curve calculator (version 2017.55) for low, intermediate, and high curves at Dauphin Island National Oceanic and Atmospheric Administration gage 8735180 (Mickey et al. 2020). These relative sea level change cases (SL), projected to 2070, are as follows:

- SL1: 0.3 m increase to water level at offshore boundary
- SL2: 0.5 m increase to water level at offshore boundary
- SL3: 1.0 m increase to water level at offshore boundary.

The full model framework was initialized with the 2015 Dauphin Island configuration. It was run for two scenarios based on the combination ST and SL discussed above. Scenarios of storminess bin 2 with SL1 (ST2SL1) and storminess bin 3 with SL3 (ST3SL3) were run for the restoration measures R2 through R7. R1 was run only with ST3SL3 and resulted in a breach in the Katrina Cut area 3.5 km wide (Mickey et al. 2020). Figures 2-9 to 2-18 show the comparison of the initial and resulting shorelines of the USGS (Mickey et al. 2020) analysis. The figures provide a comparison of changes in Dauphin Island shoreline from initiation at T=0 year to the end of the modeled period at T=10 years for the restoration measures and two scenarios run. Each figure contains an inset with a zoom of the Katrina Cut area. These results were considered in the damage progression modeling of the rubble mound structure.

Figure 2-9. R0: No action for T0 to T10 for ST2SL1.

Figure 2-10. R0: No action for T0 to T10 for ST3SL3.

Figure 2-11. R4: West and East End beach and dune nourishment T0 to T10 for ST2SL1.

Figure 2-12. R4: West and East End beach and dune nourishment T0 to T10 for ST3SL3.

Figure 2-13. R5: Back Barrier tidal flats and marsh habitat restoration for T0 to T10 for ST2SL1.

Figure 2-14. R5: Back Barrier tidal flats and marsh habitat restoration for T0 to T10 for ST3SL3.

Figure 2-15. R6: West End beach and dune nourishment for T0 to T10 for ST2SL1.

Figure 2-16. R6: West End beach and dune nourishment for T0 to T10 for ST3SL3.

Figure 2-17. R7: West End and Katrina Cut beach and dune nourishment T0 to T10 for ST2SL1.

Figure 2-18. R7: West End and Katrina Cut beach and dune nourishment T0 to T10 for ST3SL3.

3 Probabilistic Storm Forcing

Tropical cyclones represent the main driver of coastal hazards in the Gulf of Mexico coastline of the United States. As previously described, the Katrina Cut in Dauphin Island, AL, was caused by Hurricanes Ivan and Katrina in back-to-back years. The forcing for the hydrodynamic and morphological modeling, as well as the structural response modeling for Dauphin Island consisted of sampled TCs using a Monte Carlo Life-Cycle approach.

3.1 Forcing data

The statistical assessment of coastal storm hazards in hurricane-prone coastal areas requires the development of a joint probability model of TC forcing parameters. The joint probability method with optimal sampling (JPM-OS) became the standard joint probability analysis model for the assessment of coastal storm hazards for hurricane-prone US coastlines following Hurricane Katrina (Resio et al. 2007; Nadal-Caraballo et al. 2015). In this method, synthetic tropical storms that cover forcing probability space are developed from a joint probability model of tropical storm parameters (i.e., reference location $[x_o]$, heading $[\theta]$, central pressure deficit $[\Delta p]$, radius of maximum winds $[R_{max}]$, and translational speed $[V_t]$ and modeled in high-resolution numerical physics-based models to obtain the corresponding response. The annual exceedance probability of coastal storm hazards at a given site is a function of three main components: the storm recurrence rate, the joint probability of characteristic TC forcing parameters, and the TC responses (e.g., storm surge, wave height).

The forcing used for this study was based on comprehensive coastal storm modeling that was completed for coastal Alabama under Federal Emergency Management Agency (FEMA) Region IV Risk Mapping, Analysis and Planning study and Digital Flood Insurance Rate Map update for the Florida Panhandle and Alabama coasts. The modeling included waves and water levels for 295 synthetic tropical storms making landfall in the region that were derived using JPM-OS (URS/Dewberry 2012).

Water levels and waves were computed using two different models: (1) the Simulating WAVes Nearshore (SWAN) model (<http://www.swan.tudelft.nl>), used for producing offshore, regional, and coastal wave conditions, and (2) the

Advanced Circulation (ADCIRC) model (ADCIRC 2017; Luetlich et al. 1992; Westerink et al. 1992), which was used to simulate two-dimensional depth-averaged surge and circulation responses to the storm conditions.

The modeling used two different ADCIRC meshes, one that had coarser resolution covering much of the western North Atlantic Ocean, the Caribbean Sea, and the Gulf of Mexico and a finer resolution covering the floodplain areas within Mobile and Baldwin Counties, Alabama; and Escambia, Santa Rosa, Okaloosa, Walton, Bay and Gulf Counties, Florida (Figure 3-1). The SWAN model was run with three levels of grids, a basin, a nested regional, and then 5 coastal grids (Figure 3-2).

Figure 3-1. Map Showing portions of ADCIRC mesh covering Mobile and Pensacola Bay.¹

¹ Massey C., T. E. Godsey, R. Jensen, M. Bryant, Y. Ding, and M. Owens. 2020. Unpublished draft. *An Overview of the Coastal Storm Model Simulations of Waves and Water Levels for the Alabama Coastal Comprehensive Plan.*

Figure 3-2. Map showing SWAN wave model domains.¹

Save point locations were set up in the region for obtaining the ADCIRC results, and SWAN outputs were obtained at the grid nodes. Figure 3-3 shows the save point locations for the ADCIRC modeling and the extents of the coastal region 2 SWAN grid that covers the project area. The red points denote landfall locations of the synthetic storms. Save point 291 was selected to extract water levels for the structure response modeling. Wave data was extracted from the closest SWAN grid node to save point 291.

¹ Massey C., T. E. Godsey, R. Jensen, M. Bryant, Y. Ding, and M. Owens. 2020. Unpublished draft. *An Overview of the Coastal Storm Model Simulations of Waves and Water Levels for the Alabama Coastal Comprehensive Plan.*

Figure 3-3. FEMA Region IV save points and SWAN model region 2.

3.2 Monte Carlo storm sampling

The forcing for the modeling of Dauphin Island morphological evolution and structure response consisted of a suite of TCs sampled for a 10-year simulation period and multiple life cycles. The number of synthetic TCs in a given year was computed using the Poisson distribution and the storm recurrence rate (SRR). The SRR corresponding to the location of save point 291 was computed as 9.63×10^{-4} storms/year/km using the Gaussian kernel methodology developed by Chouinard and Liu (1997) and Chouinard et al. (1997). Consistent with experiments performed by Nadal-Caraballo et al. (2019), an optimal kernel size of 200 km was applied to compute the SRR. The SRR was converted to storms per year by multiplying by 400 km to represent the recurrence rate associated with storms passing within 200 km of the coastal reference location (CRL) closest to save point 291. This results in a rate of 0.385 storms/year.

The occurrence of TCs within the North Atlantic basin is often described as a Poisson process (e.g., Chouinard and Liu 1997; Elsner et al. 2001; Sommers 2003). The Poisson distribution can be used to compute the number of times an event occurs in an interval of time knowing the recurrence rate λ of X , where X is a random variable representing the number of TCs in a year. When applied to TCs, the probability of observing x TCs in an interval of time is given by

$$P(X = x) = \frac{\lambda^x e^{-\lambda}}{x!} \quad (3-1)$$

The Poisson distribution was used to determine the number of storms in a given year within the Monte Carlo sampling (e.g., two storms in the first year of the simulation, zero storms the second, one storm the third). Storm sequences that sampled a total number of storms that matched the SRR were retained for the analysis. The variability considered in the sampled life cycles included the selection of different combinations of storms and parameter uncertainty. Synthetic TCs with landfall locations within 200 km of the CRL were sampled randomly using their probability masses as sampling weights.

The workflow for the Katrina structure response analysis is shown in Figure 3-4. The TC storms are sampled at save point 291, and corresponding wave conditions transformed to the toe of the structure where structure response is computed. The reliability of the structure is then computed using the outcome of the simulations with respect to a limit state. Reliability criteria is described further in section 4.1.

Figure 3-4. General workflow for computing structure response and reliability for Katrina Cut structure.

Since the Katrina Cut structure was built in relatively shallow water and the deposition of sand seaward of the structure has further decreased the water depth there, wave conditions are considered depth limited. A limitation to the zero-moment wave heights at the structure was applied based on Battjes and Janssen (1978), applying Equation 3-2 to obtain depth limited waves based on both depth and wave steepness:

$$H_{m0_{max}} = 0.1L \tanh(kd) \quad (3-2)$$

where $H_{m0_{max}}$ is the maximum depth limited significant wave height; L is the wavelength; k is the wave number; and d the water depth. This wave-breaking criteria is also applied in spectral-wave model STWAVE (Smith et al. 1999). Both k and L are based on the peak wave period and computed using the linear wave theory dispersion relation.

