Performance-Based Design Methodology for Inundated Elevated Coastal Structures Subjected to Wave Load¹

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Abstract

This paper presents the method and results of a numerical study to develop a performance-based design methodology for elevated coastal structures with a focus on bridges. However, the methodology is applicable to other elevated coastal structures. A combined Eulerian-Lagrangian method for fluid-structure interaction was applied in order to compute forces on elevated coastal structures. The numerical results are in good agreement with test results of a large scale bridge section tested previously at the O.H. Hinsdale Wave Research Laboratory at Oregon State University. Specifically, a 5-m section of a prototype I-10 bridge section was used to demonstrate the fragility approach for performance-based design using four different levels of elevation. By introducing fragility modeling, a variety of design options can be considered consisting of either raising the elevation of the bridge or strengthening the structure itself in order to obtain the desired probability of failure for a specific intensity of hurricane surge and waves.

KEYWORDS: fragility, elevated structures, hurricane surge, hurricane wave, performance-based design

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1. Introduction

As a result of global climate change, many experts anticipate an increase in hurricane disasters along the coastal area of the U.S. with potential implications demonstrated by hurricane Ivan (2004), Katrina (2005) and Ike (2008), and most recently Sandy (2012). Elevated structures are one of the solutions to mitigate the damage and reduce risk to buildings and bridges along the coast by reducing the impact flow of surge and/or waves during hurricanes. In some surveys, structures fail or survive with just 0.5 m (1.6 ft) difference in elevation [1]. The wave impact forces include shear (lateral wave force), uplift from underneath the structure, and over turning moment which all can result in significant damage to many types of structures including highway bridges [2,3]. In addition, coastal and near-coast structures must resist forces due to buoyancy and hydrodynamic drag resulting from currents associated with hurricane surge.

A large body of research exists for wave loading on fixed and floating ocean and coastal structures and was conducted for offshore oil platforms, rubble-mound port structures and vertical caisson, etc. [e.g. 4,5]. Early laboratory studies in shallow water wave loading on elevated structures were conducted using small scale hydraulic models under simplified geometry and wave forces [6–9]. These studies focused on predicting the uplift forces including buoyancy, slamming force, drag force as well as inertial force. These forces reflect not only the wave characteristics but also the dynamic response of the structures and often the fluid-structure interaction between the waves and structure. Later in 1999, a series of tests on the offshore oil platforms exposed to hurricane wave-in-deck load [10] in Gulf of Mexico were conducted. Those tests showed that clearance height between still water level and the lower deck of the platform is the critical parameter for designing an elevated structure subjected to waves as one might expect. Other important parameters are the wave height crest and its probability distribution such as the

42 peakedness of the sea state. Some follow up research projects used different probability
43 distributions such as the Rayleigh distribution for the wave crests in deep water [11], wave
44 overtopping [12] or a truncated Weibull distribution for significant wave height (H_s) and peak
45 period (T_p) [13] were also conducted. However, most of the research has focused on deep water
46 waves, and there is less guidance for shallow water impacts, particularly for elevated coastal
47 structures.

48 A series of wave loading tests were conducted at the O.H. Hinsdale Wave Research Laboratory,
49 Oregon State University to determine quasi-static equations for wave loadings on structures. In
50 2011, Bradner et al. performed a test on a 1:5 scale concrete bridge superstructure section under
51 hurricane wave loads [14]. Differences between dynamic and static forces have been observed
52 experimentally. In 2013, a full scale test of light frame wood shear wall under tsunami load was
53 tested by Linton et al. [15] and confirmed that the transient loading was 2.2 times that of the
54 quasi-static force. The Goda equation [5] is felt by many to present state-of-the-art for predicting
55 the static shear and uplift loading on a vertical caisson. Following this equation, Wiebe et al. [16]
56 developed the Goda pressure formula for horizontal wave loads on elevated structures and
57 validated it with small scale tests.

58 Laboratory testing is both time consuming and costly and thus many researchers have focused on
59 development of robust computational fluid dynamics (CFD) models that can replicate test
60 results. In a CFD model, Navier Stokes equations are the governing equations and the basic tool
61 for modeling fluid dynamics in both deep and shallow water. In 2010, Bozorgnia et al. [17] used
62 the commercial CFD code STAR CCM+ to apply the finite volume method to solve the
63 governing fluid equations. Using this method, the effect of entrapped air on wave impact and
64 uplift forces on a 2D model of a bridge section was investigated. Recently, Chen et al. [18] used

another open source package called OpenFOAM to investigate wave-structure interaction. ABAQUS is a robust general commercial tool that can also be applied to this type of problem. In 2013, Como and Mahmoud [19] used the Eulerian-Lagrangian model in ABAQUS to study the impact loading on light-frame wood walls subjected to tsunami debris.

