Investigating the Lateral Restraining Stiffness Effect on the Bridge Deck-Wave Interaction under Stokes Waves

Guoji Xu, A. M. ASCE¹, C.S. Cai, P.E., F. ASCE²

¹ Center for Computation & Technology, Louisiana State Univ., Baton Rouge, 70803; Division of Computer Science and Engineering, Louisiana State Univ., Baton Rouge, LA 70803. (Corresponding author) E-mail: gxu2@lsu.edu

² Edwin B. and Norma S. McNeil Distinguished Professor, Dept. of Civil and Environmental Engineering, Louisiana State Univ., Baton Rouge, LA 70803. E-mail: cscai@lsu.edu

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do the bridge deck vibrations result in smaller wave forces on deck? In other words, are there any benefits to the coastal bridge structures vulnerable to waves once the bridge superstructure (deck) is allowed to vibrate to some extent, similar to the lateral restraining setups adopted to bridges in seismic areas. Based on a methodology of the mass-spring-damper system using a dynamic mesh updating technique proposed in previous studies, a wave model based on the Stokes 2nd order wave theory is applied and verified with analytical solutions and experimental measurements in the literature, assuring a valid prediction in the following parametric study. The shear stress transport (SST) $k-\omega$ model is used as the turbulence closure for the RANS equations in order to capture the turbulence effects in the bridge deck-wave interaction. In the parametric study, general characteristics of the structural vibration and the wave forces are observed and discussed. The obtained results show that increasing the structural flexibilities in the transverse/lateral direction does not necessarily benefit the bridge structure with an obvious force reduction for both the horizontal and vertical forces on the bridge superstructure. However, this will unfortunately lead to larger horizontal forces on the interface between the bridge super and substructure and larger dynamic amplification factors for the bridge decks.

Key words: Bridge deck-wave interaction; Coastal bridges; Wave forces; Computational fluid dynamics (CFD); Lateral restraining stiffness; Stokes wave.

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Abstract: This paper addresses one fundamental question on the bridge deck-wave interaction: do the bridge deck vibrations result in smaller wave forces on the deck? In other words, are there any benefits to the coastal bridge structures vulnerable to waves once the bridge superstructure (deck) is allowed to vibrate to some extent, similar to the lateral restraining setups adopted to bridges in seismic areas? Based on a methodology of the mass-spring-damper system using a dynamic mesh updating technique proposed in previous studies, a wave model of the Stokes 2nd order wave theory is applied and verified with analytical solutions and experimental measurements in the literature, assuring a valid prediction in the following parametric study. The shear stress transport (SST) $k-\omega$ model is used as the turbulence closure for the RANS equations in order to capture the turbulence effects in the bridge deck-wave interaction. In the parametric study, general characteristics of the structural vibration and the wave forces are observed and discussed. The obtained results show that increasing the structural flexibilities in the transverse/lateral direction does not necessarily benefit the bridge structure with an obvious force reduction for both the horizontal and vertical forces on the bridge superstructure. However, this will unfortunately lead to larger horizontal forces on the interface between the bridge super and substructure and larger dynamic amplification factors for the bridge decks.

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1. Introduction

The bridge deck-wave interaction has become a focal point recently because many coastal low-lying bridges along the Gulf of Mexico in the United States have been destroyed in the last decade [1-4]. The combination of the high storm surge and the high waves induced by hurricanes inundated many spans and exerted huge hydrodynamic wave loads on the bridge decks. However, those damaged bridges were rarely designed for such kinds of wave forces. To reveal the failure mechanisms of the damaged bridges and to propose potential guidelines for better designing or retrofitting coastal bridges under such extreme natural events, many experimental and numerical studies have been conducted [5-12]. However, most of these studies focus on the rigid setups where the bridge model is fixed in the wave flume/tank experimentally and computationally. Very few studies have investigated the dynamic characteristics (flexible setups) of the bridge deck-wave interaction [9, 13-15].

Dynamic analysis for bridge deck-wave interaction problems is an emerging topic and it is of significant importance. There are two necessities on this topic: (1) the bridge deck should vibrate under the wave actions due to the structure's flexibility. The structural vibration period in the transverse/lateral direction is typically small for low-lying coastal bridges, say, less than 1 s [9]. However, the bridge-scour developed during the wave actions may increase the structural vibration period and hence result in a larger vibration displacement under the same wave conditions; and (2) similar to the mitigation ideas learned from earthquake engineering where memory alloys, base isolations, and cable restraints are used to modify the interface stiffness between the super and substructure in order to reduce the damage to the bridges, the bridge superstructure (deck) can be allowed to vibrate to some extent such that the wave forces would be reduced. There were some discussions that using flexible connections between the super and

substructures may reduce the wave induced deck forces [3]. This is based on the assumption that a larger lateral displacement of the bridge deck along the wave propagation would dissipate more wave energy. However, the general observations made by Bradner et al. [9] did not support this assumption. Thus, to reach a general consensus, a systematic dynamic analysis of the bridge deck-wave interaction is essential for designing and retrofitting coastal bridges that will experience hurricane events in their service period.