3.3 Storm sampling implementation for morphological model

Storm sampling for the CA1D modeling framework for the morphologic analysis was performed by the USGS (Mickey et al. 2020) using the methodology described in section 3.2. It resulted in one thousand 20-year realizations of storm sequences. The first 10 years out of the 20-year periods were selected for the 1D analysis based on a change in the timeframe over which the measures needed to be evaluated. This 10-year simulation period is relevant for coastal planning and management and aligns with the overall objective of the Alabama Barrier Island Restoration Assessment. Each of the one thousand 10-year realizations was classified into three storminess bins (low, medium, and high) based on the island response obtained with the 1D model. Specifically, classification was based on the number of events that were estimated to result in overwash of the representative dune profile (Mickey et al. 2020).

The number of life cycles per storminess bin is shown in Table 3-1. The Katrina Cut structure response modeling was performed for the same three storminess bins to harmonize the structure analysis with the Dauphin Island evolution modeling approach. However, as mentioned in section 2.2, only a single representative storm sequence per storminess bin was selected to perform the morphologic modeling associated with the full modeling framework for all RSLC scenarios. The structure response analysis was performed for all the life cycles since there was no computational restriction for running the simulations. However, the morphologic response used in the structure modeling corresponds to a

single instance per storminess bin and RSLC case. It is recognized that representing the morphological response by a single sequence of storms or iteration precludes the quantification of the aleatory uncertainty associated with that response.

Table 3-1. Storminess bin classification and associated probability.

Storminess Bin	Number of Life Cycles	Probability
1	157	13.3%
2	576	57.3%
3	267	29.4%

Initially, 20-year life cycles were computed. As discussed in section 3.2, the 1,000 life cycles were selected from a larger group based on how well each life cycle matched the target SRR. A tolerance of 0.05 storms/year was used as selection criteria. Sampled sequences with an SRR within this tolerance were selected. However, this SRR matching was not strictly adhered to in the final life cycles used because only the first 10 years were selected for forcing the decadal model. Figure 3-5 shows the histogram of the number of storms per 10-year period for each storminess bin, compared to the expected distribution of storms per 10-year period based on the Poisson distribution and an SRR equal to 0.385 storms/year ($\lambda=3.85$ storms/decade). The histogram is shown for the combined one thousand 10-year realizations. The frequency of storms in the sampled 10-year one thousand life cycles is compared to the frequency of storms predicted by the Poisson distribution in Table 3-2. The resulting number of storms per life cycle overpredicts the Poisson distribution for 3, 4, and 5 storms per decade and underpredicts for the rest. The probability associated with the occurrence of each storminess bin was computed for the 10-year realizations using the Poisson distribution probabilities and storm relative probabilities. This process is described further in Mickey et al. (2020). The resulting probability for each storminess bin was 13.3%, 57.3%, and 29.4% for ST1, ST2, and ST3, respectively. These values were also listed in Table 3-1.

Figure 3-5. Histogram showing the distribution of number of TCs across the realizations for each storminess bin.

Table 3-2. Comparison of storm count frequency and Poisson distribution probability for one thousand 10-year life-cycle realizations.

Number of Storms per Life-Cycle Bin	Number of Life-Cycles Count	Frequency (Number of Life Cycles per Bin/1000)	Poisson Probability ($\lambda=3.85$)
0	5	0.005	0.021
1	40	0.04	0.082
2	144	0.144	0.157
3	257	0.257	0.202
4	279	0.279	0.195
5	178	0.178	0.150
6	75	0.075	0.096
7	19	0.019	0.053
8	5	0.005	0.026

4 Life-Cycle Structure Response Modeling

The armor stability of the Katrina Cut rubble mound structure was analyzed based on storm peak armor damage and seaside and leeside armor damage progression. Four assessments were performed to characterize the Katrina Cut rubble mound structure response within the context of the Alabama Barrier Island Restoration Assessment Project.

- Reliability analysis of stone armor stability based on storm peaks to evaluate the effect of a beach versus no beach in front of the structure.
- Stone armor damage progression sensitivity analysis for various forcing, RSLC, and crest height conditions to define the damage performance surface.
- Stone armor damage progression analysis using the forcing conditions and RSLC scenarios that coincide with the full barrier island evolution modeling of the restoration measures as described in section 2.2.
- A damage progression analysis that samples TCs outside the context of storminess bins by applying 10,000 life cycles from the full set of TCs to corroborate the results of the binning approach.

4.1 Armor stability and reliability

The performance of the rubble mound structure was assessed using a reliability-based probabilistic approach. The equations used for the computation of seaside and leeside stone armor stability and damage progression used for these analyses are described in Appendix A. The general objective of the approach is to determine, for a particular performance mode, the probability of exceeding a specific level of response for the structure. The reliability approach applied herein was based on sampling the storm forcing and computing the probabilistic response explicitly using Monte Carlo simulations. Because there are a large number of forcing and response variables that have significant statistical variability and correlations, this method is usually significantly more accurate for this type of engineering analysis than other simplified approaches.

Reliability is defined in terms of a limit state that characterizes the failure surface of the system. The limit state can be formulated in terms of a performance function as follows:

$$g = R - S \begin{cases} < 0 \text{ failure} \\ = 0 \text{ limit state (failure)} \\ > 0 \text{ no failure} \end{cases} \quad (4-1)$$

where g is the performance of the system, R is the strength of the system, S is the load on the system, and failure refers to the inability to meet the limit state criterion. The probability of an unsatisfactory response (e.g., failure) is given by

$$P_f = P(g \leq 0) \quad (4-2)$$

and the reliability is given by

$$R_f = 1 - P_f \quad (4-3)$$

Armor seaside and leeside stability and damage equations were rearranged into resistance (strength) and forcing (load) terms to conform to Equation 4-1. The limit state equations for seaside and leeside armor stability for storm peak forcing are described in Appendix A. The resistance terms of the equation includes the acceptable eroded area of the structure, stone size, and density. The forcing terms include wave parameters and steepness, number of waves, and water depth.

The reliability or probability of satisfactory performance was computed as the percent of realizations that the structure does not exceed the limit state. This limit state was defined with respect to specific levels of damage, S , at the end of a life cycle. In the following sections, damage is described by $S = A_e / (D_{n50})^2$ = normalized eroded area, A_e = eroded area and D_{n50} the nominal stone size. The limit state is meant to describe acceptable versus non-acceptable performance. Herein, unacceptable performance is defined as breaching given the temporary nature of the structure that is being examined. Damage is computed for every iteration and compared against the limit state. The reliability is defined by the ratio of iterations for which the limit state is not exceeded and the total number of iterations.

Melby and Kobayashi (2011) suggest that damage $S = 4.5N_{od}$, where N_{od} is the number of stones displaced, and this provides a practical description of damage. Damage levels of $S = 0 - 5$ are considered no damage, 5 to 20 are serviceable, and over 20 are near the ultimate limit state where holes in the armor layer would be exposed to the core and the structure would be

near the initiation of breaching. This is based on Melby (1999) who suggested that this damage level corresponded to exposure of the core material. However, this is somewhat heuristic and uncertain, particularly for this low-crested structure that is semi-protected by a beach. Physical modeling would yield a more accurate estimate of the failure limit state for damage for this structure, but that was not feasible within the scope of this project. For seaside stability, a maximum damage value of $S=30$ was considered to produce a breach in the structure. Leese side damage for a low-crested structure such as this one can progress more rapidly than seaside. Plunging overtopping wave action on the mostly unrestrained leese side armor stones just below the leese side corner of the crest can create a notch that concentrates damage and produces rapid erosion that can progress towards the seaside of the structure forming a breach (Melby et al. 2015). For leese side damage, a more conservative damage level of $S=15$ was considered as breaching.

4.2 Storm peak reliability analysis

An initial reliability analysis for structure stability was performed based on storm peaks for all combinations of storminess bins and RSLC conditions. Two beach configurations were considered that bracketed the variation of morphologic changes. These were a “with-beach” and “no-beach” condition. The no-beach condition was modeled using a toe elevation of -2.44 m. This represents the deepest point in the rubble mound structure alignment according to the design documents included in the permit. To determine a toe elevation representative of the beach at the initiation of the modeling (2015 conditions), a higher-resolution digital elevation model (DEM) was created using some of the same topobathy data sources used for the barrier island evolution modeling DEM. These data sources included the USGS post Isacc 2012 Lidar, USGS 2015 Barrier Island Lidar, USACE/USGS August 2015, and the USACE September 2016 topo/bathy datasets. A three-dimensional rendering of the higher-resolution DEM is shown in Figure 4-1, along with the location of transects used to evaluate different beach widths in front of the structure. Transect XS 3 represented the 2015 location with the smallest beach width seaward of the structure. A toe elevation of 0.49 m was extracted to represent the with-beach condition.

Figure 4-1. DEM for determining the beach profile (10X vertical exaggeration).

