2015

Investigating Wave Forces on Coastal Bridge Decks

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INVESTIGATING WAVE FORCES ON COASTAL BRIDGE DECKS

A Dissertation

Submitted to the Graduate Faculty of the
Louisiana State University and
Agricultural and Mechanical College
in partial fulfillment of the
requirements for the degree of
Doctor of Philosophy

in

The Department of Civil and Environmental Engineering

by
Guoji Xu
B.S., Hunan University, 2007
M.S., Hunan University, 2010
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ABSTRACT

Tsunamis and hurricane induced waves are responsible for many coastal bridge failures, especially in the last decade. In the current study, three countermeasures (reducing the entrapped air, elevating the structures and rigidifying the structures) are considered, two wave types (solitary wave theory and Stokes wave theory) are chosen, and two bridge types (single bridge deck and twin bridge decks) are taken into account. Parametric studies are conducted based on Computational Fluid Dynamics (CFD) software ANSYS Fluent.

This dissertation comprises two broad themes according to the wave types adopted. The first one is to make a great effort to investigate solitary wave forces on the coastal bridge decks, including suggesting an improved method for predicting solitary wave forces, analyzing the wave forces based on the component level, presenting a quantitative observation for the countermeasure of the air venting holes, assessing the wave forces on the bridge decks with inclinations and with different lateral restraining stiffnesses, and investigating the characteristics of the wave forces on twin bridge decks. The second one is to make exploration of Stokes wave forces on coastal bridge decks, including single bridge deck and twin bridge decks.

Based on the obtained results, interesting observations are concluded: (a) an improved method for investigating solitary wave forces on typical bridge decks is suggested and it is proven to make successful predictions; (b) the countermeasure of air venting holes can dramatically reduce the vertical force (based on quasi-static level) when the bridge superstructure is well located around the still water level (SWL); (c) while the wave forces on the landward bridge deck are generally smaller than those on the seaward deck, the interference effects due to the presence of the landward deck on the seaward deck is noticeable for stokes waves in such a way that much larger vertical forces are induced when the gap between the twin bridge decks is around half of the wave length. These observations provide potential suggestions to the future edition of the AASHTO bridge design code.
CHAPTER 1. INTRODUCTION

1.1 Background

In recent hurricanes and tsunamis, many coastal low-lying bridges bore critical damage, which was attributed to the effects of the storm surge and water wave loading, not accounted for in previous coastal bridge design. The variety of bridge damage can include the failure of the shear keys, the displacement of spans from their original positions, the washed away of spans from the supporting piles, the tilting or collapse of piles due to the movements of the spans above, the removal or tearing off of railings, and the excessive cracking in the bulky decks and girders. The severity of these incidents ranges from small visible cracks on the superstructures to the total destruction and collapse of a few spans, even the entire bridge.

In 2004, Hurricane Ivan made landfall in the Florida panhandle, causing extensive damage to the I-10 Bridge over the Escambia Bay (Sheppard and Miller 2006), as shown in Fig. 1.1. Fifty-one spans from the eastbound bridge and 12 spans from the westbound bridge were completely removed; thirty-three from the eastbound bridge and 19 spans from the westbound bridge were displaced in varying distances from their initial positions. Support structures were also damaged as twenty-five bents from the eastbound bridge and 7 bents from the westbound bridge were affected by the displacement and collapse of the above superstructures.

![I-10 Escambia Bay Bridge damage from Hurricane Ivan (2004)](image1)

In 2005, Hurricane Katrina severely damaged the gulf coasts of Louisiana and Mississippi. Three major bridges (I-10 Bridge over the Lake Pontchartrain, Louisiana, US 90
Bridge over the St. Louis Bay, Mississippi and US 90 Bridge over the Biloxi Bay, Mississippi) were brought down by the waves combined with the storm surges (Chen et al. 2005; Sheppard and Marin 2009; Bradner et al. 2011) (see Figs. 1.2 to 1.5). Twelve bridge sites of short- and medium-span bridge damaged due to hurricane Katrina were archived by Okeil and Cai (2008).

Fig. 1.2 I-10 Mobile Bay Bridge onramp spans displaced by Hurricane Katrina (2005) (adopted from Sheppard and Marin 2009)

Fig. 1.3 I-10 Lake Pontchartrain Bridge damage from Hurricane Katrina (2005) (adopted from Sheppard and Marin 2009)
In addition, the 2004 Indian Ocean Tsunami and the 2011 Great East Japan Tsunami caused thousands of casualties and catastrophes to coastal communities and structures, including many coastal bridges (Graumann et al. 2005; Shoji and Moriyama 2007; Yeh et al. 2007; Ghabarah et al. 2006; Bricker et al. 2012). It is reported that the 2004 Great Indian Ocean Tsunami caused 226,000 people (dead or missing) in countries around the Indian Ocean, and the vast seriously affected areas include Indonesia, Thailand, Malaysia, Myanmar, Bangladesh, India, Sri Lanka, and Maldives, as well as African countries. For the 2011 Great East Japan Tsunami, it is confirmed that there were 15,844 deaths and 3,393 people missing until the 17th of January in 2012. Powerful tsunami waves were caused by this earthquake and destroyed the cities in coastal area of Tohoku region. In fact, most of the infrastructures were not critically damaged by the ground motion of earthquake itself, however completely
destroyed by the massive tsunami waves. In this extreme event, more than 300 bridges experienced some type of damage.

The damaged bridges were major bridges connecting surrounding communities and the destruction of these routes was both costly to replace and a serious impact to the coastal communities served by these facilities. Unforgettable hard memories with these huge nature disasters taught engineers and researchers many lessons (Robertson et al. 2007) and drew national attention to the impact of wave forces on bridges. The loss of commerce from the destruction of the traffic routes and the cost of replacement bridges are huge (Padgett et al. 2008).

Due to the complex geometries of coastal bridge structures, different site topographies, and wave conditions, it is difficult to employ one suitable wave model in order to analyze the bridge deck-wave interaction using current design methods. As such, a few current design codes or empirical equations appear to predict wave forces on the coastal bridges (Douglass et al. 2006; Douglass and Krolak 2008; Ramey et al. 2008; AASHTO 2008).

1.2 Literature Review

Wave forces on highway bridge decks have just recently become a popular topic of interest in the engineering communities. Therefore, only limited information related directly to highway bridges damage caused by tsunamis or hurricanes can be found. However, various near shore and offshore structures with similar geometries such as plate decks and offshore platforms have received a substantial amount of attention. The information from coastal structures other than highway bridges may have a large potential of applications to wave forces on highway bridge decks.