In order to introduce performance-based design for elevated coastal structures, one may consider fragility curves. Fragilities are conditional probability distributions which represent the conditional probability of the demand exceeding a specific limit state or capacity as a function of one or more hazard intensities. Constructing fragility curves for performance-based design using fragility curves is not a new design concept. In 1996, the Federal Emergency Management Agency (FEMA) funded a large project in performance-based design for buildings subjected to earthquake [20]. The fragility concept was then studied by Rosowsky and Ellingwood [21] for wood frame housing for both wind and earthquake hazard. In earthquake engineering, the hazard intensity can be the spectral acceleration for a specified fundamental period of building. In wind engineering, performance based design for wind turbine tower base connections was introduced recently by Do et al. [22].

In this paper, a procedure for performance-based design of an elevated coastal structure for hurricane waves using the fragility methodology is introduced. The ABAQUS model was validated based on existing laboratory test data. A series of numerical simulations using an Eulerian-Lagrangian formulation for a variety of combinations of significant wave height and peak wave period were conducted. Then, fragilities for four different bridge designs (elevation and inundation level) were developed. The procedure to select a combination of bridge elevation and bridge strength/capacity using fragilities to achieve the desired performance level (in this case failure probability) is then illustrated.

For a complete performance-based design methodology, a hazard intensity would be incorporated. However, only fragility curves which present the conditional probability of failure have been developed in this study for illustrative purposes. In other words, only probability of failure for a given sea state, i.e. one intensity or return period in this case was introduced. A combination of different distributions of wave height and surge levels with fragility curves and hazard intensity could be developed if one wished to consider multiple combinations of hazard intensity and performance level in the PBD.

2. Basic fluid/structure interaction model

The governing equations for fluid dynamics for a viscous incompressible fluid in an Eulerian reference frame, $x = (x_1, x_2, x_3)$ can be written as [23]

$$\frac{\partial v_i}{\partial x_i} = 0 \quad (1)$$

$$\rho_o \left(\frac{\partial v_i}{\partial t} + v_j \frac{\partial v_i}{\partial x_j} \right) = \frac{\partial}{\partial x_j} \left[-P \delta_{ij} + \mu \left(\frac{\partial v_i}{\partial x_j} + \frac{\partial v_j}{\partial x_i} \right) \right] + \rho_o g_i \quad (2)$$

where, $v = v(x, t)$ is the velocity of the fluid flow, ρ_o is fluid density, P is fluid pressure, δ_{ij} is Kronecker delta ($\delta_{ij} = 1$ if $i = j$ and $\delta_{ij} = 0$, otherwise); μ is the viscous coefficient, and g_i is gravity in the i^{th} direction.

While the equation for fluid dynamics with large deformations can be written in an Eulerian frame work which gives the flow velocity as a function of position x and time t , small deformations are typically written in a Lagrangian reference. In the Lagrangian form, the motion of a body is described by a function of time t and the reference point, X as $x = x(X, t)$. The relation between Eulerian and Lagrangian form is $v(x(X, t), t) = \frac{\partial x}{\partial t}(X, t)$ [24]. The coupled fluid/structure model in ABAQUS used in this study uses this formulation to model the structure

with a Lagrangian formulation, the fluid with a Eulerian formulation with the underlying assumption that the water can be modeled as a viscous and incompressible Newtonian fluid [25]. A linear equation of state represented in the Hugoniot form [26] was also employed to determine pressure as a function of fluid density and internal energy per unit mass. Three parameters are required to define the water using this representation, namely density (1030 kg/m^3 for sea water), the speed of sound in water (1500 m/s), and the dynamic viscosity (0.001 kg/m/s at 20°C). The Eulerian–Lagrangian general contact approach [25] was applied for this analysis with the assumption of zero friction tangential to surfaces.

3. Numerical model validation using existing wave tank data

Tsunami loads on a wood-frame wall at full scale test

Numerous experiments have been conducted in the 2-D wave flume at Oregon State University. One such test that was used for shear force validation of the ABAQUS model is the full scale light-frame wood wall test under tsunami loads tested by Linton et al. [15]. The unprocessed data was compared to those from the undamaged test of 2x6 stud wall with stud spacing of 40.6 cm subjected to offshore wave height of 0.30 m. The tsunami loading was modeled as a single wave running up to the dry and flat shore line and hitting the transverse wall. Figure 1 presents a solid model of the numerical simulation of the wave flume with a light-frame wood wall.

Only three locations were used for validating the numerical model (Figure 1). The offshore wave height at location 1 was measured by wire resistance wave gauge while onshore wave at location 2 was measured with an ultrasonic wave gauge. Wave particle velocity was measured by acoustic-doppler velocimeter at 0.09 m above the reef at location 2. The wall was equipped with 4 load cells at each corner to collect the horizontal forces on the wall. Finally, a linear variable

129 differential transformer was used at middle of the bottom of the wall (location 3) to measure the
130 wall displacement.

131 The numerical simulations were simplified slightly for computational efficiency with the idea
132 that a large number of simulations would be needed to develop fragilities eventually. The
133 transverse light-frame wood wall, as shown in Figure 1a, has dimension of 2.24x3.58 m and
134 consisted of 2x6 studs sheathed with 13 mm plywood. This wall was modeled as a flat wood wall
135 with an equivalent thickness of 65 mm to have the same stiffness as the wall with 2x6 studs and
136 plywood sheathing. The wall was modeled with shell elements with regular dimensions of
137 approximately 0.3 m. Load cells were modeled as fix supports at the four corners as shown in
138 Figure 1b.