For the reliability analysis of seaside and leeside stability, exceedance or non-exceedance of the limit state was computed for each storm in a life cycle, reliability computed per life cycle, and results grouped by storminess bin. The number of life cycles per bin was as listed in Table 3-2. A mean reliability value was computed for each storminess bin for each RSLC scenario as shown in Table 4-1. The outcome of the reliability analysis quantified heuristic expectations of the structure response for varied beach widths and RSLC scenarios. Reliability is high for a wide beach and low for a narrow beach, which describes the fact that the structure is not vulnerable to breaches with a wide beach but will breach if no fronting beach is present. The effect of the beach is to increase the bottom elevation at the toe of the structure and limit the maximum wave height that impacts the structure. Reliability values generally decrease with an increase in storminess conditions. Also, as anticipated, reliability values decreased with an increase in sea levels.

The presence of the beach reduces the wave energy that reaches the structure and results in higher reliability values. For the no-beach condition, the reliability of the structure varies from 0.75 for ST1SL1 to 0.33 for ST3SL3. For the with-beach condition, the reliability varies from 1 to 0.79. The largest decrease in reliability due to storminess bin occurs between ST1 to ST2. There is a small decrease in reliability from storminess bin ST2 to ST3 when compared with ST1. The results also show the degree to which the reliability of the structure is sensitive to rising

relative sea levels. The percent differences across ST and RSLC scenarios were computed and are shown in Tables 4-2 and 4-3, respectively. As seen in Table 4-2, for the no-beach condition, the reduction in reliability from ST1 to ST2 for all RSLC conditions is in the order of 30%. The difference is between 4.1% and 11% for the with-beach condition. The reduction in reliability due to an increase in the RSLC for the no-beach condition is from 6.1% to 10.8% from SL1 to SL2 and 28.6% to 31.3% from SL1 to SL3 as seen in Table 4-3. For the with-beach condition, the maximum reduction in reliability from SL1 to SL2 is approximately 1% to 9.7% from SL1 to SL3.

This initial analysis did not apply empirical parameter or forcing uncertainty in order to isolate the effects in the variation in storminess bin and RSLC. Nevertheless, the analysis does consider the sampling variability associated with the storm selection for the different life cycles. Since, as seen in Figure 3-3, a relatively low number of storms is sampled in each life cycle (storms are relatively infrequent) and the number of life cycles varies from 167 for ST1 to 576 for ST2, there is significant variability in the computed reliability for each life cycle. As mentioned earlier, the above reliability values were mean values for each storminess bin. Table 4-4 shows the standard deviation of the computed reliability values per each storminess bin and RSLC scenario. The spread of the results is larger for the no-beach condition than for the with-beach condition.

Table 4-1. Mean reliability values per storminess bin and RSLC conditions for rubble mound structure stability.

Reliability - No Beach			
Case	SL1	SL2	SL3
ST1	0.75	0.67	0.53
ST2	0.49	0.46	0.35
ST3	0.48	0.44	0.33
Reliability - With Beach			
ST1	1.00	1.00	0.97
ST2	0.94	0.90	0.84
ST3	0.93	0.90	0.79

Table 4-2. Percent difference in reliability with respect to storminess bin 1.

No Beach			
	SL1	SL2	SL3
ST1	-	-	-
ST2	-34.7	-31.3	-34.0
ST3	-36.0	-34.3	-37.7
Beach			
ST1	-	-	-
ST2	-4.1	-5.0	-11.0
ST3	-5.2	-6.0	-14.4

Table 4-3. Percent difference in reliability with respect to RSLC scenario 1.

No Beach			
	SL1	SL2	SL3
ST1	-	-10.8	-29.3
ST2	-	-6.1	-28.6
ST3	-	-8.3	-31.3
Beach			
ST1	-	0	0
ST2	-	-1.0	-7.2
ST3	-	-0.8	-9.7

Table 4-4. Standard deviation of the computed reliability values.

Reliability - No Beach			
Case	SL1	SL2	SL3
ST1	0.3	0.33	0.36
ST2	0.3	0.28	0.26
ST3	0.24	0.23	0.2
Reliability - With Beach			
ST1	0	0	0.02
ST2	0.12	0.14	0.19
ST3	0.10	0.10	0.15

Collectively, the reliability values suggest that the structure would be judged as failed (breached) under the no-beach condition for all storminess bins and all RSLC scenarios. However, under the with-beach condition, the structure would be intact under all scenarios with no damage ST1 and would have serviceable damage under ST2 and ST3 scenarios.

4.3 Damage progression

Structure response over time was assessed for seaside and leeside stability following Melby and Kobayashi (2011). A description of these methodologies is presented in Appendix A. Three analyses for structure damage progression were performed to evaluate the effects of beach condition in front of the structure and storminess. First, a sensitivity analysis was performed varying elevations at the toe of the structure to represent the effect of varying beach conditions. Second, a damage progression analysis was performed for toe elevations obtained from the restoration measure model outputs described in section 2.2. Third, damage progression was performed for ten thousand 10-year life cycles without dividing the storms into storminess bins to corroborate storminess bin damage results.

Some of the sampled storms produced water elevations that were above the crest elevation of the structure. In this condition, the seaside slope is no longer directly exposed to wave action. For these conditions, a cut-off elevation was implemented to the seaside damage equations to eliminate damage accumulations for when the water level hydrograph was 0.30 m over the crest height.

4.3.1 Damage progression sensitivity analysis

A sensitivity analysis of structure damage progression was performed to get a broad understanding of the effect of different beach conditions in front of the structure, storminess, and RSLC. Toe depths were gradually varied from the with-beach condition elevation to the no-beach design depth that were used for the peaks analysis in section 4.2. Seaside and leeside damage results for toe depths of -0.49, 0.5, 1.5, 2, and 2.44 m are shown in Tables 4-5, 4-6, and 4-7 for each storminess bin and RSLC condition. Figures 4-2 to 4-7 visually represent the damage variation surface of the data presented in the tables. The figures also include the no-damage upper limit surface ($S = 5$) in green and the initiation of failure damage surface ($S = 20$) in red for assessing the damage results for seaside stability and $S=10$ for leeside.

The damage progression results show that the presence of a beach in front of the structure reduces accumulated damage on both seaside and leeside for all storminess bins and SL cases. As expected, damage also increases with increases in RSLC. Note that although figures for seaside show a vertical scale of S of up to 100, once S is above levels corresponding to approximately $S = 20$, the value of S loses significance since it is above the range of physical modeling experimental testing on which it is typically defined.

Table 4-5. Seaside and leeside damage for storminess bin 1.

Bin	Depth (m)	RSLC (m)	S_{SEA}	S_{LEE}
ST1	-0.49	0	0.0	0.0
ST1	-0.49	0.3	0.0	0.2
ST1	-0.49	0.5	0.0	0.4
ST1	-0.49	1.0	0.2	2.2
ST1	0.5	0	0.2	0.8
ST1	0.5	0.3	0.5	1.8
ST1	0.5	0.5	1.1	3.0
ST1	0.5	1.0	4.2	7.0
ST1	1.5	0	4.6	3.3
ST1	1.5	0.3	8.3	5.3
ST1	1.5	0.5	11.7	6.9
ST1	1.5	1.0	22.2	11.1
ST1	2	0	11.7	4.9
ST1	2	0.3	18.1	7.0
ST1	2	0.5	23.3	8.4
ST1	2	1.0	38.3	12.2
ST1	2.44	0	21.7	6.1
ST1	2.44	0.3	30.6	8.0
ST1	2.44	0.5	37.6	9.4
ST1	2.44	1.0	57.4	13.0

Figure 4-2. Seaside damage variation surface for ST1.

Figure 4-3. Leaside damage variation surface for ST1.

**Table 4-6. Seaside and leeside damage
for storminess bin 2.**

Bin	Depth (m)	RSLC (m)	SSEA	SLEE
ST2	-0.49	0	0.3	1.1
ST2	-0.49	0.3	0.5	1.8
ST2	-0.49	0.5	0.6	2.4
ST2	-0.49	1.0	1.2	4.4
ST2	0.5	0	2.1	2.6
ST2	0.5	0.3	3.0	4.0
ST2	0.5	0.5	3.9	5.3
ST2	0.5	1.0	7.8	9.4
ST2	1.5	0	11.3	5.6
ST2	1.5	0.3	16.3	7.9
ST2	1.5	0.5	20.8	9.7
ST2	1.5	1.0	34.8	14.1
ST2	2	0	23.2	7.4
ST2	2	0.3	32.1	9.8
ST2	2	0.5	39.5	11.6
ST2	2	1.0	61.1	15.7
ST2	2.44	0	40.1	8.9
ST2	2.44	0.3	53.3	11.3
ST2	2.44	0.5	64.0	13.0
ST2	2.44	1.0	93.9	16.8

Figure 4-4. Seaside damage variation surface for ST2.

Figure 4-5. Leaside damage variation surface for ST2.

Table 4-7. Seaside and leeside damage for storminess bin 3.