1.2.1 Previous Work on offshore and near shore platform-kind structures

For near shore structures, flat plates and docks provide an excellent basis from which to build a work and test initial theories and their viability for other structures. However, expansion from the thin plate model to complex bridge superstructure models requires considerable studies.

El Ghamry (1963) was found to be the earliest study on vertical wave forces on docks. In his study, a number of tests were done with a submerged or partially submerged deck. Entrapped air problems were noticed in his study. However, no equations or predictive methods were presented. Wang (1970) studied vertical wave forces on horizontal plates and conducted physical model experiments in a wave basin at the Naval Civil Engineering Laboratory in Port Hueneme, California. The tested results showed a slowly-varying force along with a short-duration impact force. Equations for the slowly-varying force on the underside of the plate and the short-duration impact force were provided by Wang (1970). French (1969) studied vertical wave forces on a flat horizontal plate and found wave force types similar to Wang (1970) with both a slowly-varying pressure and a short-duration, high magnitude slamming pressure. French (1979) studied vertical wave forces on a horizontal plate using a theoretical and experimental method. The equations presented in this study were based on Bernoulli flow principles and the conservation of mass and momentum. Isaacson and Bhat (1996) conducted a theoretical/experimental study of vertical forces on a rigid, suspended plate of negligible thickness, and developed a theoretical expression mathematically similar to that by Kaplan et al. (1995).
The work done on offshore platforms in the area of wave forces on suspended elements is extensive, though the majority of this work is dedicated to cylindrical components. The listed reports in this part mainly focus on those dealing specifically with deck or platform like structures with a horizontal orientation.

Kaplan (1992) and Kaplan et al. (1995) proposed a theoretical model for predicting the wave forces on suspended cylindrical elements and suspended horizontal platform decks of negligible thickness based on the Morison Equation developed by Morison et al. (1950). The theoretical model was compared with experimental data obtained from studies of offshore platform models (Murray et al. 1995). Agreement between measured and predicted forces was good in all cases except where additional structures in the wave field in front of the test platform caused significant diffraction. Suchithra and Koola (1995) examined the forces acting on a horizontal slab for regular and freak waves. According to the physical model tests, predictive equations for the maximum forces were proposed. Suchithra and Koola (1995) stated that the wave period and the clearance height are the only variables that have an effect on the force magnitude, and found that the slamming force was noticeably reduced due to the presence of trapped air. Bea et al. (1999, 2001) concentrated on offshore platform decks that were suspended beneath the structure, specifically dealing with failed decks in the field. A theoretical equation consisting of five force components was presented. Baarholm and Faltinsen (2004) performed a numerical and experimental study on the vertical wave force on an offshore platform, similar to the study done by Lai and Lee (1989). Regular waves were used in the experiments. It was noticed that measured negative magnitudes were larger than the positive magnitudes in the measured data.

Open coastal jetties, consisting of deck or dock-like platforms suspended over supportive piles and occasionally beam or girder-like elements, are used for berthing and the loading and offloading of tankers and other sizable craft. Previous works on these structures are illustrated as follows.


1.2.2 Previous Work on Wave Forces on Bridge Superstructure

To better understand the concept of wave loading mechanisms, laboratory studies are essential for engineers and researchers. Several experiments were completed to investigate wave forces caused by various combinations of surges and waves on a physical model. A
brief review of previous theoretical and experimental research on wave forces on bridge decks is summarized and discussed below.

Denson (1978) and Denson (1980) were found to be the earliest ones who studied experimentally the wave force effects on coastal bridges after hurricane Camille (1969). However, significant discrepancies were found between these two studies (Douglass et al. 2006; Bradner et al. 2011). After a comprehensive review of previous reports on coastal structures, Douglass et al. (2006) provided empirical equations as interim guidance. In AASHTO (2008) code, after a large number of laboratory experimental tests on a 1:8 scale bridge model over Escambia Bay, parametric equations were obtained based on Kaplan’s equations (Kaplan 1992; Kaplan et al. 1995). Cuomo et al. (2009) investigated wave forces on a 1:10 scaled curved bridge superstructure under regular wave conditions in a wave basin. This study deliberately evaluated the effect of entrapped air on quasi-static and impulsive forces. Henry (2011) studied the wave forces on eight 1:30 scaled bridge models with five deck clearances in a wave tank. The bridge models include the flat deck, decks with rails or girders, decks with or without diaphragms, and decks with different size of venting holes on the diaphragms.

Most recently, a 1:5 scale reinforced AASHTO type bridge superstructure was modeled in Oregon State University (Bradner 2008, Schumacher et al. 2008, Bradner et al. 2011). In the experiment, two kinds of setups were employed, the flexible setup and the rigid setup. In addition, another study considering the flexible setup was conducted by Sugimoto and Unjoh (2006), where a single span of the I-10 Twin Span Bridge was chosen for the tidal wave test with a 1:25 scale. Two types of bearing conditions were adopted: a fixed steel bearing and a movable steel bearing. For the movable steel bearing, rubber pads were used at the bearing area, and the bridge model can move in both transverse and vertical directions. These two studies shed some light to simulate preliminary relationship between wave forces and the structure damage conditions.

As discussed earlier, it is hard to consider all aspects when designing an experimental setup for wave-structure interaction problems. Due to time consuming and the high cost of laboratory experiments, numerical approaches are more attractive and are often adopted to investigate the wave-induced forces on bridges. Commercial CFD codes are under rapid development, which provides a powerful tool to investigate wave-structure interaction problems. However, until now, very few 3D simulations have been conducted. Comparing the results from 3D and 2D cases, Bozorgnia and Lee (2012) found that the differences of the maximum vertical forces (after filtered) between Test 1 (2D) and Test 5(3D) are only 11% for $H/d = 0.44$, 6% for $H/d = 0.34$, and even less for the other 3 cases, indicating that 2D simulations could predict relatively reasonable results. Moreover, 2D simulations could save huge computational cost (Bozorgnia and Lee 2012).

Some numerical simulation works has been done on coastal bridge decks. Huang et al. (2009) investigated the characteristics of wave forces acting on the Escambia Bay Bridge decks in Hurricane Ivan based on a numerical wave-load model. Three cases of bridge deck locations versus surge water elevations were simulated, showing that the uplift wave forces play significant roles to cause bridge failures in hurricane scenarios. Xiao et al. (2010) studied effects of submersion depth on wave uplift force using a developed numerical model of linear waves. Their study provided useful suggestions for future study of coastal bridges exposed to storm surges and extreme wave conditions. Bozorgnia et al. (2010) studied a
bridge with the geometry similar to the I-10 Bridge across Mobil Bay employing solitary wave model with various wave height for the fixed bridge elevation. Jin and Meng (2011) conducted numerical simulation on wave loads on the Escambia Bay Bridge using different superstructure elevations employing Flow-3D software. Based on the experimental results by Schumacher et al. (2008), Bozorgnia et al. (2012) conducted 2D and 3D simulations of stokes wave forces on the Escambia Bay Bridge and useful results of comparisons of 2D and 3D simulations were archived. In Bricker et al. (2012), possible failure mechanisms of many bridges due to the 2011 Great East Japan Earthquake were evaluated by field surveys and numerical methods.