Fig. 1 shows a schematic diagram for the bridge deck-wave interaction under periodical waves, where the superstructure movement results from the substructure flexibility and the interface movement between the super and substructure. The interface portion is not plotted in this figure for clarity. The bridge superstructure is deemed as a large concentrated mass at the top of the cantilever type pile systems and would vibrate much laterally once sufficient wave forces were exerted on them. Bradner et al. [9] developed a flexible experiment setup for the bridge superstructure (a single degree of freedom (SDOF) system) using springs to represent the structural stiffness, where the total lateral restraining stiffness is concerned, without distinguishing the stiffness from the interface or substructure. Quite limited data was reported; however, some general observations were made.

Fig. 1 Schematic diagram for the bridge deck-wave interaction under hurricane induced waves. H refers to the wave height; δ is the structural displacement for the bridge deck; SWL refers to the still water level; Mud line refers to where the piles can be deemed as rigid supported. The interface is not plotted for clarity.

In the literature, many wave models (e.g., cnoidal wave, Stokes wave, solitary wave, and stream function wave) have been utilized to investigate the bridge deck-wave interaction problems. These wave models can be classified into two typical types: one is the solitary wave model with theoretically infinite wave length and wave period; and the other is the periodical wave model with finite wave length and wave period. The wave forces on deck by employing different wave models should have some differences. For example, given the same wave height, the wave forces of a cnoidal wave with a large wave length are in good agreement with those of a solitary wave; however, there are differences between the results of these two wave models when the cnoidal wave has a comparably smaller wave length [16]. The main reason is that one single wave in the cnoidal wave trains with a large wave length is similar to a solitary wave in general wave shape. However, it should be noted that part of the cnoidal wave is under the SWL, but a solitary wave is fully above the SWL [16]. As such, the results of solitary waves may not be directly used for those coastal bridges subjected to periodical waves with relatively small wave lengths. The structural vibration induced by solitary waves decreases all the way to 0, whereas the vibration induced by periodical waves during one wave period may be further incited by the following incident waves. Hence, based on previous studies considering solitary wave actions on flexible bridge decks [13], the present study specifically investigates the flexible bridge decks under one typical periodical wave model – Stokes 2nd order wave.

Bearing the fundamental question in mind, the objective of the present study is to investigate the lateral restraining stiffness effects on the bridge deck-wave interaction under Stokes 2nd order waves that can represent hurricane induced waves. At first, the wave model

based on the Stokes 2nd order wave theory is applied and verified with analytical solutions and experimental measurements in the literature. Using a dynamic mesh updating technique, a mass-spring-damper system is realized in a commercial CFD program (Fluent, Academic Version, V15.0) [17] to represent the bridge deck vibrations in the transverse/lateral direction under stokes waves. Then, a parametric study is conducted and general characteristics of the structural vibration and the wave forces are observed and discussed. Conclusions are finally given.

2. Numerical Method and Experimental Verification

2.1 Realization of the Mass-Spring-Damper System

Due to the complexity of this physical problem, there are limitations for adequately describing the bridge superstructure system and for realizing the procedure of the bridge deck-wave interaction with sufficient accuracy. In order to obtain an appropriate balance among the physical realities, model sophistication, and computational cost, two dimensional (2D) simulations that are widely adopted in the literature for this topic are used in the present study. The bridge deck model for flexible setups can be deemed as a mass-spring-damper system (as shown in Fig. 2) in order to accommodate the SDOF system. As shown in Fig. 2, the whole computational domain is partitioned into three parts, two fixed zones and one remeshing zone, and the mesh in the fixed zones (zones 1 and 2) remains the same as its original mesh. The mass-spring-damper system (Fig. 2(b)) is incorporated in the remeshing zone (Fig. 2(a)) and their combination is treated as a rigid body, which will move laterally in both directions using the layering mesh method (Fig. 2(c)). For the layering mesh method, height based method is chosen in the current study. The cell layer adjacent to the moving boundary, i.e., layer j , is split or merged with the cell layer next to it, i.e., layer i , based on the cell height in layer j (h_j). When the bridge deck moves to the left side, the cells in layer j are expanding and their height is

117 allowed to increase if $h_{min} > (1 + \alpha_s)h_{defined}$, where h_{min} is the minimum cell height for layer
 118 j , $h_{defined}$ is the predefined cell height with a constant value, and α_s is the split factor. When the
 119 bridge deck moves to the right side, the cells in layer j are being compacted and they can be
 120 compacted if $h_{min} < \alpha_c h_{defined}$, where α_c is the collapse factor. Once the condition is met, the
 121 cells in layer j are split into two layers, i.e., the new layers i and j , or merged with those in the
 122 adjacent cell layer i into one layer, i.e., the new layer j . The split factor and collapse factor used
 123 are 0.4 and 0.2, respectively.

124 (a) Remeshing zone and Fixed zone in the computational domain

126 (b) The mass-spring-damper system

(c) Enlarged view for the layering mesh method

Fig. 2 Schematic diagram of the computational domain for the mass-spring-damper system. In this system, m is the unit length weight of the bridge deck, k_s is the lateral restraining stiffness, and c is the damping coefficient.