Bin	Depth (m)	RSLC (m)	SSEA	SLEE
ST3	-0.49	0	0.9	2.6
ST3	-0.49	0.3	0.9	3.5
ST3	-0.49	0.5	1.1	4.3
ST3	-0.49	1.0	1.6	6.8
ST3	0.5	0	4.4	4.5
ST3	0.5	0.3	5.3	6.1
ST3	0.5	0.5	6.2	7.6
ST3	0.5	1.0	10.6	12.6
ST3	1.5	0	19.3	8.0
ST3	1.5	0.3	24.7	10.7
ST3	1.5	0.5	29.8	12.8
ST3	1.5	1.0	46.3	18.2
ST3	2	0	36.9	10.2
ST3	2	0.3	46.9	13.2
ST3	2	0.5	55.6	15.3
ST3	2	1.0	81.7	20.3
ST3	2.44	0	61.5	12.1
ST3	2.44	0.3	76.9	15.0
ST3	2.44	0.5	89.6	17.1
ST3	2.44	1.0	126.1	21.8

Figure 4-6. Seaside damage variation surface for ST3.

Figure 4-7. Leaside damage variation surface for ST3.

4.3.2 Incorporation of uncertainty

Uncertainty was considered in the structure damage progression analysis for the restoration measures. Two main types of uncertainties are generally recognized: aleatory and epistemic. Aleatory uncertainty relates to the natural randomness of a process while epistemic is associated with a lack of knowledge associated with models and data used to represent a system. The analysis incorporated uncertainty related to storm forcing, structure physical parameters, and empirical structure response model parameters. The statistical parameters for the sources of uncertainty

considered are shown in Table 4-8. Refer to Appendix A for parameter descriptions. The uncertainty associated with storm sampling was quantified through the implementation of the life-cycle approach. The total uncertainty was ultimately represented by confidence levels of 10%, 16%, 84%, and 90%.

Table 4-8. Table of statistical quantities where N (mean, standard deviation) denotes normally distributed parameter and D (value) represents a deterministic value.

Parameter	Distribution	Description
H_{m0}	from data	from data
T_m	from data	from data
N_z	from data	from data
h	from data	from data
$\cot\alpha$	N(p, 0.1)	p = 2
P	N(p, 0.05)	p = 0.4
Δ	N(p, 0.06)	p = 1.65
K_s	D(0.4)	for initial damage
K_{m1}	N(5, 0.3)	-
K_{m2}	N(5, 0.4)	-
K_s	D(1)	-
K_{ls1}	N(125, 10)	-
K_{ls2}	N(0.4, 0.1)	-
M_{50}	N(450, 45)	-

4.3.3 Damage progression for restoration measures

The damage progression results presented in section 4.3.1 covered a range of structure toe depths between the no-beach condition and the with-beach condition. It was shown that the variation of this parameter had a significant effect on the computed damage. To harmonize the Katrina Cut structure analysis with the Dauphin Island evolution modeling for the proposed restoration measures, the DEMs produced at the end of the 10-year modeling periods were used to extract toe elevations for the structure response modeling. Restoration measures R4 through R7 impact the Katrina Cut area and were the focus of the analysis. The interpretation of the morphology analysis results was that the structure was considered fully exposed for storminess bin 3 for all

restoration measures except 7. Profiles were extracted from transects along the Katrina Cut, and the location of the structure toe was identified from the 2016 topographic survey. Model grid resolution turned out to be inadequate to resolve elevations at the structure toe; therefore, elevations from the breach near the west end of the structure for ST3SL3 were examined for use as toe elevations. The transect from which these elevations were extracted is shown as a red line in Figure 4-8. It is placed on the west side of the resulting barrier island breach. The ST3SL3 profiles at the end of the 10-year simulation period for restoration measures R0, R4, R5, R6, and R7 are shown in Figure 4-9. The location at which the toe elevations were extracted is shown as a black dashed line. The figure also lists the toe elevations that represent each restoration measure.

Figure 4-8. Profile line for toe depth selection at known breaching point.

Figure 4-9. Profiles extracted from ST3SL 3 DEMs for R0, R4, R5, R6, and R7 restoration measures at the end of 10-year simulation.

The damage progression analysis was performed for the barrier island modeling output of restoration measures R4 through R7, represented by different toe elevations based on the ST3SL3 results. ST2SL1 was analyzed for the same toe elevations even though breaching did not occur for the restoration measure. This exercise was done to vary forcing conditions for the analysis and for comparison with the reliability computed from storm peaks. In practical terms, the morphology outputs for ST2SL1 show that the beach remains in front of the structure over the analyzed 10-year period for all restoration measures, and therefore the expected structure damage is small. The ST2SL1 resulting shorelines can be seen in section 2.2, starting with Figure 2-9 for R0.

The probability of failure for the damage progression analysis was evaluated by computing the percent of realizations per each storminess bin that the rubble mound structure breaches. As mentioned at the beginning of the section, damage values over $S=20$ are near the ultimate limit state and the structure would be near the initiation of breaching.

The resulting mean damage values at the end of the 10-year simulations are shown in Table 4-9. The confidence levels associated with the computed damage values are presented in Tables 4-10 and 4-11 for seaside

and leeside damage, respectively. The results were consistent with the previous analysis with respect to the effect of the beach. Reliability estimates were determined for the damage progression assessment based on the percent of realizations that cause breaching at the end of the 10-year simulation period.

Table 4-9. Damage progression for STSL3 restoration measures representative toe depths.

Case	Restoration Measure	Toe Depth (m)	RSLC (m)	S_{sea}	Breach Percent (%)	S_{lee}	Breach Percent (%)
ST2SL1	R7	-0.39*	0.3	0.8	0.0	1.9	0.0
ST2SL1	R5	1.745*	0.3	25.4	11.0	9.6	1.0
ST2SL1	R6	1.825*	0.3	28.9	15.1	10.0	1.2
ST2SL1	R0	1.91*	0.3	32.5	18.5	10.2	1.8
ST2SL1	R4	2.283*	0.3	52.6	39.1	11.6	4.0
ST3SL3	R7	-0.39	1	1.9	0.0	6.7	0.0
ST3SL3	R5	1.745	1	68.2	49.7	19.9	23.4
ST3SL3	R6	1.825	1	74.6	56.9	20.3	23.7
ST3SL3	R0	1.91	1	79.6	59.9	20.9	26.3
ST3SL3	R4	2.283	1	123.9	78.0	22.8	28.6

* ST3SL3 toe depths.

Table 4-10. Accumulated mean seaside damage and confidence limits for restoration measures at the end of 10-year simulation period.

Case	Restoration Measure	S (10%)	S (16%)	S (50%)	S (84%)	S (90%)
ST2SL1	R7	0.0	0.0	1.0	1.4	1.6
ST2SL1	R5	14.2	16.0	25.4	35.2	37.6
ST2SL1	R6	14.6	16.5	28.2	39.7	46.1
ST2SL1	R0	18.3	21.4	30.9	46.1	52.2
ST2SL1	R4	28.4	30.7	47.9	73.7	82.3
ST3SL3	R7	1.1	1.2	1.8	2.7	3.2
ST3SL3	R5	42.1	48.2	65.1	90.9	100.5
ST3SL3	R6	49.5	53.1	70.8	95.4	101.9
ST3SL3	R0	51.6	57.2	74.1	105.0	108.3
ST3SL3	R4	81.1	90.4	114.9	158.4	170.9

Table 4-11. Accumulated mean leeside damage and confidence limits for restoration measures at the end of 10-year simulation period.

Case	Restoration Measure	S (10%)	S (16%)	S (50%)	S (84%)	S (90%)
ST2SL1	R7	0.4	0.6	2.0	3.1	3.3
ST2SL1	R5	6.4	7.3	9.2	12.1	13.2
ST2SL1	R6	6.4	7.2	9.7	12.8	13.7
ST2SL1	R0	7.3	7.7	9.9	12.8	13.2
ST2SL1	R4	8.1	8.8	11.4	14.2	15.4
ST3SL3	R7	4.8	5.2	6.3	8.6	9.1
ST3SL3	R5	14.6	15.7	19.2	23.6	25.5
ST3SL3	R6	15.4	16.4	19.7	24.1	25.5
ST3SL3	R0	15.8	16.5	20.5	25.6	26.7
ST3SL3	R4	17.0	18.1	21.7	26.8	29.9

A comparison between reliability computed using peaks as described in section 4.2 and reliability estimates based on damage progression is presented in Table 4-12. For this comparison, the damage progression modeling was executed using the same -.49 m and 2.44 m toe depths used in the peaks analysis, which were representative of the with-beach and no-beach conditions. The results show comparable trends in terms of structure performance with respect to the previous analyses. The reliability for the no-beach condition was similar for ST2SL1 while for ST3SL3 the reliability based on damage progression was lower. For the with-beach condition, the reliability value was 1 for the damage progression stability modeling for both cases. The results reinforce the idea that the with-beach condition does not breach while the no-beach condition breaches.