1.3 Mitigation Methods

In 2006, efforts by the AASHTO Subcommittee on Bridges and Structures lead to the Retrofit Manual (Modjeski and Masters 2008). It was stated that each bridge presents unique vulnerabilities and constraints that require each project to be approached individually. Retrofit strategies are divided into seven categories: 1) buoyancy load reduction; 2) wave load reduction; 3) adjacent span connection; 4) strengthening connection of superstructure to substructure; 5) strengthening substructure; 6) strengthening foundation; and 7) accepting loss of superstructure to protect substructure.

Buoyancy load reduction needs to core venting holes in a bridge deck to alleviate the entrapped air. Strengthening the connection between the superstructure and substructure is conceivable feasible. Strengthening the substructure and the foundation are not feasible for a rapid retrofit. Accepting loss of the superstructure to protect the substructure could be the most feasible alternative for a very large storm, especially if the bridge is at a low elevation. However, more efforts are still needed to quantify these retrofit methods. Hence, finding better retrofitting methods and mitigating countermeasures is the driving motivation behind this study.

Based on the literature review of recent studies on wave forces on coastal structures, three general countermeasures to mitigate the wave loadings on the bridge superstructures are proposed, as shown in Table 1.1. Since many techniques and practices are still in the research level or may be applied in some specific project, their actual functions need to be further investigated and quantified.

The basic idea of reducing entrapped air countermeasures is to add air venting holes in the external decks that are partitioned by the external girders and the adjacent inter girders, and diaphragms. As such, the entrapped air will be allowed to escape from the air pocket and therefore to reduce the wave impact forces and the buoyant force. AASHTO (2008) has considered entrapped air effect as a TAF factor when calculating the vertical quasi-static force. The TAF factor should be no larger than 1 and used to account for the reduction in the buoyancy component of the vertical quasi-static force due to entrapped air proportion of the whole chamber below the bridge deck.

\[
TAF = A_{AIR}(\%AIR) + B_{AIR} \leq 1
\]

where:

\[
A_{AIR} = 0.0123 - 0.0045e\left(\frac{Z_c}{\eta_{max}}\right) + 0.0014\ln(W/\lambda)
\]

\[
B_{AIR}
\]
\[ B_{AIR} = e^{\left(-2.477 + 1.002e^{-\left(\frac{Z_c}{\eta_{max}}\right)^2} - 0.403\ln(W/\lambda)\right)} \] (1.3)

If \( 0 < \frac{\eta_{max} - Z_c}{d_g} \leq 1 \), then \% AIR may be selected from the range \( 100 \left[ 1 - \frac{\eta_{max} - Z_c}{d_g} \right] \) to the maximum amount possible. If \( \frac{\eta_{max} - Z_c}{d_g} > 1 \), then \% AIR may be selected from the range 0 to the maximum amount possible.

| Table 1.1 Proposed countermeasures to mitigate the wave loading on coastal bridges |
|----------------------------------|-----------------|-----------------|-----------------|
| Principle                        | Reducing entrapped air | Elevating structures | Rigidifying structures |
|                                  | air venting hole | truss system | Increase structural natural frequency to reduce dynamic vibration effect |
| Principle                        | Allow air to escape from below the deck to reduce the effective air volume when partially submerged | Elevate structure to avoid wave impact | Methods easily understood and applied |
| Advantages                       | Most available type; easy to use; works well in bridge deck; let on-deck water run away | Available in bridge girder system; works very well | Reduce wave forces effectively |
| Problems                         | adverse to deck structure | Investment increased; maintenance fee needed | More foundation analysis; cost increased; wind hazards increase |
| Problems                         | Investment increased; maintenance fee needed | More foundation analysis; cost increased; wind hazards increase | Hard to quantify; high requirements of connections |
| Research methods                 | Laboratory preferred; Numerical alternatively | Laboratory method | Numerical method preferred; Laboratory alternatively |
| Research methods                 | Laboratory method | Numerical method | Numerical method preferred; Laboratory alternatively |
| Relative cost                    | Low | High | High | Medium |

Bozorgnia et al. (2010) found that air venting holes in the bridge deck and overhang could reduce both the slamming force and the quasi-static uplift force effectively. The wave energy dissipates quickly when the wave crest passes the bridge superstructures and the reduction factor for the quasi-static uplift force ranges from 53% to 71%.

For elevating structures countermeasures, numerical methods should be preferred owing to that the experimental setup has difficulties in adjusting the bridge specimen to different elevations, especially for the large scale experiments. Therefore, numerical methods are widely adopted. Xiao et al. (2010) studied the effects of different elevations on the wave forces on the Biloxi Bay Bridge decks which bore much damage during Hurricane Katrina. This study gives potential suggestions that when the bridge superstructure is located around the SWL, elevating the bridge superstructures will reduce much wave forces.

As for rigidifying structures countermeasures, Bradner et al. (2011) employed this concept to investigate the wave forces on a 1:5 scale typical coastal bridge model. The sketch of the test setups, the rigid setup and the flexible setup, is shown in Fig. 1.6. In the flexible setup, a spring was chosen with an appropriate stiffness to simulate the bridge superstructure infield conditions. The flexible setup allows the bridge deck to have a large displacement under wave actions in order to investigate the dynamic characteristics of the bridge deck-wave interaction problems. The stiffness of the spring should be changed accordingly in
order to accommodate the conditions that the superstructure may be located at different elevations.

(a) Rigid setup

(b) Flexible setup

Fig. 1.6 Elevation view of the test setups, dimensions: m (ft)
(Adapted from Schumacher et al. (2008))

1.4 Wave Theory

While hurricane induced waves can be generally considered as random by nature and can be idealized and then described by using deterministic theories, tsunamis are theoretically expressed by solitary waves. The water waves are mainly divided into two parts, linear and non-linear waves. Moreover, the water waves can be classified based on their relative depth, \( d/L \), as shown in Table 1.2, where \( d \) is the still water depth, set as the vertical distance between the ocean floor and the mean water level, and \( L \) is the wave length. Shallow water waves are constricted by the gravity and wavelength. As the deep water waves propagate to the shore, they are confined by the near shore floor profile and transformed into shallow water waves. In the process, while the wavelength begins to shorten, the wave period remains the same.
Table 1.2 Criteria for defining water depth (Chakrabarti 2005)

<table>
<thead>
<tr>
<th>Classification</th>
<th>Criterion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shallow water wave</td>
<td>$\frac{d}{L} \leq \frac{1}{20}$</td>
</tr>
<tr>
<td>Transition</td>
<td>$\frac{1}{20} \leq \frac{d}{L} \leq \frac{1}{2}$</td>
</tr>
<tr>
<td>Deep water wave</td>
<td>$\frac{1}{2} \leq \frac{d}{L}$</td>
</tr>
</tbody>
</table>

1.4.1 Linear Wave Theory

Linear wave theory (also named small-amplitude wave theory) is derived for two-dimensional, freely propagating, and periodic gravity waves, which is the simplest wave theory developed to explain the mechanics of waves. Several wave parameters are essential to be introduced as follows and these wave parameters are defined visually in Fig. 1.7.