The boundary conditions in the computational domain are as follows: the line AB is pressure outlet, keeping the pressure in the air phase the same as the operating pressure (101, 325 Pa); the line AC is velocity inlet; the bottom line CD is modeled with a no slip stationary wall condition; and the line BD is set as pressure outlet. The water phase in the computational domain for the prescribed wave conditions will be patched through the Fluent at the initialization stage. The sketch of the geometry of a typical coastal bridge deck model is also shown in Fig. 2. This prototype bridge, which will be used in the parametric study, consisting of a slab and six AASHTO type III girders, is designed to carry two traffic lanes on the deck and can be commonly found connecting coastal communities [7, 18]. The width of the superstructure is 10.45 m, the girder height is 1.05 m, and the slab depth is 0.3 m. All six of the girders, each with a width of 0.3 m, are simplified as rectangles and evenly distributed.

The vibration of the bridge deck in the x /lateral direction can be described as the following equations:

$$\ddot{x} + 2\xi\omega_0\dot{x} + \omega_0^2x = F(t)/m \quad (1)$$

where x is the deck displacement in the x direction, ξ is the damping ratio ($c = 2\xi\omega_0m$), ω_0 is the natural frequency of the bridge deck ($\omega_0 = 2\pi/T_s$), $F(t)$ is the horizontal force on the bridge deck, and T_s is the structural vibration period. Therefore, based on the known mass, vibration period, and the damping ratio of the bridge structure and the instantaneous horizontal force on the deck, the corresponding structural vibrations can be obtained. Regarding the lateral vibration of the bridge deck, the utilized SDOF system is more suitable for simply supported coastal low-lying bridges than continuously supported and long span ones which are deemed as having involvement of multiple degree of freedoms. As a matter of fact, it seems very difficult, if not impossible, currently to simulate the whole bridge superstructure in the computational domain in order to catch the features of multiple degree of freedoms due to the limitation of the computational power. However, since the lateral fundamental frequency (usually the 1st lateral mode) is the most important one for the lateral vibration, it can be decoupled from the other modes. Therefore, the SDOF system serves well for the demonstration purpose of the present study. The details regarding the realization and verification of the mass-spring-damper system can be found in Xu and Cai [13] and Xu et al. [14].

2.2 Wave Generation and Verification with Analytical Results

The bridge deck-wave interaction is of high turbulent features, where transported quantities, e.g., mass and momentum, fluctuate in time and space. The wave impact on the bridge superstructure is unsteady and stochastic, and it is actually unpredictable in detail. In this sense, turbulent models should be essentially preferred for this bridge deck-wave interaction topic. However, in order to simplify the problem, the laminar model can be used as a first approximation. The results in Xu et al. [14] show that there are no big differences between the

laminar model and the SST $k-\omega$ model (mostly less than 10% difference). It should be noted that using the laminar model can make a good approximation in some cases. However, in cases, for example, with high Reynolds number and for the wave breaking (say, wave run-up), the laminar model would produce unreliable results.

To capture the turbulent features for the bridge deck-wave interaction, the shear stress transport (SST) $k-\omega$ model is used as the turbulence closure for the RANS equations. The reasons for choosing this turbulent model over the $k-\varepsilon$ model and the Large Eddy Simulation (LES) model are: (1) While the $k-\varepsilon$ model is one of the most common turbulence model, it is more suitable for the flow with high Reynold numbers. However, the flow near the walls of the bridge model tends to be characterized as the low Reynold number flow, which may lead to unrealistic results for the $k-\varepsilon$ model. Though many wall functions can be therefore employed for the $k-\varepsilon$ model to make up this shortcoming, the SST $k-\omega$ model has its advantages over the $k-\varepsilon$ model such that the flow domain with a high Reynold number and the near wall domain with a relatively low Reynold number can be more appropriately resolved; and (2) The LES model requires more strictly for the grid mesh in the computational domain, especially for the near wall mesh, i.e., $y^+ \leq 1$, where y^+ is used to calculate the height of the first grid cell along the walls. As such, LES model is much more computationally expensive, though it can yield more turbulent features. The equations for the SST $k-\omega$ model are given as follows:

$$\frac{\partial}{\partial t}(\rho k) + \frac{\partial}{\partial x_i}(\rho k u_i) = \frac{\partial}{\partial x_j} \left(\Gamma_k \frac{\partial k}{\partial x_j} \right) + \widetilde{G}_k - Y_k + S_k \quad (2)$$

$$\frac{\partial}{\partial t}(\rho \omega) + \frac{\partial}{\partial x_j}(\rho \omega u_j) = \frac{\partial}{\partial x_j} \left(\Gamma_\omega \frac{\partial \omega}{\partial x_j} \right) + G_\omega - Y_\omega + D_\omega + S_\omega \quad (3)$$

where k is turbulent kinetic energy which determines the energy in the turbulence, ω is the specific dissipation which determines the scale of the turbulence; Γ_k and Γ_ω are the effective

diffusivity of k and ω , respectively; $\widetilde{G_k}$ represents the generation of turbulence kinetic energy due to the mean velocity gradients, calculated from G_k ; G_ω is the generation of ω ; Y_k and Y_ω are the dissipation of k and ω , respectively; D_ω is the cross-diffusion term; and S_k and S_ω are user-defined source terms.