139 In addition, the model of the flume in ABAQUS was shortened from 104 m to 35.2 m, as shown
140 in Figure 1c, to reduce computational time. The flat section of the flume bathymetry in front of
141 the wave maker was reduced from 29 m to 2.5m. The section after the wall, which is used to
142 dissipate the wave, was reduced to only 0.5 m, since numerically this could be done instantly.
143 The flume consists of two parts: sea water material and empty material. Sea water material has
144 been defined in section 2. The empty material part consisted of an empty mesh grid block. As the
145 moving wave and hit the wall, water material will occupy the empty blocks. Wave height at a
146 specific location is determined by the percentage of occupation and the height of each block in a
147 vertical column of flume. Velocity and pressure can be collected at any grid node as the direct
148 output variables from the model. As the water surface fluctuates mainly in the still water level
149 and when water hits the wall, regular meshes at these locations are at higher resolution compared
150 to other locations at shown in Figure 1d for saving computational cost.

Boundary conditions were defined as shown in Figure 1d. The flume bottom was modeled as zero velocity, which does not allow water to slide at the flume bottom. However, at side faces, only the velocity component which is perpendicular to the flume side wall was set to zero. This will allow the water to slide freely at the side walls to keep the 2D representation of the flume. At the flow-out face (flume end), pressure is set to be zero, which means water can run out freely. While a piston-type wave maker was used in the experiment, this was replaced by the velocity profile of the in-flow face in the numerical simulation. The frequency and amplitude of the input velocity was selected to generate same wave elevation that result in the experiment. Finally, general contact between the water and modeled transverse wall will allow water to move along with the wall in the horizontal direction, but move freely up in the vertical direction and drop due to gravity.

Initially, wave elevation time series were compared between the measured (tested) and the numerical model values at the toe of the slope (Location 1) which most accurately represents the offshore wave height and at 3.6 m seaward of the wall (Location 2) for the onshore wave height. The results are presented in Figure 2. From inspection of Figure 2, one can see that the model was able to reproduce the same wave as was tested in windows 2a and 2b. Other aspects such as wave velocity, wave flux and transient wave loading on the wall are also computed and presented in Figure 2c, 2d and 2e, respectively. All the results matched well with the test data. To our knowledge, this represents one of the first validations of the structural response of a full scale wood structure subjected to solitary (tsunami) force. The displacement is presented in Figure 2f, which recall was matched by using a flat wall in place of the 2x6 sheathing with plywood.

Uplift forces on a large scale bridge superstructure

While the first verification using the light-frame wood wall provided shear force validation for the modeling procedure, it was still necessary to validate the uplift forces. To accomplish this, the results from an experimental test of a 1:5 scale reinforced concrete bridge super structure subjected to hurricane waves by Bradner et al. [14] was used for comparison.

A simple layout of the test specimen can be found in Figures 3a & 3b. The bridge specimen has overall dimensions of 1.94x6.45 m and height of 0.28 m. Vertical forces (up lift loads) were measured by four vertical load cell at four corners and two horizontal load cells at two offshore corners of the specimen. The bridge was mounted to the wave flume through reaction frame which can represent both rigid and flexible connections. For this validation, only rigid connections were considered. The load cells was model as two rigid supports at offshore corners and two simple supports at onshore corners. The bridge was modeled by solid elements with normal 40 psi concrete material properties.

The model of flume was also reduced as discussed earlier to reduce computational time. Only the horizontal section with a length of 30 m was modeled as shown in Figure 3c. The same modeling technique from the previous test was applied for the flume. The mesh distribution and boundary conditions were shown in Figure 3d.

Only one test trial was used herein. This consisted of a regular single wave that was generated from $H = 0.5$ m, $T = 2$ s. The still water level was at the bottom of the bridge girders to model a level of inundation caused by surge that was observed during hurricane Katrina near New Orleans, Louisiana, USA, and was representative of the worst-case loadings. The total uplift load computed for the bridge is shown in Figure 4 compared to uplift load measured during the test.

Inspection of Figure 4 shows an excellent match in the peak force and agreement in overall shape.

Based on the two examples presented herein, one for shear force and one for uplift force verification using two different specimen materials (wood, concrete), the numerical model is felt to match the test results well, and therefore the authors are confident that it can be used to develop the fragility curves as explained in a latter section.

4. Wave power spectrum for deep and shallow water

The generation of shallow water waves is typically performed by modifying a deep water sea state power spectrum density function.