Amplitude ($a_c$ or $a_t$): defined as the distance from the still water level (SWL) to the crest or trough of the wave height. For the linear wave theory, $a_c = a_t$.

Wave height ($a_c + a_t$): the vertical distance between a wave crest and the adjacent trough.

Wave length ($L$): the horizontal distance between two successive crests (or troughs).

Wave period ($T$): The time it takes for a wave to move a distance of one wavelength.

Wave frequency ($\omega = 1/T$): the number of wavelengths that pass a fixed point per second.

Wave celerity ($c$): speed at which a wave crest moves in the defined direction.

The results of linear wave theory equation (Sarpkaya and Isaacson 1981) are listed in Table 1.3. In Table 1.3, $k = \frac{2\pi}{L}$ is the wave number; $z = d + \eta$ is the distance from the ocean floor to the water surface, and $\eta$ is the free surface profile (vertical distance calibrated from the SWL). The dispersion relationship is used to describe the relationship among the wave number, the wave frequency, and the water depth.
Table 1.3 Results of linear wave theory equation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dispersion relationship</td>
<td>( c^2 = \frac{\omega^2}{k^2} = \frac{g}{k} \tanh(kd) )</td>
</tr>
<tr>
<td>Wave profile</td>
<td>( \eta = \frac{H}{2} \cos(kx - \omega t) )</td>
</tr>
<tr>
<td>Horizontal velocity</td>
<td>( u = \frac{\pi H \cosh(k s)}{T \sinh(k d)} \cos(kx - \omega t) )</td>
</tr>
<tr>
<td>Vertical velocity</td>
<td>( v = \frac{\pi H \sinh(k s)}{T \sinh(k d)} \sin(kx - \omega t) )</td>
</tr>
<tr>
<td>Horizontal acceleration</td>
<td>( \dot{u} = \frac{2 \pi^2 H \cosh(k s)}{T^2 \sinh(k d)} \sin(kx - \omega t) )</td>
</tr>
<tr>
<td>Vertical acceleration</td>
<td>( \dot{v} = \frac{2 \pi^2 H \sinh(k s)}{T^2 \sinh(k d)} \cos(kx - \omega t) )</td>
</tr>
<tr>
<td>Dynamic pressure</td>
<td>( p = \rho g \frac{H \cosh(k s)}{2 \cosh(k d)} \cos(kx - \omega t) )</td>
</tr>
</tbody>
</table>

1.4.2 Nonlinear waves

Nonlinearities on the wave profiles are produced owing to wind blowing, refraction and other factors. Hence, nonlinear wave theories are developed to accommodate the infield waves more accurate, such as for describing large-amplitude waves. Stokes higher order wave theories, Cnoidal wave theory, and solitary wave theory are most adopted analytical wave theories of nonlinear waves (Tedesco et al. 1999). More wave theories, such as trochoidal wave theory and hyperbolic wave theory, can be found in Sarpkaya and Isaacson (1981). Compared with linear waves, nonlinear waves have higher and sharper wave crests and shallower and longer troughs with retaining higher order terms during the derivation.

The water particle velocities \( u \) and \( v \) and the free surface profile \( \eta \) of the Stokes 2\textsuperscript{nd} wave theory are expressed as follows:

\[
\begin{align*}
    u &= \frac{H g k \cosh(k h)}{2} \frac{\cos(k x - \omega t)}{\cosh(k h)} + \frac{3 H^2 \omega k \cosh(2 k (h + z))}{16} \frac{\cos(2(k x - \omega t))}{\sinh^2(k h)} \tag{1.4} \\
    v &= \frac{H g k \sinh(k h)}{2} \frac{\sin(k x - \omega t)}{\cosh(k h)} + \frac{3 H^2 \omega k \sinh(2 k (h + z))}{16} \frac{\sin(2(k x - \omega t))}{\sinh^2(k h)} \tag{1.5} \\
    \eta &= \frac{H}{2} \cos(k x - \omega t) + \frac{H^2 k \cosh(k h)}{16} \frac{\cos(2(k x - \omega t))}{\sinh^2(k h)} (2 + \cosh(2 k h)) \tag{1.6}
\end{align*}
\]

where \( k \) is the wave number, \( \omega \) 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 with the gravitational acceleration, \( t \) is time, and \( x \) is the distance from the defined original point. The dispersion relationship retains valid for the second-order Stokes waves, as listed in Table 1.3. However, the dispersion relationship becomes invalid for the third-, fourth-, fifth-, and higher-order Stokes waves, and correction terms must be considered in the dispersion relationship (Lin 2008). More information about the dispersion equation can be found in related documents (Sarpkaya and Isaacson 1981; Lin 2008). In most of the engineering analysis, the fifth-order Stokes waves are considered as sufficient to make adequate predictions (Lin 2008).
While Stokes higher order wave theories are used to be chosen for nonlinear waves in deep and transition water depth, cnoidal wave theory may be a better choice with much sharper crests and flatter troughs as compared with Stokes higher order waves under the shallow water conditions. Here, the Ursell number, \( U_r = H L^2 / d^3 \), is employed to determine which wave theory is a better model, Stokes wave theory or cnoidal wave theory. When \( U_r > 25 \), cnoidal wave theory is a better choice, and vice versa (Aguiniga et al. 2008).