In the present study, the Stokes 2nd order wave theory is employed to represent one typical wave type induced by hurricanes. The water particle velocities u and v and the free surface profile η of the Stokes 2nd order wave theory are expressed as follows [19]:

$$u = \frac{H}{2} \frac{gK}{w} \frac{\cosh K(h+z)}{\cosh Kh} \cos(Kx - wt) + \frac{3H^2 w K}{16} \frac{\cosh 2K(h+z)}{\sinh^4(Kh)} \cos 2(Kx - wt) \quad (4)$$

$$v = \frac{H}{2} \frac{gK}{w} \frac{\sinh K(h+z)}{\cosh Kh} \sin(Kx - wt) + \frac{3H^2 w K}{16} \frac{\sinh 2K(h+z)}{\sinh^4(Kh)} \sin 2(Kx - wt) \quad (5)$$

$$\eta = \frac{H}{2} \cos(Kx - wt) + \frac{H^2 K}{16} \frac{\cosh(Kh)}{\sinh^3(Kh)} (2 + \cosh 2Kh) \cos 2(Kx - wt) \quad (6)$$

where K is the wave number, w is the wave frequency, h is the still water depth, g is the gravitational acceleration, z is the distance from the still water level and is negative if it has the same direction as the gravitational acceleration, t is the simulation time, and x is the distance from the inlet boundary.

In the simulations, water is assumed to be an incompressible, viscous fluid, and the Volume of Fluid (VOF) method is utilized to track the dynamic free surface [17, 20]. The VOF model is a surface-tracking method applied for two or more immiscible fluids where the position of the interface between the fluids is the focal point. In this VOF model, a single set of momentum equations are solved for all the shared fluids and the volume fraction for each of the shared fluids is tracked in the whole computational domain. For the two phase problems, $\alpha_q = 1$ refers to that the mesh cell is fully occupied with the q th phase, which is the primary phase, and $\alpha_q = 0$ means that the other phase fluid, the secondary phase, occupy the very mesh cell.

However, $0 < \alpha_q < 1$ indicates that the mesh cell is shared by both fluids with their respective occupation ratio. In the current bridge deck-wave interaction simulations, air is set as the primary phase and water is the secondary phase, and α_q is determined by the equations as follows:

$$\frac{\partial \alpha_q}{\partial t} + \frac{\partial(u\alpha_q)}{\partial x} + \frac{\partial(v\alpha_q)}{\partial z} = 0 \quad (q = 1, 2)$$

$$\sum_{q=1}^2 \alpha_q = 1$$

It should be noted that the pressure-based solver should be chosen when the VOF method is employed and the VOF method is not available with the density-based solver.

For the setups of the SST $k-\omega$ model in the Fluent, the pressure-based solver (segregated) is chosen for the transient flow, and the Pressure-Implicit with Splitting of Operators (PISO) scheme is utilized for the pressure-velocity coupling method. Further details of the numerical setups can be found in Xu and Cai [13].

For the wave generation, the equations of the velocity components, u and v , and the free surface profile η are compiled into the Fluent at the inlet boundary (see Fig. 2 (a)) simultaneously using the user defined function (UDF) macros of DEFINE_PROFILE. A numerical flume with the size of 55 m in length and 3.2 m in height is chosen for the verification of the wave profiles with analytical results. Based on the mesh sensitivity study by Xu and Cai [13] and many previous studies (e.g., [7, 8, 18, 21]), several summarized guidelines need to be followed for successful simulations of the bridge deck-wave interaction: (1) To well represent the surface waves, sufficient grids need to be considered per the wave length and wave height. Practically speaking, 100 nodes in one wave length and 20 nodes in the wave height can well predict the wave shape. However, high nonlinear waves may need more dense meshes; (2) The Courant-Friedrichs-Lewy condition needs to be strictly satisfied; (3) The skewness of the cell, $\frac{dy}{dx}$

or $\frac{dx}{dy}$, needs to be smaller than 10 [17]. However, smaller values, say, 1~2, would produce more accurate results; and (4) To accurately capture the near-wall features, very fine meshes should be adopted around the walls of the bridge deck model in order to satisfy the requirement of the log-law for the “law-of-the-wall”. Further elaboration of this treatment can be found in Xu and Cai [13]. The final grid resolutions used here are: $dx= 0.02$ m, 0.01 m, and 0.04 m are used for the near velocity inlet zone, main computational zone, and far field from the main computational zone, respectively; and $dy = 0.04$ m, 0.01 m, and 0.02 m are used for the air zone, the near water zone, and the deep water zone, respectively. The time step is 0.002 s. The comparison of the wave profiles between the numerical simulations and the analytical results is shown in Fig. 3, where three wave heights, 0.43 m, 0.50 m, and 0.63 m, and one SWL, 1.89 m, are considered in order to be consistent with the experimental study by Bradner et al. [9]. Good agreement is obtained as shown in this figure.

Fig. 3 Comparisons of the free surface profiles between the current method and the analytical results

2.3 Deck Force Prediction and Verification with a Flat Deck Model

McPherson [22] considered periodic waves on a 1:20 scaled flat deck, and the result is used here for verification purpose. The experiment was conducted in the Haynes Coastal Engineering Laboratory 3D shallow water wave basin at Texas A&M University. The flat deck is fixed with 68.58 cm in width, 106 cm in length, and 1 cm in thickness, and the bottom of the flat deck is located 0.48 m above the bed. During the experimental study, end plates were placed in both ends of the flat deck to eliminate the 3D effects. Here, 3D effects specifically refer to that the wave forces on the two end parts of the bridge deck model may not equal to those on the middle part. Two typical treatments are usually adopted in order to alleviate the 3D effects, i.e., building a longer model by taking the middle part as the effective part and placing end plates at both ends of the model [6, 22].