JONSWAP spectrum for deep water waves

The Joint North Sea Wave Observation Project (JONSWAP) started in 1967 and simultaneously measured the wave and wind at 160 km away from the island of Sylt in the Germany Bight, and applied the results to develop the JONSWAP spectrum for deep water sea waves [27]. The original spectral formula, $S_J(f)$, showed the fetch dependence of the wave spectrum as

$$S_J(f) = \frac{\alpha g^2}{f^5 (2\pi)^4} \exp \left[-\frac{5}{4} \left(\frac{f}{f_m} \right)^{-4} \right] \gamma^{\exp \left[-\frac{(f-f_m)^2}{2\sigma^2 f_m^2} \right]} \quad (3)$$

where γ = peak shape parameter, $\gamma = 3.3$ as an average; f_m = model frequency, $f_m = 3.5(g/U)\bar{x}^{-0.33}$, also recognized as peak frequency, f_p ; $\alpha = 0.076\bar{x}^{-0.22}$, $\sigma = 0.07$ for $f \leq f_m$ and 0.09 for $f > f_m$; $\bar{x} = gx/U_{10}^2$ dimensionless fetch; U_{10} = mean wind speed at 10 m; and g = gravity. For hurricane-associated wave spectra in the U.S., the JONSWAP spectrum

213 was modified to match the measurement of four major hurricanes [28]. The modified JONSWAP
 214 spectrum in term of significant wave height and peak frequency is expressed as

$$S(f) = \frac{4.5g^2H_s^2f_p^4}{(2\pi)^4f^5} \exp\left[-0.125\left(\frac{f}{f_m}\right)^{-4}\right] (9.5f_pH_s^{0.34})^{\exp\left[-\frac{(f-f_p)^2}{2\sigma^2f_p^2}\right]} \quad (4)$$

215 in which the two parameters, α and γ from the original spectrum can be derived as

$$216 \quad \alpha = 4.5f_p^4H_s^2; \quad \gamma = 9.5f_pH_s^{0.34};$$

217 *The Texel-MARSEN-ARSLOE (TMA) spectrum for shallow water waves*

218 The TMA spectrum was developed to reproduce wind-generated finite depth water gravity waves
 219 and was named by Hughes [29] as a result of research conducted by Bouws et al. [30], which
 220 was used in the field verification and development of the spectrum. The experiments covered the
 221 wind speed measured at 10 m ranging from 4 to 25 m/s with a sea bed slope ranging from 1:150
 222 to nearly flat, and with water depths between 5 and 45 m. The TMA spectrum effectively
 223 transforms the infinite depth sea spectrum (JONSWAP) to finite depth water, h , as

$$S_{TMA}(f, h) = S_J(f)\Phi(\omega, h) \quad (5)$$

224 where

$$\Phi(\omega, h) = \left[\frac{k^{-3}(\omega, h) \frac{\partial k(\omega, h)}{\partial \omega}}{k^{-3}(\omega, \infty) \frac{\partial k(\omega, \infty)}{\partial \omega}} \right] \quad (6a)$$

225 is the transform function. The numerical expression for $\Phi(\omega, h)$ was developed by Kitaigorskii
 226 et al [31] as

$$\Phi(\omega_h) = \{K(\omega_h)\}^{-2} \left[1 + \frac{2\omega_h K(\omega_h)}{\sinh(2\omega_h^2 K(\omega_h))} \right]^{-1} \quad (6b)$$

227 where $\omega_h = \omega\sqrt{h/g}$ and $K(\omega_h)$ can be determined from the constraint

228 $K(\omega_h) \tanh(\omega_h^2 K(\omega_h)) = 1$.

5. Non-Gaussian simulation for shallow water waves

While deep water wind-generated waves can often be treated as Gaussian, waves in shallow water are non-Gaussian due to high crests and shallow troughs as the waves near the shore and the water becomes shallower. Recent studies have focused on the simulation of non-Gaussian time series which match both the power spectral density (PSD) and the probability density function (PDF). These simulation methods vary from direct simulation to iterative methods. Direct simulation includes ARMA models for non-Gaussian signals [32,33] and Hermite polynomials which generate non-Gaussian processes from a Gaussian process [34]. Iteration methods first generate a Gaussian process from the target PSD then transform them into a non-Gaussian process. The simulated spectrum may be distorted from the target PSD and PDF. The updated PSD is then computed for the next iteration until the simulated PSD and PDF match the target. These iterative methods are well explained by Yamazaki and Shinozuka [35] and further developed by Grigoriu [36] by introducing the covariance function of the translation.

Methods for simulating non-Gaussian processes have been developed for more efficient and accurate simulation of the free surface. Deodatis and Micaletti [37] modified the Yamazaki and Shinozuka iteration algorithm in order to generate a highly skewed probability distribution. Another method introduced by Masters and Gurley [38] used cumulative distribution function mapping instead of conventional probability distribution functions. For generating a broad range of non-Gaussian processes using Monte Carlo simulation, Grigoriu [39] proposed translation models for the process which can handle both direct and inverse problems of an arbitrary prescribed marginal distribution function. Recently, a simple and efficient translation process has been developed for a general non-Gaussian stationary stochastic vector process [40], and for non-Gaussian and non-stationary stochastic processes [41,42].