The water particle velocities, \( u \) and \( v \) and the free surface profile \( \eta \) under a cnoidal wave are expressed as follows (Lin 2008):

\[
\eta = \eta_t + H cn^2 \left[ 2K(k) \left( \frac{x}{L} - \frac{L}{T} \right), k \right]
\]

\[
\frac{u}{\sqrt{gd}} = -\frac{5}{4} + \frac{3(\eta_t + d)}{2d} - \frac{(\eta_t + d)^2}{4d^2} + \left[ \frac{3H}{2d} - \frac{(\eta_t + d)H}{2d^2} \right] cn^2(0) - \frac{H^2}{4d^2} cn^4(0) - \frac{8HK^2(k)}{L^2} \left[ \frac{d}{3} - \frac{(\eta_t + d)^2}{2d} \right] \left[ -m \cdot sn^2(0) \cdot cn^2(0) + cn^2(0) \cdot dn^2(0) - sn^2(0) \cdot dn^2(0) \right] \tag{1.7}
\]

\[
\frac{v}{\sqrt{gd}} = (z + d) \frac{2HK(k)}{Ld} \cdot sn(0) \cdot cn(0) \cdot dn(0) \left\{ 1 + \frac{(\eta_t + d)}{d} + \frac{H}{d} cn^2(0) + \frac{32K^2(k)}{3L^2} \left( d^2 - \frac{(\eta_t + d)^2}{2} \right) m \cdot sn^2(0) - m \cdot cn^2(0) - dn^2(0) \right\} \tag{1.8}
\]

where:

\( cn \) is the Jacobian elliptic function associated with the cosine;

\( \eta_t = \frac{H}{m} \left[ 1 - m - \frac{E(k)}{K(k)} \right] \), the distance between the trough and the SWL (always negative);

\( cn^2(0) = cn^2 \left[ 2K(k) \left( \frac{x}{L} - \frac{L}{T} \right), k \right] \);

\( sn^2(0) = 1 - cn^2(0) \);

\( dn^2(0) = 1 - m[1 - cn^2(0)] \);

\( K(k) \) is the complete elliptic integral of the first kind with modulus \( k = \sqrt{m} \) and \( E(k) \) is the corresponding second kind with the same modulus.

Solitary waves are regarded as weakly nonlinear and dispersive waves, where the wave nonlinearity is well balanced by the wave dispersion. Hence, the solitary waves can propagate a long distance without much shape distortion and energy loss. Solitary waves are used to typify the leading wave front of tsunamis (Lin 2008). The water particle velocities \( u \) and \( v \), water pressure \( p \), and free surface profile \( \eta \) of the solitary wave of the 2nd-order (Sarpkaya and Isaacson 1981) are expressed as follows:

\[
\eta = \varepsilon \text{sech}^2 q - \frac{3}{4} \varepsilon^2 \text{sech}^2 q \text{tanh}^2 q \tag{1.10}
\]

\[
\frac{u}{\rho gd} = \frac{\eta}{d} + 1 - \frac{s}{d} - \frac{3}{4} \varepsilon^2 \text{sech}^2 q \left\{ \left( \frac{s}{d} \right)^2 - 1 \right\} (2 - 3\text{sech}^2 q) \tag{1.11}
\]

\[
\frac{v}{\sqrt{gd}} = \varepsilon \text{sech}^2 q + \varepsilon^2 \text{sech}^2 q \left\{ \frac{1}{4} - \text{sech}^2 q - \frac{3}{4} \left( \frac{s}{d} \right)^2 \right\} (2 - 3\text{sech}^2 q) \tag{1.12}
\]

\[
\frac{v}{\sqrt{gd}} = \varepsilon \sqrt{3\varepsilon} \left( \frac{s}{d} \right) \text{sech}^2 q \text{tanh} q \left\{ 1 - \varepsilon \left[ \frac{3}{8} + 2\text{sech}^2 q + \frac{1}{2} \left( \frac{s}{d} \right)^2 (1 - 3\text{sech}^2 q) \right] \right\} \tag{1.13}
\]

where \( \varepsilon = \frac{H}{d} \), \( q = \frac{\sqrt{3\varepsilon}}{2d} \left( 1 - \frac{5}{8} \varepsilon \right) (x - ct) \), \( s = y + d \), \( d \) is the still water depth, \( H \) is the wave height, and \( y \) is the distance from the still water level and is negative if it has the same direction with the gravitational acceleration.
Fig. 1.8 Recommended wave theory selection (adopted from Sarpkaya and Isaacson 1981)

The wave celerity \( c \) is calculated as:

\[
\frac{c}{\sqrt{gd}} = 1 + \frac{1}{2} \varepsilon - \frac{3}{20} \varepsilon^2
\]  

(1.14)

To make reasonable estimates of the wave forces on the coastal structures, appropriate wave theory should be chosen based on the as-obtained infield parameters, such as the wave height, wave period, wave length, and water depth, as shown in Fig. 1.8. However, it should be noted that no unique results can be obtained according to the characteristics of interest for the specified projects.

1.5 Overview of the Dissertation

The main objective of this dissertation is to investigate the tsunami and hurricane induced wave forces on coastal bridge superstructures considering different infield conditions: one single bridge deck with or without deck inclinations, the bridge deck considering the lateral restraining stiffness under different wave conditions, and the wave forces on the twin bridge decks. There are nine chapters in this dissertation based on papers that have been published, are under review, or are to be submitted to peer-reviewed journals. A brief summary of each chapter is presented as follows.

Chapter 2 gives efforts to develop an improved method to calculate the solitary wave forces based on the examined empirical methods from the previous studies. The general characteristics of solitary wave-induced forces on a typical coastal bridge deck with girders are obtained. The results may help gain better understanding of the bridge deck-wave interaction under solitary waves. Moreover, researchers may further improve the suggested method proposed in the current study to other kinds of bridge decks with different geometries and to the bridge decks under hurricane events.
Chapter 3 demonstrates the component level-based analysis of the solitary wave forces on bridge superstructures. Differences between the wave forces on the bridge decks considering the same wave height but different SWLs are found and analyzed with comparisons between the wave forces on each girder and each partitioned deck. The countermeasure of the air venting holes is investigated with different venting ratios. Interesting observations are found, indicating that this countermeasure benefits the bridge decks significantly when the bridge structure is well around the SWL.

Chapter 4 studies the solitary wave forces on typical coastal bridge decks with various inclinations. Based on the extensive study under the prescribed conditions, the general trends of the characteristics of wave forces with variable deck inclinations were observed, accompanied with analysis of the normalized ratios of the forces on a specific inclined deck to those on a level deck. The results shed some lights on the engineering problems concerning the wave loading on the bridge superstructures with different super-elevations, especially on the ramps.

Chapter 5 presents the analysis of the dynamic characteristics in the bridge deck-wave interaction problems considering the various lateral restraining stiffnesses in order to simulate different infield conditions of bridge decks. The mass-spring-damper system is implemented into Fluent to represent different restraining stiffnesses. The results show there is a big difference between the horizontal wave forces with and without considering inertia forces that is introduced by the mass of the superstructure and the acceleration of the mass in the deck movement. Plus, increase of the restraining effect does not necessarily reduce the horizontal forces and the vertical forces. Meanwhile, the submersion coefficient (the relative elevation between the bridge superstructure and the SWL) plays a significant role. These obtained results will help bridge engineers to better understand the bridge deck-wave interaction problem under tsunami conditions.