In the verification process, three water depths, 0.48 m, 0.51 m, and 0.54 m, are considered with one wave height of 0.14 m, and the computational domain is 13 m in length $\times$ 0.9 m in height. The grid resolutions for this verification are: $dy=0.02$ m, 0.0025 m and 0.005 m for the air zone, the near water zone, and the deep water zone, respectively; and $dx=0.005$ m, 0.0025 m, and 0.02 m for the near velocity inlet zone, main computational zone, and far field from the main computational zone, respectively.

Comparisons between the results by the current method and by McPherson [22] are shown in Fig. 4, where a small difference between the maximum vertical forces is found when $d=0.48$ m. One possible reason may be because there are difficulties in keeping the same wave height of 0.14 m all along the length of the bridge deck model (1.06 m) in the wave basin. However, good agreements are observed for this verification.

Fig. 4 Comparisons of the time histories of the vertical forces on a flat deck

2.3 Deck Force Verification with a Girder Deck Model

Taking the I-10 Bridge over Escambia Bay as the prototype bridge, a 1:5 scaled bridge deck model is built and tested in the study by Bradner et al. [9]. The geometric parameters for the bridge deck model with some simplifications for the girders are shown in Fig. 5, where the width of the bridge deck is 1.94 m, the girder height is 0.23 m, and the deck depth is 0.05 m. A 0.15 m (6 inch) steel angle is used in the experimental study to consider the railing effect [23]. The bottom of the bridge deck model is placed at the SWL, i.e., 1.89 m.

Fig. 5 Dimensions of the simplified bridge deck for verification purpose

In this verification, the 2D numerical domain with the size of 55 m in length and 3.2 m in height is used, where the bridge deck model is placed at the middle of the numerical domain. The time histories of the wave forces for two typical wave heights (0.50 m and 0.63 m) are shown in

286 Fig. 6, where the dashed line refers to the peak values obtained in the experimental
 287 measurements by Bradner et al. [9]. The available experimental time history forces are plotted in
 288 Fig. 6 (a). It is noted that good agreement for the horizontal forces is obtained, whereas there are
 289 some discrepancies for the vertical forces. The reason is probably that the entrapped air in each
 290 chamber between two nearby girders underneath the deck plays as a cushion since the air cannot
 291 escape in the longitudinal direction in the 2D simulation. This would make the force duration
 292 time longer than it should be, i.e., “chubbier” in the pattern than the experimental results. This is
 293 one limitation of the 2D simulations. Letters (b) to (f) in Fig. 6 (a) refer to the snapshots of the
 294 VOF profiles in the bridge deck-wave interaction, and the corresponding snapshots are shown in
 295 Fig. 7.

Fig. 6 Comparisons of the time histories of the wave forces on a girder deck. Letters (b) to (f) in Fig. 6 (a) refer to the snapshots of the VOF profiles in the bridge deck-wave interaction, as shown in Fig. 7. The dash line refers to the averaged peak values obtained in the experimental measurements.

Fig. 7 Snapshots of the VOF profiles in the bridge deck-wave interaction. (a) $t = 0.0$ s; (b) $t = 21.5$ s; (c) $t = 22.0$ s; (d) $t = 22.5$ s; (e) $t = 23.0$ s; (f) $t = 23.5$ s. The red color refers to the water phase, while the blue color refers to the air phase.

The obtained wave forces by the current method are further compared with the experimental measurements by Bradner et al. [9], and the numerical results by Bozorgnia and Lee [10], as shown in Fig. 8, where five wave heights (0.20 m, 0.34 m, 0.43 m, 0.50 m, and 0.63m) are considered. The wave heights adopted by Bozorgnia and Lee [10] are 0.34 m, 0.43 m, 0.54 m, and 0.65 m, slightly different with those reported by Bradner et al. [9]. The polynomial relationships between the wave height and the wave forces regressed by Bradner et al. [9], $F_h = 2.431H + 6.988H^2$ and $F_v = 18.01H + 18.69H^2$, are also plotted in Fig. 8, where F_h is the horizontal force and F_v is the vertical force.

Fig. 8 Comparisons of the wave forces on a girder deck

As noted in Fig. 8, good agreements are found between the results of the current method and those by Bradner et al. [9], especially for the horizontal forces. It is also noted that the 3D simulations (Test 5) by Bozorgnia and Lee [10] estimates larger horizontal wave forces than their corresponding 2D ones (Test 1), whereas close predictions for the vertical wave forces are obtained. However, the horizontal wave forces in both of their 3D and 2D simulations are smaller than those by Bradner et al. [9]. The reason Bozorgnia and Lee [10] underestimated the horizontal forces for large wave heights is probably because the 0.15 m (6 inch) steel angle was

not considered in their study. For the vertical forces, the current method overestimates the values for small wave heights, which is probably attributed to the entrapped air between the girders underneath the deck slab. In the present 2D simulations, the entrapped air cannot escape in the direction normal to the wave propagating direction (longitudinal direction of the bridge deck), while in the experiment the entrapped air can be released in this direction in a timely manner [10, 13, 24].