Table 4-12. Comparison of computed reliability using peaks and storm hydrograph.

Reliability - No Beach		
Case	Peaks	D. Progression
ST2SL1	0.49	0.50
ST3SL3	0.33	0.19
Reliability - With Beach		
ST2SL1	0.94	1
ST3SL3	0.79	1

4.3.4 Damage progression analysis without storminess bins

Up to this point, the analyses have considered the decadal-scale variation in storm frequency and intensity through the classification of sampled 10-year storm sequences into storminess bins. This classification was based on the barrier island morphologic response for each of the 1,000 realizations performed for the analysis. It did not consider the coastal rubble mound structure response modeling. Because this structure was not designed to resist extreme events without catastrophic damage, analysis using a 10-year series was reasonable to define performance in the context of morphological behavior. It was of interest to compare damage computed under the prior life-cycle storminess approach with a sampling approach that did not involve storminess bins. The analysis was performed for 10,000 life cycles, without classifying each sampled 10-year realization into a storminess bin. In this approach, each realization maintains the storm recurrence rate. A large number of life cycles were sampled to ensure stability of the computed mean damage and evaluate the appropriateness of the number of realizations used for the storminess bins. This analysis would be expected to quantify the probability of damage over the full range of hazard that this structure will be exposed to over a 10-year period.

The damage progression modeling for the 10,000 life cycles was performed for RSLC SLO, SL1, and SL3 using the response measure toe elevations from the ST3SL3 modeling. SLO was added to evaluate the effect of the full range of RSLC on the structure response. An evaluation of ST1SLO was included in the storminess bin analysis to have a basis of comparison with all three storminess bins. The seaside and leeside mean damage for both approaches is presented in Tables 4-13 and 4-14, respectively. The mean damage results for the 10,000 life-cycle approach closely match the mean damage results for storminess bin 2 for both the seaside and leeside. The 10,000 life-cycle mean damage is significantly higher than the damage of ST1, while the mean damage compared with ST3 is noticeably lower. These results reasonably confirm that the storminess bin 2 is representative of the decadal-scale variation in storm frequency and intensity corresponding to the uncensored TC climate.

Table 4-13. Seaside damage comparison between no-storminess bin and storminess bin approaches.

Case	Restoration Measure	Seaside S			Breaching (%)		
		No Bins	With Bins	Difference	No Bins	With Bins	Difference
ST1SL0	R7	0.6	0.0	0.6	0	0	0
ST1SL0	R5	19.7	9.8	9.9	4.8	0	4.8
ST1SL0	R6	22.2	10.4	11.7	6.5	0	6.5
ST1SL0	R0	24.7	11.3	13.3	8.8	0	8.8
ST1SL0	R4	39.7	21.2	18.6	23.5	4.7	18.8
ST2SL1	R7	0.9	0.8	0.1	0	0	0
ST2SL1	R5	27.8	25.4	2.4	11.5	11	0.5
ST2SL1	R6	30.7	28.9	1.8	14.9	15.1	0.2
ST2SL1	R0	34.3	32.5	1.7	18.7	18.5	0.2
ST2SL1	R4	53.6	52.6	1.0	38.9	39.1	0.2
ST3SL3	R7	1.4	1.9	0.5	0	0	0
ST3SL3	R5	52.2	68.2	16.0	43.5	49.7	6.2
ST3SL3	R6	56.9	74.6	17.7	48.5	56.9	8.4
ST3SL3	R0	62.3	79.6	17.4	53.7	59.9	6.2
ST3SL3	R4	91.2	123.9	32.7	70.7	78	7.3

Table 4-14 Leaside damage comparison between no-storminess bin and storminess bin approaches.

Case	Restoration Measure	Leaside S			Breaching (%)		
		No Bins	With Bins	Difference	No Bins	With Bins	Difference
ST1SL0	R7	1.5	0.2	1.4	0	0	0
ST1SL0	R5	7.2	4.7	2.5	0.1	0	0.1
ST1SL0	R6	7.6	4.9	2.7	0.1	0	0.1
ST1SL0	R0	7.9	5.5	2.4	0.2	0	0.2
ST1SL0	R4	9.2	6.5	2.7	0.7	0	0.7
ST2SL1	R7	2.0	1.9	0.2	0	0	0
ST2SL1	R5	9.6	9.6	0.1	0.9	1	0.1
ST2SL1	R6	10.0	10.0	0.0	1.2	1.2	0
ST2SL1	R0	10.3	10.2	0.1	1.6	1.8	0.2
ST2SL1	R4	11.7	11.6	0.1	3.9	4	0.1
ST3SL3	R7	5.0	6.7	1.7	0	0	0

Case	Restoration Measure	Leeside S			Breaching (%)		
		No Bins	With Bins	Difference	No Bins	With Bins	Difference
ST3SL3	R5	15.9	19.9	4.0	17.1	23.4	6.3
ST3SL3	R6	16.1	20.3	4.2	18.1	23.7	5.6
ST3SL3	R0	16.5	20.9	4.4	19.2	26.3	7.1
ST3SL3	R4	17.5	22.8	5.3	22.9	28.6	5.7

5 Wave Transmission

The Katrina Cut rubble mound structure assessment included evaluating the transmitted wave conditions on the leeside due to wave overtopping of the structure. Wave transmission coefficients (C_t) and transmitted wave heights (H_{mo_t}) were computed for ST2SL1 and ST3SL3 scenarios used in the damage progression analysis. The impact of restoration measure 5 on transmitted wave heights was also assessed.

5.1 Wave transmission analysis

The wave transmission analysis was applied to all life cycles for each of the storminess bins taking into the account the variation of crest height, toe depth, and RSLC. The combinations of these three parameters each correspond to a specific configuration, and each configuration was evaluated separately for ST1 and ST3. Figure 5-1 shows a schematic diagram of the wave transmission analysis configurations. Although crest reduction was not modeled explicitly as part of the damage progression analysis, survey data show that the structure has been reduced at various locations. Two values of crest elevations of 0.91 m and 1.83 m were modeled in addition to the design crest height of 2.44 m for the wave transmission analysis. Wave transmission was evaluated using the empirical relationship described in van der Meer et al. (2005). Refer to Appendix B for a description of this empirical relationship as well as its comparison to the wave transmission formula from the *Coastal Engineering Manual* (USACE 2002).

The transmitted wave heights (H_{mo_t}) at the leeside of the structure were computed using the empirical relationships for the wave transmission coefficient, C_t , (refer to Appendix B) and the incident wave heights at the toe of the structure obtained from the Monte Carlo simulation (H_{mo_i}). The C_t was applied to all the configurations mentioned previously, and the maximum of each life cycle was averaged to get a representative H_{mo_t} for the specific configuration. Table 5-1 shows the maximum mean transmitted wave height for all storminess bins, toe depths, and SLR configurations with a crest height of 2.44 m. The results for the full analysis are presented in Appendix C. As expected, wave transmission increases with storminess bin and sea level rise. This is due to the structure being submerged for longer periods of time, hence dissipating less wave energy. The same is true when evaluating lower crest heights

than the one shown in Table 5-1. The computed maximum mean transmitted wave heights are used in the following subsection and Appendix C to evaluate the impact restoration measure R5 would have on transmitted waves.

Figure 5-1. Schematic diagram of wave transmission analysis configurations.

Table 5-1. Mean maximum transmitted waves for all storminess bins, toe depths, and RSLC configurations with a crest height of 2.44 m.

Bin	Toe Depths (m)	$H_{m0,t}$ (m)			
		SL0	SL1	SL2	SL3
1	-0.39	0.02	0.04	0.04	0.15
1	1.75	0.10	0.11	0.12	0.21
1	1.83	0.11	0.13	0.16	0.49
1	1.91	0.11	0.12	0.13	0.28
1	2.28	0.14	0.28	0.43	0.79
2	-0.39	0.15	0.31	0.43	0.75
2	1.75	1.15	1.39	1.55	1.89
2	1.83	0.27	0.49	0.64	1.00
2	1.91	0.94	1.18	1.33	1.67
2	2.28	0.37	0.61	0.76	1.12
3	-0.39	1.75	1.88	1.97	2.17
3	1.75	0.87	1.12	1.29	1.65
3	1.83	1.48	1.71	1.85	2.14
3	1.91	0.75	0.99	1.16	1.53
3	2.28	1.91	2.11	2.22	2.42

5.2 Impact of restoration measure 5 on transmitted wave energy

Restoration measure 5 is implemented in the leeward side of the Katrina Cut structure. Figure 5-2 shows with a red polygon the footprint of the areas with an elevation difference between the proposed tidal flats and marsh habitat restoration and the existing conditions. This measure has the potential to impact the wave transmission to the leeward side of the structure as it significantly reduces the water depths. The profile along the length of the structure was extracted for existing conditions and for R5 morphologic modeling results. These profiles include the initial condition for R5 and the end of simulation period profiles for ST2SL1 and ST3SL3. The location of the profile line is shown in Figure 5-2 as a yellow line 30 m from the structure crest, and the profiles are shown in Figure 5-3.