Chapter 6 intends to supply a quantitative understanding of the solitary wave forces on typical coastal twin bridge decks and to fill a gap that there are very rare studies concerning the wave forces on the twin bridge decks under solitary waves, if any. The obtained factors based on the wave forces on the seaward deck may give engineers one possible criterion for judging or rating how much wave forces will be exerted on the landward deck and what is the extent of damage on the twin bridge decks.

Chapter 7 illustrates the numerical replication of a large-scale bridge deck subjected to Stokes waves based on one experimental study conducted in Oregon State University, where a relatively large 1:5 scale reinforced concrete bridge superstructure model was built and special experimental setups were chosen to represent different dynamic characteristics of the field bridge. Numerical wave models based on the Stokes 1st order and 2nd order wave theory were first developed to replicate the wave conditions in the laboratory. By taking advantage of this precious experimental data, the numerical results verify the capability of the numerical methodology for predicting bridge performance under wave actions. The dynamic characteristics of the flexible setups were also analyzed and the simulation results agree well with the Oregon Experiment observations.

Chapter 8 describes the hydrodynamic interference effects on coastal twin bridge decks under hurricane waves. The results show that at most times the wave forces on the landward bridge deck are comparably smaller than those on the seaward bridge deck due to
the hydrodynamic interference effects. For the cases considering different gaps, the superimposed waves (consisting of the coming wave and the reflected wave) may weaken the vertical forces on the seaward deck when the reflected wave travels a distance of about half wave length from leaving the seaward deck to arriving at the seaward deck the second time, and may strengthen the vertical forces when the reflected wave travels about one wave length from leaving the seaward deck to arriving at the seaward deck the second time. However, the superimposed waves may play a more significant role for horizontal forces when the twin bridge decks are partially submerged than that when the twin bridge decks are just above the SWL.

Chapter 9 summarizes all of the studies performed in this dissertation and recommends possible future studies based on findings of this dissertation.

1.6 References


CHAPTER 2. AN IMPROVED METHOD FOR PREDICTING SOLITARY WAVE FORCES ON A TYPICAL COASTAL BRIDGE DECK WITH GIRDER

2.1 Introduction

Two recent natural disasters, the 2004 Indian Ocean Tsunami and the 2011 Great East Japan Tsunami, have refreshed people’s memories of the devastating tsunami impacts on coastal communities and structures, including many coastal bridges, and cost billions of dollars (Sugimoto and Unjoh 2006; Shoji and Moriyama 2007; FHWA 2008). It is reported that a large portion of the damaged coastal bridges, though withstood the Great East Japan Earthquake, were destroyed by the following tsunami waves (Maruyama et al. 2013). Due to the complex geometries of coastal bridge structures and other variable parameters, such as the bridge site bathymetry, the clearance between the bottom of the superstructure and the still water level (SWL), and the tsunami wave stages (breaking or non-breaking), it is difficult to analyze tsunami wave forces (vertically and horizontally) on bridges using current design methods (AASHTO 2008; Douglass and Krolak 2008). As such, it is of significant importance to further reveal the failure mechanisms of coastal bridges and develop possible guidelines for retrofitting or designing coastal bridges in tsunami-prone areas.

French (1969, 1979), Iradjpanah (1983), Lai (1986), and Lai and Lee (1989) mainly focused on the incident wave (deemed as the solitary wave) induced forces on horizontal platforms and elevated slabs. These useful observations have shed some lights on the understandings of the solitary wave forces on bridge superstructures. Recently, the devastating damage of bridges due to the tsunamis motivated more research on the bridge deck-wave interaction problems since the last decade (McPherson 2008; Bozorgnia et al. 2010; Seiffert et al. 2014; Hayatdavoodi et al. 2014). However, the documented observations and qualitative analytical results seem not much straightforward and convenient for practical applications. Further treatment is needed on the as-obtained findings in order to provide guidelines for practical engineering projects based on the previous studies. In addition, very few current design codes deal with solitary wave generated forces on structures, if any. Hence, recommended calculation equations based on the comprehensive data and concrete validations are very valuable.

As such, the objective of the current study is to study the solitary wave forces on a typical coastal bridge deck with girders and develop possible design guidelines for assessing the wave loadings on such kinds of bridge decks in the tsunami prone zones. It is found that Douglass et al. (2006), McConnell et al. (2004), Cuomo et al. (2007), and Boon-intra (2010) established empirical formulae for predicting wave induced forces on coastal structures (including the bridge decks) other than solitary waves and McPherson (2008) developed a method to assess the wave loadings under both the periodical waves and solitary waves. In the current study, the appropriateness of these procedures will be examined and expanded, if necessary, to the cases of the solitary wave induced forces on a typical coastal bridge deck with girders. It should be noted that the tsunami breaker bores (wave breaking) on the coastal bridge decks are not in the scope of the current study, though this topic is still at its early stage (Thusyanthan and Martinez 2008; Lao et al. 2010; Lau et al. 2011; Shoji et al. 2011).

Focusing on the objective of the current study, in the rest parts of this paper, the solitary wave model based on the 2nd-order solitary wave theory, the governing equations, the numerical wave model setups in ANSYS Fluent (v14.5, Academic Version), and the
wave model validations are introduced. The Shear Stress Transport (SST) $k-\omega$ model is adopted as the turbulent closure for the RANS equations. Different structure elevations are chosen to study the bridge deck-wave interaction with different wave heights. Then, the time histories of the horizontal forces and vertical forces are demonstrated and analyzed. In addition, an improved method, based on the examined empirical formulae to simply calculate the solitary wave forces, is suggested. Technical justification of the suggested method is also provided and conclusions of the current study are finally presented.

2.2 Methodology and Validation

Most of the previous laboratory studies investigate relatively small scale physical models due to the limitations of the wave tank. It is generally believed that larger scale tests result in more reliable data. Meanwhile, computational methods are under fast development and are widely adopted for investigating the bridge deck-wave interaction problems. In theory, full scale numerical models can be easily realized and adjusted according to different projects. However, until now, very few 3D simulations have been conducted due to the intensive computation power needed. Bozorgnia and Lee (2012) showed that the differences of the maximum vertical forces between Test 1 (2D) and Test 5 (3D) are only 11% for $H/d = 0.44$, 6% for $H/d = 0.34$, and even less for the other 3 situations, indicating that 2D simulations can provide reasonably accurate results. Moreover, 2D simulations could save significant computational cost (Bozorgnia and Lee 2012). For these reasons, in the present study, 2D numerical simulations are adopted to study the wave loadings on coastal bridges.