3. Parametric Study

3.1 Determination of the Lateral Restraining Stiffness

In Bradner et al. [9]’s experimental study, to determine the lateral restraining stiffness in the flexible setup, a finite element analysis was conducted to choose the spring stiffness representing the natural frequency ranges in the field condition. Meanwhile, three different elevations of the superstructure, 3.05 m, 6.10 m, and 9.14 m (distance from the mud-line to the bottom soffit of the girders), were chosen. The period of vibrations was then obtained by using the linear-elastic finite element analysis. For example, the fundamental period of one bridge model with the elevation of 9.14 m and pinned at the foundation, representing the most flexible configuration, was calculated as 0.88 s, corresponding to a model value of 0.40 s based on the Froude similarity criterion. A suitable support stiffness was thereafter chosen to match this period value in the 1:5 scale model tests.

Generally speaking, the lateral restraining stiffness, represented by the elastic springs in Bradner et al. [9]’s study, consists of two parts. One is the substructure stiffness, which depends on the soil condition, the structural stiffness of the piers/piles, etc. The other one is the interface stiffness that depends on the connections between the superstructure and substructure, for example, the bearing types or restraining cables. In the present study, only the total lateral

restraining stiffness of the bridge deck, without distinguishing the substructure stiffness and the interface stiffness, is concerned, i.e., the same as that adopted in Bradner et al. [9]. The Froude similarity criterion is automatically satisfied because full scale bridge models are chosen in the computational domain.

For the parametric study, six sets of stiffnesses corresponding to six vibration periods and a damping coefficient ξ of 0.05 are chosen based on the study by Bradner et al. [9], as shown in Table. 1. In the simulations, the mass is taken as 9716 kg per unit length according to the study by Xiao et al. [7] (154000 kg/15.85 m=9716 kg per unit length). The case name is defined according to the corresponding value of the stiffness. For example, k1813 refers to the case with the lateral restraining stiffness of 1812.724 kN/m.

Table 1 Parameters for the mass-spring-damper system

Cases	T_s (s)	m (kg)	ξ	k_s (N/m)	c (N·s ² /m)
k1813	0.46	9716	0.05	1812724	13271
k998	0.62	9716	0.05	997847	9846
k630	0.78	9716	0.05	630461	7827
k425	0.95	9716	0.05	425011	6426
k170	1.5	9716	0.05	170477	4070
k96	2.0	9716	0.05	95893	3052

The wave height considered is 2.0 m, the water depth is 8.4 m, and the wave period is 5.5 s [25]. The computational domain is 10 m in height and 200 m in length. The geometry of the bridge deck model is shown in Fig. 2 (a) earlier. Six different structure elevations are considered to simulate in field conditions, as listed in Table 2, where C_s ($C_s = S/H_b$) is the coefficient of submersion depth and is negative when the bottom of the superstructure is under the SWL; S = the distance from the bottom of the bridge superstructure to the SWL; and H_b = superstructure depth. The momentum center is the moment center due to the vertical force and the horizontal force, and it is located at the middle height of the deck for each case. The abbreviation name of each case is designated according to both the bridge elevation (the value refers to the elevation of

the bottom of the girder) and the submersion coefficient; for example, E8.4/CS(0) stands for the case when the bottom of the bridge model is 8.4 m from the sea bed and the corresponding coefficient of submersion depth C_s is 0.

Table 2 Structure elevations and corresponding coefficients

Cases	Bridge elevation (Bottom of the girder) (m)	$S(m)$	$C_s = S/H_b$	Momentum Center x (m) y (m)	
E9.0/CS(0.444)	9.0	0.6	0.444	95.225	10.2
E8.7/CS(0.222)	8.7	0.3	0.222	95.225	9.9
E8.4/CS(0)	8.4	0	0	95.225	9.6
E8.1/CS(-0.222)	8.1	-0.3	-0.222	95.225	9.3
E7.8/CS(-0.444)	7.8	-0.6	-0.444	95.225	9.0
E7.5/CS(-0.667)	7.5	-0.9	-0.667	95.225	8.7

3.2 Structural Vibration

One example of the bridge deck displacement is shown in Fig. 9 where the case of E7.8/CS(-0.444) with different restraining stiffnesses starts from the original position of the bridge deck, 95.225 m, when the simulation begins. The vibration amplitude of Case k96 is relatively larger than those of other cases with higher stiffnesses. The maximum bridge deck displacements for all the six structure elevations with different restraining stiffnesses are shown in Fig. 10. The displacement values (in the landward direction) show the same trends, namely, the less restraining stiffness, the larger the deck displacement. For a given stiffness of all the six cases, the deck displacement differs from each other, which is also related to the values of the horizontal forces. This is the same as observed for solitary wave actions on flexible bridge decks [13].