Restoration measure 5 increases the bottom elevation of the area at the back of the Katrina Cut structure to an elevation of near 0.5 m. As seen in Figure 5-3, the existing profile in the lee of the structure is variable and has depths over 2 m at the deepest point. This location is identified in plan view in Figure 5-2. The initial R5 profile is shown as the black dashed line in Figure 5-3 while the morphology modeling result for ST2SL1 is represented by the blue line. The bottom elevation is reduced less than 0.30 m at the end of the 10-year simulation period throughout the profile. For ST3SL3, the elevation profile continuously decreases towards the west end of the Katrina Cut where breaching is predicted to occur.

An assessment of the impact of restoration measure 5 on wave transmission was performed by evaluating the transmitted wave heights with respect to the resulting water depths. The wave breaking depth limitation criteria discussed in section 3.2 was used to determine if the mean maximum transmitted waves were susceptible to wave breaking. The occurrence of wave breaking on the leeward side of the Katrina cut structure for transmitted waves is shown in Figures 5-4 to 5-6 for the initial R5 condition and three crest height elevations and five toe depths. Conditions where wave breaking occurs are identified with a red circle, and conditions where the transmitted waves are not depth limited are identified with a green square. These figures are meant to give a general idea of the effect of R5 on transmitted waves at the deepest location in the existing conditions profile. Breaker criteria analysis for the other configurations are shown in Appendix C. Since water depths vary along the profile at the end of the 10-year simulations, in particular for ST3SL3, profiles of maximum transmitted wave heights (H_{mo_max}) were generated that take into account

depth limitation. Figures C-28, C-29, and C-30 from Appendix C show the maximum transmitted wave heights along the length of the profile leeside of the Katrina Cut structure for three crest height elevations.

Figure 5-2. Line for profile extraction of water depths and footprint of restoration measure 5 on the leeside of the Katrina Cut structure.

Figure 5-3. Comparison between restoration measure 5 profiles and existing profile (NAVD88).

Figure 5-4. Mean maximum transmitted waves and wave breaking classification at leeside for all ST1 toe depths and SLR configurations with a crest height of 2.44 m.
Leeside depth: -0.47 m.

Figure 5-5. Mean maximum transmitted waves and wave breaking classification at leeside for all ST2 toe depths and SLR configurations with a crest height of 2.44 m.
Leeside depth: -0.47 m.

Figure 5-6. Mean maximum transmitted waves and wave breaking classification at leeside for all ST3 toe depths and SLR configurations with a crest height of 2.44 m.
Leeside depth: -0.47 m.

6 Summary and Conclusions

The rubble mound structure placed at the Katrina Cut in Dauphin Island, AL, was designed to be a temporary structure whose ultimate objective was to prevent oil migration into the Mississippi Sound after the Deep Water Horizon oil spill. As such, the armor stone was undersized with respect to generally accepted armor design criteria that limits catastrophic damage from extreme storm events. Within the first 2 years of its construction, the structure showed damage as the seaside stone armor was directly exposed to wave action. The survey performed in 2016 revealed areas with significant damage and lowering of the crest height at multiple locations. Since construction, morphological changes near the structure have been characterized by the steady accumulation of sand and the formation of a broad beach on the seaside (south) of the rubble mound structure. The assessment of the Katrina Cut rubble mound structure was performed as part of the Alabama Barrier Island Restoration Assessment study undertaken to evaluate potential restoration measures for Dauphin Island considering varying RSLC and storminess scenarios.

The rubble mound structure response was evaluated using a life-cycle modeling approach. The analysis considered the outcomes of the morphologic modeling of the barrier island performed by the USGS for proposed restoration measures that impact the Katrina Cut area. For the morphological response analysis, one thousand 10-year life cycles were sampled and classified based on storm frequency and intensity into three storminess bins. Morphological modeling of measures was performed for an intermediate scenario of storminess bin 2 (ST2) and low RSLC scenario (SL1) and an extreme scenario of ST3 and high RSLC scenario (SL3).

The beach in front of the structure plays an important role in protecting the structure. The beach dissipates wave energy through wave breaking due to depth limitation. An initial assessment of the effect of the beach on the reliability of the structure showed that the absence of the beach can reduce its reliability more than 50%. The structure is expected to be protected for scenario ST2SL1 for all restoration measures since the beach was retained. Scenario ST3SL3 results in a loss of beach for restoration measures R4, R5, and R6 and corresponding expected breaching 78, 50, and 57 percent of the time for seaside damage, respectively. No breaching is expected for restoration alternative R7 since a beach is retained seaward

of the structure. If the structure holds for ST3SL3, breaching occurs to the west of the structure at the end of the 10-year period.

Since the Katrina Cut structure is low crested, overtopping wave transmission is expected. Wave transmission analysis was applied for all life cycles for each of the storminess bins taking into the account the variation of crest height, toe depth, and sea level rise. Mean maximum transmitted wave heights were computed for the various configurations and, as expected, wave transmission increases with storminess bin and with sea level rise. Restoration alternative 5 is implemented in the leeside of the Katrina Cut structure and increases sea floor elevations there. The effect of R5 is to reduce transmitted wave heights compared to existing conditions. Wave transmission is increased at the end of the 10-year simulation period for ST3SL3 due to a decrease in the profile, but it still remains higher than the existing conditions profile.

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Appendix A: Stability and Damage Analysis

This appendix discusses the stability equations used for the Katrina Cut structure.

A.1 Seaside armor stability

The seaside armor stability was based on maximum momentum flux methods of Melby and Hughes (2004) with generalizations from Melby and Kobayashi (2011). The approach captures the effects of wave steepness and water depth on stability and damage progression over a wide range water depths primarily because these processes are included in the wave momentum flux basic principles included in derivation of the equations. The maximum wave momentum flux is highly nonlinear for nonlinear waves (i.e., steep waves in shallow water). This corresponds to the case where armor stability is at its minimum. The nonlinear wave momentum flux was approximated numerically using a Fourier solution. The resulting approximate relation was found to be

$$\left(\frac{M_F}{\rho_w g h^2} \right)_{\max} = A_0 \left(\frac{h}{g T_m^2} \right)^{-A_1}$$

$$A_0 = 0.639 \left(\frac{H_{m0}}{h} \right)^{2.026} \quad (A-1)$$

$$A_1 = 0.180 \left(\frac{H_{m0}}{h} \right)^{-0.391}$$

where H_{m0} is the significant wave height, h is the local water depth, T_m is the mean wave period, M_f is the momentum flux, ρ_w is the density of water, and g is the acceleration of gravity. A nonlinear approximation for momentum flux is important because stability is at its minimum when the incident wave is the most nonlinear.

Two stability equations resulted from the fit to data using Equation A-1. The recommended equations for stability are

$$N_m = \frac{1}{a_m} \left(\frac{S}{K_s \sqrt{N_z}} \right)^{0.2} \quad (\text{A-2})$$

and

$$N_m = \left(\frac{(M_F / \gamma_w h^2)_{\max}}{(S_r - 1)} \right)^{1/2} \frac{h}{D_{n50}} \quad (\text{A-3})$$

Plunging waves:

$$a_m = \frac{1}{K_{m1} P^{0.18} \sqrt{\cot \theta}} \quad s_m \geq s_{mc} \quad (\text{A-4})$$

Surging waves:

$$a_m = \frac{s_m^{P/3}}{K_{m2} P^{0.18} (\cot \theta)^{0.5-P} s_m^{-P/3}} \quad s_m < s_{mc} \quad (\text{A-5})$$

$$s_{mc} = H_{m0} / L_m, \quad s_{mc} = -0.0035 \cot \theta + 0.028 \quad (\text{A-6})$$

Solving for the stable stone size yields

$$D_{n50} = h a_m \left(\frac{S}{K_s \sqrt{N_z}} \right)^{-1/5} \left(\frac{(M_F / \gamma_w h^2)_{\max}}{\Delta} \right)^{1/2} \quad (\text{A-7})$$

Here, $D_{n50} = (V_{50})^{1/3}$ = nominal stone size, $V_{50} = M_{50}/\rho_r$ = median volume of armor stone, M_{50} = median mass, $\cot \theta$ = structure seaward slope, $\Delta = S_r - 1$, $S_r = \rho_r/\rho_w$ = relative specific gravity of armor stone, ρ_r = density of stone, ρ_w = density of water that the structure is in, N_m is the momentum flux stability number, P = notional permeability of the structure, $S = A_e/(D_{n50})^2$ = normalized eroded area, A_e = eroded area, and N_z = storm duration/ T_m . S and A_e are illustrated in Figure A-1. $K_s = 1.3$ is an empirical parameter to account for accelerated damage that occurs with constant wave conditions, γ_w is the specific weight of water, θ is the seaside structure slope, and s_m is the local wave steepness. K_{m1} and K_{m2} are empirical fit coefficients.