2.2.1 Governing Equations of the Wave Model

For the turbulent flow simulations, water is assumed as an incompressible, viscous fluid. The fluid motion is described based on the Navier-Stokes equations, which are shown as follows:

\[
\rho \left( \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} \right) = S_m \tag{2.1a}
\]

\[
\rho \left( \frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} \right) = - \frac{\partial p}{\partial x} + \mu \left( \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) + S_x \tag{2.1b}
\]

\[
\rho \left( \frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} \right) = - \frac{\partial p}{\partial y} - \rho g + \mu \left( \frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) + S_y \tag{2.1c}
\]

where $\rho$ is the mass density, $u$ and $v$ are the velocity components, $p$ is the pressure, $\mu$ is the viscosity, $g$ is the gravitational acceleration, $S_m$ is the mass source, and $S_x$ and $S_y$ are the momentum sources in the $x$ direction and $y$ direction, respectively.

To account for the turbulent fluctuations in the bridge deck-wave interaction problem, the RANS equations are used to describe the turbulence effects and the SST $k-\omega$ model is used as the turbulence closure for the RANS equations with the equations as follows.

\[
\frac{\partial}{\partial t} (\rho k) + \frac{\partial}{\partial x_i} (\rho ku_i) = \frac{\partial}{\partial x_j} \left( \Gamma_k \frac{\partial k}{\partial x_j} \right) + \overline{G_k} - Y_k + S_k \tag{2.2a}
\]

\[
\frac{\partial}{\partial t} (\rho \omega) + \frac{\partial}{\partial x_j} (\rho \omega u_j) = \frac{\partial}{\partial x_j} \left( \Gamma_\omega \frac{\partial \omega}{\partial x_j} \right) + G_\omega - Y_\omega + D_\omega + S_\omega \tag{2.2b}
\]

where $\Gamma_k$ and $\Gamma_\omega$ are the effective diffusivity of $k$ and $\omega$; $\overline{G_k}$ represents the generation of turbulence kinetic energy due to the mean velocity gradients, calculated from $G_k$; $G_\omega$ is the

21
generation of $\omega$; $Y_k$ and $Y_\omega$ are the dissipation of $k$ and $\omega$, respectively; $D_\omega$ is the cross-diffusion term; and $S_k$ and $S_\omega$ are user-defined source terms. All the expressions of the parameters can be referred to the theory guide in Fluent and the constants are set as default values.

For the setups of the SST $k$-$\omega$ model in Fluent, the pressure-based solver (segregated) is chosen for the transient flow, the Pressure-Implicit with Splitting of Operators (PISO) scheme (FHWA 2009; Bricker et al. 2012) is utilized for the pressure-velocity coupling method, and the PRESTO (PREssure STaggering Option) scheme is set for the pressure spatial discretization. The turbulence damping is turned on and the damping factor is set 50. For the velocity inlet boundary, the turbulent intensity is 2% and turbulent viscosity ratio is 10%. For the top and outlet of the calculation domain (see Fig. 2.1), the backflow turbulent intensity and the backflow turbulent viscosity ratio are the same as that set for the velocity inlet boundary. As a two-phase problem, the VOF (Hirt and Nichols 1981) method is employed to prescribe the dynamic free surface. Least squares cell based scheme is used for the gradient discretization, second order upwind for momentum advection terms, and Geo-Reconstruct for the volume fraction equations. Second order upwind is also used for the spatial discretization of the turbulent kinetic energy and the specific dissipation rate.

### 2.2.2 Theory of the 2nd Order Solitary Wave

The water particle velocities $u$ and $v$, water pressure $p$, and free surface profile $\eta$ of the solitary wave of the 2nd-order (Sarpkaya and Isaacson 1981) are expressed as follows:

\[
\eta = \frac{2\eta}{d} = \varepsilon \text{sech}^2 q - \frac{3}{4} \varepsilon^2 \text{sech}^2 q \tanh^2 q \tag{2.3a}
\]

\[
\frac{p}{\rho gd} = \frac{\eta}{d} + 1 - \frac{s}{d} - \frac{3}{4} \varepsilon^2 \text{sech}^2 q \left[\left(\frac{s}{d}\right)^2 - 1\right] (2 - 3 \text{sech}^2 q) \tag{2.3b}
\]

\[
\frac{u}{\sqrt{gd}} = \varepsilon \text{sech}^2 q + \varepsilon^2 \text{sech}^2 q \left(\frac{1}{4} - \text{sech}^2 q - \frac{3}{4} \left(\frac{s}{d}\right)^2 (2 - 3 \text{sech}^2 q)\right) \tag{2.3c}
\]

\[
\frac{v}{\sqrt{gd}} = \varepsilon \sqrt{3 \varepsilon} \left(\frac{s}{d}\right) \text{sech}^2 q \tanh q \left[1 - \varepsilon \left(\frac{3}{8} + 2 \text{sech}^2 q + \frac{1}{2} \left(\frac{s}{d}\right)^2 (1 - 3 \text{sech}^2 q)\right)\right] \tag{2.3d}
\]

where $\varepsilon = \frac{H}{d}$; $q = \frac{\sqrt{3 \varepsilon}}{2d} \left(1 - \frac{5}{8} \varepsilon\right) (x - ct)$, $s = y + d$, $d$ is the still water depth, $H$ is the wave height, and $y$ is the distance from the SWL and is negative if it is in the same direction with the gravitational acceleration. Hence, the wave celerity $c$ can be calculated as:

\[
\frac{c}{\sqrt{gd}} = 1 + \frac{1}{2} \varepsilon - \frac{3}{20} \varepsilon^2 \tag{2.4}
\]

It can be observed from Eqn. (2.3a) that the solitary wave crest is located at $x = 0$ when $t = 0s$, namely, the wave crest is just at the inlet boundary. To more appropriately simulate the wave profile, the incident solitary wave should be shifted leftward by replacing $t$ with $t - t_0$, where $t_0 = L_{min}/c$ and $L_{min}$ is defined as the minimum length to allow the wave crest to reach the inlet boundary after a certain time. In this way the water surface could increase gradually at the inlet boundary. $L_{min}$ should be greater than the effective wave length $L_e$, where $L_e = 2\pi d / \sqrt{\frac{3H}{d}}$. This method was adopted from Dong and Zhan (2009).
2.2.3 Numerical Calculation Domain and Boundary Conditions

Fig. 2.1 shows the schematic diagram for the computational domain of the 2D cases, where the line EF is the SWL, which separates the regions of the air phase and water phase at the initial point. The geometry of a typical coastal bridge deck model that with six girders is introduced here firstly for the convenience of discussion, and the numerical simulations employing this bridge model will be described later. This prototype bridge designed to carry 2-lane on the deck consists of a slab and six AASHTO type III girders supporting the slab and can be commonly found connecting island communities (Hayatdavoodi et al. 2014). The width of the superstructure is 10.45m, the girder height is 1.05m, and the deck depth is 0.3m. All the six girders, each with a width of 0.3m, are simplified as rectangles. The railing effect will be considered later.