The present study focuses on the original iterative method for non-Gaussian simulation proposed by Yamazaki and Shinozuka [35] simply as a tool to reproduce shallow water wave sea states. The non-Gaussian sea state was simulated from the target PSD for shallow water (TMA spectrum), and the prescribed random water wave PDF. After several iterations, the PSD of the simulated wave provides a reasonable match to the target PDS power spectrum as well as the prescribed wave height distribution. A reasonable match is defined here as having an area under the PSD of within 5% of the TMA spectrum. The expression for converting a Gaussian process to a non-Gaussian process was written as

$$\eta = F^{-1}(\Phi(\eta_o)) \quad (7)$$

where, η_o is Gaussian sea state, η represents non-Gaussian sea state, F^{-1} is the inverse of prescribed cumulative distribution function (CDF) of a non-Gaussian sea state, and $\Phi(\eta_o)$ is standard normal CDF of Gaussian sea state.

The prescribed wave distribution is traditionally assumed to be a Weibull distribution. Once the target TMA spectrum and the sea state is defined, the Biésel transformation function [43] is then applied to transform the height of the generated wave to the displacement of the wave paddle in the time domain. For the numerical flume presented by the simulation procedure described earlier in this paper, the paddle was not modeled. Instead, the movement was replaced by the paddle velocity applied directly to the fluid block as the boundary conditions at one end of the flume. Therefore, the original Biésel transformation function was modified to obtain the paddle velocity instead of the paddle displacement. The procedure for getting wave paddle velocity is as below:

1- Choose the cut-in and cut-off frequencies, f_{min} and f_{max} , and the resolution of the spectrum, N . The TMA spectrum is divided into N bins. The width of each bin denotes the frequency resolution ($\Delta f = (f_{max} - f_{min})/N$). Random sea wave are then assumed to be the combination of N single harmonic waves.

2- The i^{th} single wave has frequency f_i ($f_i = f_{min} + i\Delta f - \frac{\Delta f}{2}$). The area under each bin represents the discrete wave energy spectrum, $\sigma_\eta^2(f_i)$

$$\sigma_\eta^2(f_i) = S_{TMA}(f_i)\Delta f \quad (8)$$

3- Compute wave amplitude for the i^{th} wave

$$a_i = \sqrt{2\sigma_\eta^2(f_i)} \quad (9)$$

4- Compute time series elevation for the i^{th} wave with random phase δ_i

$$\eta_i(t) = a_i \cos(2\pi f_i t + \delta_i) \quad (10)$$

5- Compute the Biésel transform function

For hinged-type wave maker

$$B_i(z) = \frac{h+z}{h} \frac{k_i h}{2 \sinh(k_i h)} \frac{[\sinh(k_i h) \cosh(k_i h) + k_i h]}{[1 - \cosh(k_i h) + k_i h \sinh(k_i h)]} \quad (11)$$

For piston-type wave maker

$$B_i = \frac{\sinh(k_i h) \cosh(k_i h) + k_i h}{2 \sinh^2(k_i h)} \quad (12)$$

6- Transform time series sea state to time series wave paddle movement for single wave

$$x_i(t, z) = B_i(z)\eta_i(t) = B_i(z)a_i \cos(2\pi f_i t + \delta_i) \quad (13)$$

$$v_i(t, z) = B_i(z) \frac{\partial}{\partial t} \eta_i(t) = B_i(z)a_i [2\pi f_i \sin(2\pi f_i t + \delta_i)] \quad (14)$$

7- Compute the time series sea state and time series wave maker velocity

$$\eta(t) = \sum_{i=1}^N \eta_i(t); \quad x(t, z) = \sum_{i=1}^N x_i(t, z); \quad v(t, z) = \sum_{i=1}^N v_i(t, z) \quad (15)$$

8- Apply this velocity profile to the numerical flume model in ABAQUS and get the sea state spectrum at the desire location (S_i). Compare the generated wave spectrum and the

target TMA spectrum, compute the ratio $\xi = \frac{S_i}{S_{TMA}}$

9- Multiply $v(t, z)$ in step 7 with ξ and repeat step 8 until $S_i = S_{TMA}$

After obtaining the input velocities ready for wave generation, a finite element model for the water flume was generated in the ABAQUS environment. Detail of the finite element model will be discuss later in section 6. H_s and the wave spectrum at a specific location in the numerical wave flume were collected and compared to the values from the TMA spectrum. The values will also depend on the location of the measured point in the flume. Therefore, changing the measuring location allows H_s to be close to the target values. A velocity coefficient for different H_s and the height of the numerical water flume was also applied to obtain the target significant wave. This adjustment coefficient was applied to account for the simplification of the numerical wave flume boundary conditions. In this study, the bottom of wave flume was modeled as having zero velocity or no-slip condition while water can slide freely along the side walls (free slip condition) of the numerical flume.