Fig. 9 Structure displacement for Case E7.8/CS(-0.444)

Fig. 10 Maximum displacement of the structural vibration

3.3 Horizontal Forces without considering Inertia Forces

Wave forces on coastal structures generally refer to the net hydraulic forces without including the inertia forces from the bridge deck, i.e., the horizontal force $F(t)$ in Eq. (1). These kinds of wave forces are usually used to design and evaluate the performance of the bridge superstructure. One example of the time-history horizontal forces without considering inertia forces in Case E7.8/CS(-0.444) is shown in Fig. 11. It can be observed in this figure that the time-history force curves slightly differ from each other with different structure stiffnesses. The

401 positive horizontal forces, F_h , obtained from the time histories versus the restraining stiffness are
 402 plotted in Fig. 12 (a), and a normalized expression, $\frac{F_h}{F_h k_{1813}}$, is used to demonstrate the general
 403 trends of the horizontal forces, as shown in Fig. 12 (b). Here $F_h k_{1813}$ is the value of the positive
 404 horizontal force corresponding to the restraining stiffness of 1812.724 kN/m. It can be seen in
 405 Fig. 12 (b) that the horizontal forces vary along with the increase of the structure stiffness, but
 406 without a clear general trend. This is slightly different from the case of solitary wave actions on
 407 the flexible bridge decks reported by Xu and Cai [13], where the values using this normalization
 408 generally decrease and approach 1 with the increase of the restraining stiffness.

409 Fig. 11 Time histories of the horizontal forces for Case E7.8/CS(-0.444)
 410
 411

(a) Before normalization (b) After normalization
Fig. 12 Horizontal forces considering different restraining stiffnesses

3.4 Horizontal Forces with considering Inertia Forces

The horizontal forces with considering inertia forces from the bridge deck are those transferred from the bridge deck to the interface and substructure. Hence, they should be used to design and evaluate the bearing supporting and the substructure. The difference between the horizontal force with and without considering the inertia forces can be expressed as the difference between $F(t)$ and $F(t) - m\ddot{x}$, where $-m\ddot{x}$ is the inertia forces of the bridge superstructure, see Eq. (1). For fixed bridge decks, there is no difference between $F(t)$ and $F(t) - m\ddot{x}$ since the inertia force part ($-m\ddot{x}$) is zero. In the study by Bradner et al. [9], the wave forces were collected from the load cells and hence, the horizontal forces they obtained for the flexible setups are actually the forces including the inertia forces. In the following discussions, the inertia force part is expressed as F_I .

Fig. 13 shows an example of the comparisons of the time-history horizontal forces with and without considering inertia forces for Case E8.1/CS(-0.222). From this figure, obvious differences can be found when the restraining stiffness is relatively small (k96). However, for large restraining stiffness (k1813), the curves for the two types of horizontal forces almost

coincide with each other, indicating that the inertia force in this specific case is relatively small. This observation is attributed to that more flexible setups tend to have larger accelerations for the bridge deck under the horizontal wave forces in the same magnitude and hence result in larger inertia forces for the bridge deck. The inertia forces are then transferred from the bridge deck to the interface and substructure.

Fig. 13 Example of the comparisons of the time-history horizontal forces with and without considering inertia forces for Case E8.1/CS(-0.222)

A typical comparison of the horizontal forces with and without considering the inertia forces for different stiffnesses is shown in Fig. 14. It shows that with the increase of the restraining stiffness, these two types of horizontal forces tend to be closer with smaller difference. In addition, the horizontal forces with considering inertia forces are larger than those without considering inertia forces. As shown in Fig. 14 (a), the proportion caused by the inertia force (related to acceleration) can be clearly identified as discussed above, especially for more flexible setups. However, other factors, while not well determined, also add credits for this proportion. Specifically speaking, the horizontal forces without considering inertia forces are not constant

for different structure stiffness. Their variance along with different restraining stiffnesses is probably due to that: (1) The positive horizontal forces obtained in the current study are averaged over four wave periods. Long time simulations would produce more stable results per the averaging procedure and hence, less variance would be predicted for the different restraining stiffnesses; and (2) The bridge deck vibration may have certain effects on the horizontal forces. In Fig. 14 (b), the value of $\frac{F_h+F_I}{F_h}$ is 1.2 for Case k96, indicating that when the bridge superstructure is allowed to vibrate with a period of 2.0 s, an increment of 20 percent of the horizontal wave forces will be transferred to the bearing (interface) and the substructure. Similar observations were obtained in Xu and Cai [13] for solitary waves, but the quantitative results are quite different.

Fig. 14 One example of the comparisons of the horizontal forces with and without considering inertia forces

3.5 Vertical forces

One example of the time-history vertical forces for Case E7.8/CS(-0.444) is shown in Fig. 15. The obtained vertical forces versus the restraining stiffness are plotted in Fig. 16(a) and, similarly, the vertical forces are normalized using an expression of $\frac{F_v}{F_{v_{k1813}}}$, as shown in Fig.

466 16(b), where $F_{v k1813}$ is the vertical force corresponding to the restraining stiffness of 1812.724
 467 kN/m. The negative vertical forces are not discussed here since they are significantly smaller
 468 than the positive vertical forces. It is observed in Fig. 16 that the vertical forces do not have a
 469 clear general trend with the increase of the restraining stiffness. The values of $\frac{F_v}{F_{v k1813}}$ are quite
 470 close to 1 for the studied cases, showing that increasing the structural flexibilities does not have
 471 obvious benefits to reducing the vertical forces during the bridge deck-wave interaction. This
 472 confirms the observation in Xu and Cai [13].