Figure A-1. Illustration of rubble mound damage parameters.

Damage progression was assessed for seaside and leeside stability following Melby and Kobayashi (2011) and the methodology presented in Melby et al. (2015). The equations for damage progression used for the assessment of Katrina Cut structure are presented in the latter.

A.2 Leeside armor stability

Leeside stability was determined using Van Gent and Pozueta (2004) stability equations, revised to be in similar form to the seaside equations (Melby 2009).

$$D_{n50} = a_{ls} \left(\frac{S_{ls}}{K_{ls} \sqrt{N_z}} \right)^{-1/r} \left(\frac{u_{1\%} T_{m-1,0}}{125 \sqrt{\Delta}} \right) \quad (\text{A-8})$$

$$a_{ls} = (\cot \phi)^{-2.5/r} [1 + 10 \cdot \exp(-R_{c-rear} / H_s)]^{1/r} \quad (\text{A-9})$$

where S_{ls} is the leeside damage, $r = 6$ for constant wave conditions, $T_{m-1,0} = m_{-1}/m_0$ of incident spectrum, $T_{m-1,0} \sim T_p / 1.1$ for a JONSWAP incident wave spectrum, R_{c-rear} = freeboard of leeside edge of crest, $\cot \phi$ = leeside slope, $u_{1\%}$ = maximum crest velocity exceeded by 1% of the waves, $H_s = H_{m0}$ of incident wave spectrum, K_{ls} and r are empirical fit parameters, and $(D_n)_{ls}$ and Δ_{ls} are the nominal stone size and density parameter for the

leeside armor, respectively. A leeside stability number, N_{ls} , is introduced where

$$N_{ls} = \left(\frac{u_{1\%} T_{m-1,0}}{125(D_n \sqrt{\Delta})_{ls}} \right) = \frac{1}{a_{ls}} \left(\frac{S_{ls}}{K_{ls} \sqrt{N_z}} \right)^{1/r} \quad (A-10)$$

Then, the single storm leeside damage for constant wave conditions can be expressed as

$$S_{ls} = K_{ls} \sqrt{N_z} (a_{ls} N_{ls})^r \quad (A-11)$$

The n^{th} moment of the incident wave energy density spectrum is given by

$$m_n = \int_0^\infty f^n S(f) df \quad (A-12)$$

The crest velocity exceeded by 1% of the waves is given by

$$\frac{u_{1\%}}{\sqrt{gH_s}} = \frac{1.7(\gamma_{f-C})^{0.5} \left(\frac{z_{1\%} - R_c}{\gamma_f H_s} \right)^{0.5}}{\left(1 + 0.1 \frac{B_c}{H_s} \right)} \quad (A-13)$$

where γ_{f-C} = friction factor on crest, γ_f = friction factor on seaward slope, R_c = freeboard of seaside crest, B_c = breakwater crest width, and $z_{1\%}$ = runup exceeded by 1% of waves. The friction coefficients and runup can be computed by Equations 14 and 15, respectively:

$$\gamma_f = \gamma_{f-C} = \begin{cases} 0.55 & \xi_{s,-1} \leq 2 \\ 0.05625 * (\xi_{s,-1} - 2) + 0.55 & 2 < \xi_{s,-1} < 10 \\ 1.0 & \xi_{s,-1} \geq 10 \end{cases} \quad (A-14)$$

$$\frac{z_{1\%}}{\gamma H_s} = \begin{cases} c_0 \xi_{s,-1} & \text{for } \xi_{s,-1} \leq p \\ c_1 - c_2 / \xi_{s,-1} & \text{for } \xi_{s,-1} > p \end{cases} \quad (A-15)$$

Here $c_2 = 0.25 c_1^2/c_0$, $p = 0.5 c_1/c_0$, and $\gamma = \gamma_f$ = reduction factor for roughness and angular wave attack. The Iribarren parameter based on the first negative moment wave period is

$$\xi_{s,-1} = \frac{\tan \theta}{\sqrt{\frac{H_s}{L_{m-1,0}}}} \quad (\text{A-16})$$

where

$$L_{m-1,0} = \frac{gT_{m-1,0}^2}{2\pi} \quad (\text{A-17})$$

and $c_0 = 1.45$ and $c_1 = 5.1$ for $z_{1\%}$. For the Katrina Cut rubble mound structure, Equation A-15 becomes

$$\frac{z_{1\%}}{H_s} = \begin{cases} 1.45\xi_{s,-1} & \text{for } \xi_{s,-1} \leq 1.76 \\ 5.10 - 4.485 / \xi_{s,-1} & \text{for } \xi_{s,-1} > 1.76 \end{cases} \quad (\text{A-18})$$

A.3 Reliability

Reliability is defined in terms of a limit state that characterizes the failure surface of the system. The limit state can be formulated in terms of performance, g , which is obtained by comparing the strength (R) of the system against the load (S) on the system:

$$g = R - S \quad (\text{A-19})$$

The probability of failure is given by

$$P_f = P(g \leq 0) \quad (\text{A-20})$$

and the reliability is given by

$$R_f = 1 - P_f \quad (\text{A-21})$$

Armor seaside and leeside stability and damage equations can be rearranged into resistance (strength) and forcing (load) terms. The limit

state equation for seaside armor stability was obtained by rearranging Equation A-7 as

$$g = \frac{\sqrt{\Delta} D_n S^{0.2}}{a_m} - h N_z^{0.1} \left(\frac{(M_F)_{max}}{\rho_w g h^2} \right)^{0.5} \quad (A-22)$$

Equation A-8 was rearranged to obtain the leeside damage limit state equation, expressed as

$$g = \frac{\sqrt{\Delta} D_{nls} S_{ls}^{1/6}}{a_{ls}} - N_z^{1/12} \left(\frac{u_{1\%} T_{m-1,0}}{K_{ls}} \right) \quad (A-23)$$

Appendix B: Wave transmission

The formulas for wave transmission for rock armored low crested structures by van der Meer and D'Angremond (1991) and van der Meer et al. (2005) were evaluated for application in current study. Both equations consist of empirical relationships that estimate the wave transmission coefficient (C_t), which is defined as

$$C_t = \frac{H_{m0,t}}{H_{m0,i}} \quad (B-1)$$

where $H_{m0,i}$ is the incident significant wave height and $H_{m0,t}$ the transmitted wave height.

B.1 Comparison of wave transmission formulas

The *Coastal Engineering Manual* (USACE 2002) Table VI-5-15 provides a wave transmission coefficient (C_t) equation for conventional low-crested structures developed from physical model experiments (van der Meer and D'Angremond 1991):

$$C_t = \left(0.031 \frac{H_s}{D_{n50}} - 0.24\right) \frac{R_c}{D_{n50}} + b \quad (B-2)$$

$$b = -5.42 S_{op} + 0.0323 \frac{H_s}{D_{n50}} - 0.0017 \left(\frac{B_c}{D_{n50}}\right)^{1.84} + 0.51 \quad (B-3)$$

$$S_{op} = \frac{H_s}{L_{op}} \quad (B-4)$$

$$R_c = elev_{crest} - \eta \quad (B-5)$$

where H_s = incident significant wave height at structure toe, η = deepwater water level, S_{op} = deep water wave steepness based on the peak period, L_{op} = deepwater wavelength based on the peak period, D_{n50} = seaside nominal stone size, R_c = structure freeboard, $elev_{crest}$ = structure crest elevation. The parameter b varies depending on the type of structure being analyzed; for this study, Dauphin Island's structure was modeled as a conventional structure. The results obtained from Equation B-2 are bounded to $0.075 \leq C_t \leq 0.75$ where the expected behavior is that C_t will increase as the structure becomes more submerged ($R_c < 0$) until it reaches the upper bound.

A second formulation for computing wave transmission by van der Meer et al. (2005) was considered. Based on Equation B-2, it was developed using an expanded database that included a broader range of rubble mound designs and forcing results. Wave transmission was estimated as

$$C_t = \begin{cases} -0.4 \frac{R_c}{H_s} + 0.64 \left(\frac{B}{H_s}\right)^{-0.31} * (1 - \exp(-0.5 \xi_{op})), \frac{B}{H_s} < 10 \\ -0.35 \frac{R_c}{H_s} + 0.51 \left(\frac{B}{H_s}\right)^{-0.65} * (1 - \exp(-0.41 \xi_{op})), \frac{B}{H_s} > 10 \end{cases} \quad (\text{B-6})$$

$$\xi_{op} = \frac{\tan(\theta)}{s_{op}} \quad (\text{B-7})$$

where ξ_{op} = Irribaren parameter based on the peak period and $\tan(\theta)$ = is the inverse of the structure's seaside slope. Similar to Equation B-2, the results obtained by this new relationship are bounded to $0.075 \leq C_t \leq K_{ut}$ where the upper bound limit is defined by Equation B-8.

$$K_{ut} = -0.006 \frac{B}{H_s} + 0.93 \quad (\text{B-8})$$

Applying equation for a conventional structure which truncates values at $0.075 \leq C_t \leq K_{ut}$ where K_{ut} is defined by

$$K_{ut} = -0.006 \frac{B}{H_s} + 0.93 \quad (\text{B-9})$$

Finally, with the estimated wave transmission coefficient, the transmitted wave height by overtopping at the leeside of the structure was obtained applying rearranging equation B-1:

$$H_{m0_t} = H_{m0_i} * C_t \quad (\text{B-10})$$

B.2 Assessment of wave transmission equations

Wave transmission coefficients were computed for all life cycles and time-steps using equations B-2 and B-6. The latter was selected for the wave transmission analysis based on a comparison of the results from the two equations. Figure B-1 shows an example application of Equation B-2. It can be seen that for values of R_c/D_{n50} greater than -4, Equation B-2 produces a reduction in C_t as the structure becomes more submerged. In contrast, Figure B-2 shows that Equation B-6 computed C_t values keep

increasing to its maximum as the structure becomes more submerged. Therefore, Equation B-6 was selected to compute wave transmission values in this study.