6. Numerical simulation setup and results

Recall that the objective of this study was to illustrate a performance-based design methodology using fragilities for uplift capacity on elevated coastal structures subjected to wave loading. To accomplish this, a numerical flume, as described earlier, was utilized with a 3 m height, 5 m width and length of 100 m. The sea floor was assumed to be flat for the illustrative examples in

this study but could easily be alternated for future studies. The other end of flume was modeled as a wall with 3 m height and has zero flow velocity to store the water in a resting or still condition. Water was modeled such that it had the ability to spill out of the flume end freely when the water level was above 3 m by setting a boundary condition of zero pressure. Flow velocity was also assumed to be zero at the bottom of the flume. Figure 5 shows the numerical model setup dimensions in a generic fashion for the prototype.

Prior to investigating the wave/structure interaction, generating a sea state from a given wave spectrum is required. In order to do that, only the wave flume and water was analyzed without the presence of the structure. The model of the wave flume mesh and boundary conditions can be found in Figure 5b but without the bridge section. Again, sea water material and empty material were used to define the flume. Initially, the sea water had dimensions of 10x5x3 m. Water can freely occupy the empty material up to 6 m above the initial water level. At the bottom face, all particle velocity components were set equal to zero which will not allow water to slide freely. At the side faces, the velocity component which is perpendicular to the face was set to be zero to prevent water from running out. The out-flow face was divided into two parts. The lower part was associated with the initial water and had the same boundary conditions to that of the side face to keep the water in place. However, the higher part associated with the empty material was set to zero pressure, which means water can flow through the face when the water surface is higher than the still water level.

This solution will reduce the effect of the reflecting wave when the wave hits the end wall of the wave flume. At the in-flow face, the input velocity profile, $v_1 = vw(z, t)$, was applied to generate waves. The wave spectrum was collected along the wave flume and compare to the

target wave spectrum. The structure was then placed at the location where the generated wave spectrum fits the desire wave spectrum.

The structure was modeled as a section of the prototype I-10 bridge [14] with simple supports at beam ends. The prototype bridge section is 5m long and the geometry was simplified slightly as shown the Figure 5a. The distance, L , from wave maker to the structure was set equal to the location in the analysis where wave spectrum matched the TMA spectrum. The clearance height, d , was set to 4 different values: 0.0, 0.5, 1.0, and 1.5 m above the still water level as mentioned earlier.

The combinations of H_s and T_p used in this study for generating input velocities were from an environment contour having a 100 year return period for the Gulf of Mexico [44]. The contours were constructed based on the inverse first order reliability method. The existing data set only used a sea state with H_s equal to or greater than 8 m. Sea states with H_s smaller than 8 m were obtained by extrapolation. The peak period was modeled as conditional probability in relation to significant wave height, thus there are two values of peak period (T_p) which have the same probability of occurrence termed the upper and lower T_p (Table 1).

Figures 6 through 9 present the results for a specific sea state ($H_s = 12\text{ m}, T_p = 13\text{ s}$). The JONSWAP wave power spectrum of each sea state (H_s, T_p) for deep water waves was transformed to the TMA spectrum for a 3 m depth shallow water wave and is shown in Figure 6. From the TMA spectrum, a time series for the sea state was simulated numerically using the procedure described earlier in section 5 (step 7). This simulated time series of a sea state (water elevation) was then transformed to the frequency domain for comparison to the target spectrum

in Figure 7a. It should be noted that the term “simulated” indicates the time series wave is transformed directly from TMA spectrum, matching the target TMA spectrum as described in the procedure. Recall that the time series also follows the Weibull distribution as the prescribed distribution. Finally, through this process the velocity profile for the numerical wave maker can be obtained.

This velocity profile was then applied to the numerical water flume in step 8 to generate the numerical waves. The time series of water surface elevation measured from these numerical waves were specified as the “generated” waves as shown in Figure 7b. This generated time series was then transformed to the frequency domain to create the “generated” wave spectrum shown in Figure 7a, which also matched the desire TMA spectrum. The generated wave distribution very closely matches the simulated wave distribution and the Weibull distribution as shown in Figures 8 and 9. This generated wave spectrum can be compared to the measured wave spectrum from the experimental wave flume. The area under the spectrum represents the variance of sea surface displacement, σ^2 . The generated significant wave height was determined as $H_{sg} \cong 4\sigma$. This H_{sg} decreases as the distance from the wave maker to the measurement location increases due to energy loss.

The maximum values for shear and uplift when the wave hits the structure for four different clearance heights ($d = 0$ m, $d = 0.5$ m, $d = 1.0$ m, and $d = 1.5$ m) are illustrated in Figure 10. In this Figure, the left windows show the results for the upper peak period while the other side shows for the lower T_p values. In addition, the H_s in deep water is shown on the abscissa of each window plot and therefore represents the reference significant wave height prior to its being converted to the TMA spectrum with a time series generated for subsequent analysis. Shear and

uplift forces were obtained directly from the support reactions. In these figures, second order polynomial curves were fit to the data.

Inspection of Figure 10 indicates that wave loadings appears to escalate linearly with significant wave height in most cases. For example, an increase in significant wave height from 8 m to 10 m increases the maximum shear from 86 kN to 126 kN in Figure 10a if $d = 0.5$ m. as shown through the insert arrows. At the same time, in Figure 10b, the maximum uplift force increases 290 kN to 375 kN for $d = 0.5$ m.