473
 474 Fig. 15 Time histories of the horizontal forces for Case E7.8/CS(-0.444)
 475

(a) Before normalization (b) After normalization
Fig. 16 Vertical forces considering different restraining stiffnesses

3.6 Dynamic Amplification Factor

It is well known that in the static structural analysis, $\delta = \frac{F}{k}$, where δ is the structure displacement, F is the force applied in the direction of the structure displacement, and k is the structure stiffness. For the bridge deck-wave interaction under Stokes 2nd order waves, the dynamic effect (the amplification factor) needs to be considered, where the amplification factor is expressed as follows:

$$c_{amp} = \frac{k*\delta}{F_h} \quad (7)$$

The results of the amplification factor are plotted in Fig. 17. It can be observed in this figure that the values of $c_{amp} = \frac{k*\delta}{F_h}$ are larger than 1.0 and the general trends tend to be closer to 1.0 as the restraining stiffness increases. It is predicted that as the restraining stiffness goes to infinity, the value of $c_{amp} = \frac{k*\delta}{F_h}$ should be closer to 1.0. The same trend is reported in Xu and Cai [13], but the values of $c_{amp} = \frac{k*\delta}{F_h}$ are quite different between the results by these two types of waves.

Fig. 17 The dynamic amplification factor, c_{amp} , versus the restraining stiffness

3.7 Discussion of the Setups in the Laboratory Studies regarding Lateral Stiffness

In the laboratory studies for predicting the wave forces on bridge deck models, an important problem usually exists that there is certain lateral flexibility for the supporting structures [6, 26]. One typical experimental setup in the literature is shown in Fig. 18, where the support rods cannot be deemed as rigid structures and they will probably vibrate under the wave impacts. As such, the wave loads obtained from the load cell should include the proportion of the inertia forces, and the vibration of the bridge deck may have some effects on the values of the as-obtained wave forces. Sheppard and Marin [6] proposed a more rigid setup for developing the AASHTO code [5] and the load cell was placed very adjacent to the bridge deck model. Though the supporting steel frame would vibrate at a certain frequency, the load cell should vibrate along with the bridge deck model. Nevertheless, the support structure for acquiring the data in developing the AASHTO code [5] can be considered as a rigid setup. Therefore, attentions should be paid to the differences of the horizontal forces with and without considering the inertia forces since the deck vibrations cannot be avoided under infield conditions.

(a) Front view (b) Side view
Fig. 18 One experimental setup in the literature (adopted from Henry [26])

4. Concluding Remarks

This paper has addressed one fundamental question on the bridge deck-wave interaction: do the bridge deck vibrations result in smaller wave forces on deck? Based on a methodology of the mass-spring-damper system using a dynamic mesh updating technique proposed in the previous studies, a wave model of the Stokes 2nd order wave theory is applied and verified first with analytical solutions and experimental measurements in the literature. The SST $k-\omega$ model is used as the turbulence closure for the RANS equations in order to capture the turbulence effects in the bridge deck-wave interaction. A parametric study was then conducted to investigate the characteristics of the bridge deck-wave interaction for different structure elevations with different lateral restraining stiffnesses. Based on the numerical predictions, the following conclusions can be drawn.

(1) For the bridge superstructures with different elevations studied in the present study, a smaller restraining stiffness tends to result in a larger deck displacement as expected.

(2) When the inertia forces are ignored, there is not much difference for the horizontal forces and vertical forces with the variable restraining stiffness. Therefore, increasing the structural flexibilities in the lateral/transverse direction does not necessarily benefit the bridge structure with an obvious force reduction for both the horizontal and vertical forces on the bridge superstructure.

(3) With the increase of the structural flexibilities, larger differences can be found between the horizontal forces with and without considering the inertia forces. It is noted that in the development of the AASHTO [5] guide specifications [6], the support structure for acquiring the data was considered as a rigid setup. As such, attentions should be paid to the differences of

the horizontal forces with and without considering the inertia forces since the deck vibrations cannot be avoided under infield conditions.

(4) The amplification factor c_{amp} is larger than 1.0 as predicted for the horizontal forces with different structure elevations. In addition, the amplification factor tends to be closer to 1.0 as the restraining stiffness increases.

(5) In general, compared with the observations for solitary wave actions on flexible bridge decks [13], a slight difference exists for the general trends of the horizontal forces versus the restraining stiffness. Other than this, these two kinds of wave types share almost the same general trends for the considered aspects, but different result values.

The limitations of the current study and future work are described as follows:

(1) The methodology for the 2D numerical simulations employed in the current study may not fully capture all the features in the bridge deck-wave interaction process. The entrapped air in each chamber between two nearby girders underneath the deck, playing as a cushion between the water surface and the bottom of the bridge deck, is an issue in the 2D simulations since the entrapped air cannot escape in the longitudinal direction. Therefore, improvements and more realistic results would be expected in the 3D simulations for both the fixed and flexible setups. However, valid 3D simulations with sufficient fine model meshes require much more powerful computational capabilities.

(2) A countermeasure of air venting holes in the bridge deck-wave interaction is identified to in favor of reducing the vertical force substantially, but an increase of the horizontal force is therefore resulted [15]. The bridge deck-wave interaction with considering both the flexible setups and air venting holes would be a topic of interest and need to be further systematically studied.

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