Figure B-1. Wave transmission coefficient for SB 2 using Equation B-2.

Figure B-2. Wave transmission coefficient for SB 2 using Equation B-6.

Appendix C: Wave Transmission Analysis Results (Figures)

Figure C-1. Mean maximum transmitted waves and wave breaking classification at leeside for all ST1 toe depths and RSLC configurations with a crest height of 0.91 m. Leeside depth: -0.47 m.

Figure C-2. Mean maximum transmitted waves and wave breaking classification at leeside for all ST1 toe depths and RSLC configurations with a crest height of 1.83 m. Leeside depth: -0.47 m.

Figure C-3. Mean maximum transmitted waves and wave breaking classification at leeside for all ST1 toe depths and RSLC configurations with a crest height of 2.44 m. Leeside depth: -0.47 m.

Figure C-4. Mean maximum transmitted waves and wave breaking classification at leeside for all ST2 toe depths and RSLC configurations with a crest height of 0.91 m. Leeside depth: -0.47 m.

Figure C-5. Mean maximum transmitted waves and wave breaking classification at leeside for all ST2 toe depths and RSLC configurations with a crest height of 1.83 m.
Leeside depth: -0.47 m.

Figure C-6. Mean maximum transmitted waves and wave breaking classification at leeside for all ST2 toe depths and RSLC configurations with a crest height of 2.44 m.
Leeside depth: -0.47 m.

Figure C-7. Mean maximum transmitted waves and wave breaking classification at leeside for all ST3 toe depths and RSLC configurations with a crest height of 0.91 m. Leeside depth: -0.47 m.

Figure C-8. Mean maximum transmitted waves and wave breaking classification at leeside for all ST3 toe depths and RSLC configurations with a crest height of 1.83 m. Leeside depth: -0.47 m.

Figure C-9. Mean maximum transmitted waves and wave breaking classification at leeside for all ST3 toe depths and RSLC configurations with a crest height of 2.44 m.
Leeside depth: -0.47 m.

Figure C-10. Mean maximum transmitted waves and wave breaking classification at leeside for all ST1 toe depths and RSLC configurations with a crest height of 0.91 m.
Leeside depth: -0.19 m.

Figure C-11. Mean maximum transmitted waves and wave breaking classification at leeside for all ST1 toe depths and RSLC configurations with a crest height of 1.83 m.
Leeside depth: -0.19 m.

Figure C-12. Mean maximum transmitted waves and wave breaking classification at leeside for all ST1 toe depths and RSLC configurations with a crest height of 2.44 m.
Leeside depth: -0.19 m.

Figure C-13. Mean maximum transmitted waves and wave breaking classification at leeside for all ST2 toe depths and RSLC configurations with a crest height of 0.91 m. Leeside depth: -0.19 m.

Figure C-14. Mean maximum transmitted waves and wave breaking classification at leeside for all ST2 toe depths and RSLC configurations with a crest height of 1.83 m. Leeside depth: -0.19 m.

Figure C-15. Mean maximum transmitted waves and wave breaking classification at leeside for all ST2 toe depths and RSLC configurations with a crest height of 2.44 m.
Leeside depth: -0.19 m.

Figure C-16. Mean maximum transmitted waves and wave breaking classification at leeside for all ST3 toe depths and RSLC configurations with a crest height of 0.91 m.
Leeside depth: -0.19 m.

Figure C-17. Mean maximum transmitted waves and wave breaking classification at leeside for all ST3 toe depths and RSLC configurations with a crest height of 1.83 m. Leeside depth: -0.19 m.

Figure C-18. Mean maximum transmitted waves and wave breaking classification at leeside for all ST3 toe depths and RSLC configurations with a crest height of 2.44 m. Leeside depth: -0.19 m.

Figure C-19. Mean maximum transmitted waves and wave breaking classification at leeside for all ST1 toe depths and RSLC configurations with a crest height of 0.91 m. Leeside depth: 0.38 m.

Figure C-20. Mean maximum transmitted waves and wave breaking classification at leeside for all ST1 toe depths and RSLC configurations with a crest height of 1.83 m. Leeside depth: 0.38 m.

Figure C-21. Mean maximum transmitted waves and wave breaking classification at leeside for all ST1 toe depths and RSLC configurations with a crest height of 2.44 m. Leeside depth: 0.38 m.

Figure C-22. Mean maximum transmitted waves and wave breaking classification at leeside for all ST2 toe depths and RSLC configurations with a crest height of 0.91 m. Leeside depth: 0.38 m.

Figure C-23. Mean maximum transmitted waves and wave breaking classification at leeside for all ST2 toe depths and RSLC configurations with a crest height of 1.83 m. Leeside depth: 0.38 m.

Figure C-24. Mean maximum transmitted waves and wave breaking classification at leeside for all ST2 toe depths and RSLC configurations with a crest height of 2.44 m. Leeside depth: 0.38 m.

Figure C-25. Mean maximum transmitted waves and wave breaking classification at leeside for all ST3 toe depths and RSLC configurations with a crest height of 0.91 m. Leeside depth: 0.38 m.

Figure C-26. Mean maximum transmitted waves and wave breaking classification at leeside for all ST3 toe depths and RSLC configurations with a crest height of 1.83 m. Leeside depth: 0.38 m.

Figure C-27. Mean maximum transmitted waves and wave breaking classification at leeside for all ST3 toe depths and RSLC configurations with a crest height of 2.44 m. Leeside depth: 0.38 m.

Previous figures provide the mean maximum transmitted waves in the leeside of the structure at location near the center of the structure. Nevertheless, the longitudinal east-west profile along the back of the structure at the end of the 10-year simulation period for restoration measure 5 slopes westward at the end of ST3SL3. The profiles of maximum transmitted wave heights (H_{mo_t}) based on ST2SL1 and ST3SL3 forcing and the effects of depth limitation considering their respective profiles at the end of the 10-year simulation period are shown in Figures C-28, C-29, and C-30 for structure crest heights of 0.91 m, 1.81 m, and 2.44 m.

Figure C-28. Restoration measure 5 modeling conditions and leeside bottom influence on transmitted waves for 0.91 m crest height.

Figure C-29. Restoration measure 5 modeling conditions (after 10 years) and leeside bottom influence on transmitted waves for 1.81 m crest height.

Figure C-30. Restoration measure 5 modeling conditions and leeside bottom influence on transmitted waves for 2.44 m crest height.

Acronyms and Abbreviations

1D	one-dimensional
ADCIRC	Advanced Circulation
CRL	coastal reference location
DEM	digital elevation model
FEMA	Federal Emergency Management Agency
JPM-OS	joint probability method with optimal sampling
NAVD88	North American Vertical Datum of 1988
CA1D	one-dimensional model framework
RSLC	relative sea level change
SLC	sea level change
SRR	storm recurrence rate
ST	storminess bin
SWAN	Simulating WAVes Nearshore
TC	tropical cyclone
USACE	US Army Corps of Engineers
USGS	US Geological Survey

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14. ABSTRACT Dauphin Island, a barrier island off the coast of Alabama, plays an important role in the protection of the state's coastal natural resources. In 2011, the State of Alabama constructed a rubble mound berm across a 2 km breach in the western end of the barrier island to prevent oil from the Deep Water Horizon oil spill from migrating into the Mississippi Sound. The breach, referred to as the Katrina Cut, was caused by Hurricane Katrina in 2005. The US Army Corps of Engineers and the US Geological Survey (USGS) performed the Alabama Barrier Island Restoration Assessment study to assess the current and future function of Dauphin Island and evaluate potential restoration measures. A Monte Carlo life-cycle structure response assessment of the Katrina Cut rubble mound structure was performed as part of this study by the US Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory. Damage, wave transmission, and reliability were computed within the context of the decadal barrier evolution analysis performed by the USGS for various storminess and relative sea level change scenarios. The presence of a beach in front of the structure plays an important role in its protection. The breaching potential for measures was evaluated.						
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