From Figure 10, it is apparent that for the same significant wave height, the sea states with a lower peak period experience higher wave forces. For example, maximum shear observed in the lower T_p (Figure 10c) are almost double the case for the upper T_p (Figure 10a). In terms of maximum uplift force, it is interesting that in the upper case, maximum uplifts do not exceed the self weight of the bridge section although the wave surge level are at the bottom of the bridge girder and H_s is at a height of 12 m (Figure 10b). However, the uplift forces exceed the self weight of structure in the lower case (Figure 10d) for $d = 0$ m and $d = 0.5$ m, which may result in damage to the structure. The observation that (1) shorter periods results in substantially larger uplift forces, and (2) certain combinations of H_s and T_p results in uplift forces greater than the self-weight of the bridge, is consistent with the conclusion from Brader et al. [45] experimental work.

In addition, increasing the clearance height of the structure will also reduce the effects of wave loading and is a routinely utilized approach, but can be costly if approaches to the bridge are taken into account. For example, maximum shear decreases from 370 kN to 230 kN and 155 kN when the clearance height is increased from 0.0 m to 0.5 m and 1.0 m, respectively, for the lower

393 T_p case with $H_s = 8$ m (Figure 10c). Similar trends were observed for uplift forces. For the
 394 same conditions, the maximum uplift force falls from 840 kN to 580 kN and 450 kN (Figure
 395 10d).

396 7. Methodology for Coastal Structure Fragility Development

397 A fragility, F_r , can be expressed as

$$F_r = P[D > C|I] \quad (16)$$

398 where D is the demand, C is capacity, and I is the hazard intensity. In most cases, a fragility can
 399 be expressed [21] using the assumption of lognormal peak responses for the structure of interest
 400 as,

$$F_R(I) = \Phi\left(\frac{\ln(I) - m_R}{\xi_R}\right) \quad (17)$$

401 in which $\Phi()$ is cumulative distribution function (CDF) of the standard normal distribution,
 402 m_R, ξ_R are logarithmic mean and logarithmic standard deviation, respectively.

403 In this paper, fragility curves are constructed only for the uplift forces as a function of H_s and
 404 surge level or clearance height, d , for illustrative purposes. Further, fragility curves are only
 405 generated for the case of the lower T_p which resulted in higher significant wave heights in
 406 shallow water and larger wave loading as shown earlier in Figure 9. The probability of failure in
 407 a 300 second sea state (5 minutes) for an uplift force time series was determined by the number
 408 of maximum uplift in each event that exceeds the capacity, f_o , divided by the total number of
 409 wave events. The number of wave events in a specific time period was determined as the total
 410 time divided by the peak period for each sea state. The fragility curves were constructed by
 411 applying the lognormal fit to all the single failure points as shown in Figure 11a, 11b, and 11c for
 412 $d = 0.5$ m, $d = 1.0$ m, and $d = 1.5$ m, respectively. Three levels of capacity were specified to

illustrate three different fragility curves for each case. The levels of capacity are assumed to be 400 kN, 600 kN and 800 kN, which are equal to 60%, 90%, and 120% of the self-weight of the prototype bridge structure. The capacity level of 800 kN represents a case when the structure has been retrofitted to resist uplift force that can exceed its self-weight.

In general, probability of failure increases with the significant wave height and decreases with an increase in clearance height, d , as one might logically anticipate. When $d = 0.5$ m and $H_s = 9$ m, for example, the probabilities that the maximum uplift exceeds three different levels of capacity are 24×10^{-3} , 11.8×10^{-3} , and 6.4×10^{-3} , respectively, as shown by the arrows in Figure 10a. However, if the clearance height is raised to $d = 1.5$ m, these numbers decrease significantly to 2.3×10^{-3} , 0.15×10^{-3} , and 0.71×10^{-3} , respectively, as shown in Figure 10c.

To explain the probability of failures for a hurricane with a longer duration (e.g, 3 hours of a particular sea state), the transform action function which was modified from the equation by Luco et al. [46] as

$$P_f[\text{in } X \text{ hours}] = 1 - \left(1 - P_f[\text{in } Y \text{ hours}] \right)^{\left(\frac{X}{Y} \right)} \quad (18)$$

can be used. This indicates that the probability of failure in 3 hours is, of course, larger than that in a shorter duration, i.e. 300 seconds. The fragility curves for uplift loadings in 3 hours are presented in Figure 11d, 11e, and 11f for $d = 0.5$ m, $d = 1.0$ m, and $d = 1.5$ m, respectively.

From these figures for a known sea state, a design optimization can be established. In these figures, six levels of capacity are presented. To design a structure for uplift loading with a target probability of failure, P_f , one can adjust the uplift capacity, f_o , or change the clearance height, d . For example, a structure with an uplift capacity of 600 kN and clearance height of 0.5 m has a

probability of failure equal to 0.31 during a hurricane with $H_s = 9$ m (Figure 12). By raising the clearance height to 1.0 m, the same probability of failure can be achieved with an uplift capacity somewhere between 400 kN and 500 kN, as shown in Figure 12. By applying linear interpolation, the capacity for obtaining the same probability of failure when raising a structure to 1.0 m is approximately 450 kN.

To reduce the probability of failure, one can also obviously consider raising the clearance height to reduce the uplift force. For the same significant wave height of 9 m, if a structure with capacity of 600 kN is raised from 0.5 m to 1.0 m or to 1.5 m, the probability of failure decreases from 0.31 to 0.19 and 0.02, respectively as shown in Figure 13a. If a retrofit solution was applied to increase the uplift capacity to 800 kN (120% self-weight), the probability of failure decreases significantly as one can see in comparing Figure 13a and 13b.

It is noted that although the probabilities of failure in the examples mentioned above are significantly high, these are conditional probabilities (fragilities) and assume the occurrence of an extreme event. If an unconditional probability of failure was desired, the fragility would be convolved with a hazard curve for the location of interest. The failure probabilities computed herein were based on the assumption that hurricane with the stated significant wave height occurred for 3 hours. The clearance height of an elevated structure also depends on the surge level which could be varied in further analysis. The combination of significant wave height distribution and surge level with the proposed fragility curves will result in a complete model of probability of failure during a hurricane.

8. Summary and Conclusions

This paper introduced a performance-based design approach for elevated coastal structures when a prescribed extreme sea state is considered using fragility methodology. The closed-form numerical method for generating random shallow water waves from a target wave power spectrum and the distribution type was presented. Different hurricane sea states (H_s, T_p) for deep water (the JONSWAP spectrum) were transformed to shallow water (TMA spectrum) at a 3 m depth where the hypothetical structure was located. The numerical generation method was validated itself by comparing the output and the input wave power spectrum and the distribution type. The benefit of this method is the generation of full scale waves, which typically cannot be replicated in a laboratory. By applying the coupled Eulerian-Lagrangian model for fluid/structure interaction in ABAQUS, a numerical water flume simulation was utilized. The fragility methodology was then explored to illustrate the ability to select from various design options (combinations of clearance height and capacity) to obtain a target probability of failure for a specified hurricane/sea state. The illustrative example for a coastal bridge design based on a target failure probability demonstrates how the concepts presented herein could be applied for performance-based design of coastal structures.

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Table 1. *Sea state from 100-year PDF environmental contour for Gulf of Mexico*

	H_s (m)									
	4	5	6	7	8	9	10	11	12	12.8
upper T_p (s)	11.6	12.1	12.6	13.1	13.6	14.1	14.5	14.7	14.9	14.4
lower T_p (s)	8.3	8.8	9.3	9.8	10.3	10.8	11.5	12.2	13.0	14.4

Figure 2. Numerical validation for Tsunami loads on transverse wall test at full scale

- a) Offshore wave height at location 1
- b) Onshore wave height at location 2
- c) Wave velocity at location 2
- d) Momentum flux at location 2
- e) Total horizontal force at 4 supports
- f) Deflection at location 3

Figure 3. Test model set up for bridge subjected to waves

- a) Flume cross view
- b) Flume plan view and instruments layout
- c) Numerical Model of flume model and bridge
- d) Model flume mesh and boundary conditions

Figure 4. Comparison between tested and modeled uplift loading

Figure 5. Full scale bridge section model and numerical wave flume

- a) Bridge section and test set up
- b) Flume mesh and boundary conditions

Figure 6. Transform JONSWAP spectrum to TMA spectrum
for $H_s = 12 \text{ m}$, $T_p = 13 \text{ s}$, shallow water at height, $h = 3 \text{ m}$

Figure 7. Generated versus Simulated wave spectrum and generated sea state
for $H_s = 12\text{ m}$, $T_p = 13\text{ s}$, $h = 3\text{ m}$

(a) Frequency domain
(b) Time domain

Figure 8. Wave height distribution for $H_s = 12\text{ m}$, $T_p = 13\text{ s}$, $h = 3\text{ m}$

Figure 9. Weibull fit for generated wave height distribution

(a)

(c)

(b)

(d)

Figure 10. Maximum impact loadings on structure

- (a) Shear force, upper T_p
- (b) Uplift force, upper T_p
- (c) Shear force, lower T_p
- (d) Uplift force, lower T_p

Figure 11. Fragility curves for impact loading on structure

(a) $d = 0.5 \text{ m}, t = 300 \text{ s}$; (b) $d = 1.0 \text{ m}, t = 300 \text{ s}$; (c) $d = 1.5 \text{ m}, t = 300 \text{ s}$,
(d) $d = 0.5 \text{ m}, t = 3 \text{ hour}$; (e) $d = 1.0 \text{ m}, t = 3 \text{ hour}$; and (f) $d = 1.5 \text{ m}, t = 3 \text{ hour}$

Figure 12. Design example using fragility curves for the same probability of failure

Figure 13. Design example using fragility curves to reduce probability of failure

(a) uplift capacity $f_o = 600\text{ kN}$

(b) uplift capacity $f_o = 800\text{ kN}$