The boundary conditions are the same for all the simulations in the present study and are specified as follows:

AB: pressure outlet. This keeps the pressure in the air phase being the static gauge pressure that is the same as the operating pressure (101,325 pascal).

AC: velocity inlet. The Eqns. of $u$ (2.3c) and $v$ (2.3d) are compiled into Fluent as the velocity inlet components in the $x$ and $y$ directions, respectively, by the User Defined Functions (UDF). The free surface profile $\eta$ is controlled by Eqn. (2.3a).

CD: No slip stationary wall condition.

BD: pressure outlet.

2.2.4 Wave Model Validation with Theoretical Results

To better calculate the wave loads on coastal structures and predict the structural responses under tsunami wave conditions, wave models should be validated and examined with theoretical results. During this validation process, the computation domain is 14m (length) $\times$ 0.7m (height). Model sensitivity studies are conducted and different mesh resolutions, $dx = 0.005m$ and $0.001m$ in the $x$ direction and $dy = 0.01m$ and $0.0025m$ in the $y$ direction are used, respectively. Time steps of 0.001s and 0.005s are studied. The results show that there are no significant differences on the wave profiles and wave forces. Therefore, the cell dimensions $dx = 0.005m$ in the $x$ direction and $dy = 0.01m$ in the $y$ direction are selected for further studies and the fixed time step $dt = 0.005s$ is adopted.
Fig. 2.2 shows the comparisons of the free surface profiles between the numerical results and the analytical solutions for $\varepsilon$ being 0.12, 0.18, 0.24, 0.30, 0.36, and 0.42, respectively. The plots show that the numerical results agree very well with the analytical solutions even for a high value $\varepsilon = 0.30$. While when $\varepsilon = 0.36$, a phase difference and wave decay between the results of the turbulent flow and the analytical method can be observed, which become larger when $\varepsilon = 0.42$.

The critical reasons for this phenomenon are believed to include: (a) While the theoretical equations of the solitary waves are derived from the Navier-Stokes equations based on the in-viscid fluid assumption, there are limitations to the accuracy of the turbulent flow simulations in the numerical model; (b) The effects caused by the higher order terms beyond the 2nd-order terms in the analytical model may be prominent when larger ratios of $\varepsilon$ are considered. However, the results show that the maximum difference of the wave crest of the surface profiles between the current method and the analytical solutions is 5% when $\varepsilon = 0.42$, and much less when $\varepsilon = 0.36$. Thus, valid results can be expected when the wave propagates to the bridge model with the surface profile much close to that of the prescribed wave.

2.2.5 Model Validation for the Wave Uplift Force on a Horizontal Platform

French (1969) conducted laboratory experiments to investigate the wave uplift forces acting on a platform caused by solitary waves under several ratios of $\varepsilon$, namely, 0.24, 0.28, 0.32, 0.36, and 0.40, as shown in Fig. 2.3. In the figure, $F_s$ is the weight of water in the approaching solitary waves above the platform, marked as shaded water area; and $d$ is the water depth. This experimental setup was widely employed to validate numerical results by many researchers, including Lai (1986), Xiao and Huang (2008), Huang and Xiao (2009) and Bozorgnia et al. (2010). The results of this laboratory experiment are also used to verify the present procedures before they are used to predict the wave forces on the bridge decks.

The parameters in the setup are as follows. The computation domain is 14m (length) $\times$ 0.7m (height). The still water depth $d$ is 0.381m; the solitary wave height $H$ is 0.24 $d$, i.e. 0.0914m; the distance from the bottom of the platform to the still water surface, $S$, is 0.2 $d$, i.e. 0.0762m; the length of the cross section of the platform $L_W$ is 4 $d$, i.e. 1.524m; and the height of the cross section of the platform is 0.2m. Here in this particular example, the wave profiles of the numerical simulations and theoretical ones are expected to agree with each other well for $\varepsilon = 0.24$ according to Fig. 2.2. The ratio of $x/d$ is around 20 at the position of the platform, indicating the prescribed wave profiles can be expected.

Based on the log-law for the “law-of-the-wall” used for identifying the viscous layer, blending layer, and the fully turbulent layer, very fine meshes are adopted near the walls of the horizontal platform and the time step is chosen according to the requirements of Courant Number. To take full advantage of the SST $k-\omega$ model, $y+$ should be less than 2, where $y+$ is used to calculate the height of the first grid cell along the walls of the platform model in the turbulent flow. While it is very difficult to satisfy this requirement, the height of the first grid should be in the logarithmic layer and valid results can still be produced. After a few trials of the mesh sensitivity studies, the grid resolutions are set as: $dy=0.02m$, 0.0025m and 0.005m for the air zone, the near water zone, and the deep water zone, respectively; $dx=0.005m$, 0.0025m, and 0.02m for the near velocity inlet zone, main computational zone, and far field from the main computational zone, respectively. The time step is set as $dt=0.0025s$. 

\[ 24 \]
Fig. 2.2 Comparisons of the free surface profiles for solitary waves near the location of the bridge model (the bridge model is placed at around 35m in the $x$ direction from the inlet boundary)
Fig. 2.3 Experimental setup of French (1969)

Fig. 2.4 shows the wave profile and the horizontal velocities at $t = 5.0 \text{ s}$ when $\varepsilon = 0.24$. Moreover, the horizontal velocities at 3 different positions $x = 5.946 \text{ m}$, $6.446 \text{ m}$ and $6.946 \text{ m}$ are investigated and compared with the theoretical results. Here, $x = 6.446 \text{ m}$ is the location of the wave crest. There is an 8% maximum difference in the wave profiles. The horizontal velocities agree with each other very well, with the maximum difference 5% when $x = 5.946 \text{ m}$ and the maximum difference 8% when $x = 6.446 \text{ m}$ for the current method.

The predicted time histories of the uplift force are compared with other numerical and experimental results (French 1969; Lai and Lee 1989; Huang and Xiao 2009; Bozorgnia et al. 2010) in Fig. 2.5. In this figure, $F_s$ is the integrated result based on the first order solitary wave theory and it is $88.37\text{ N}$, $399.31\text{ N}$ and $796.62\text{ N}$, corresponding to $\varepsilon$ of 0.24, 0.32 and 0.40, respectively.

(a) Wave profile $t = 5.0 \text{ s}$

(b) Comparison of wave profiles at $t = 5.0 \text{ m}$

(c) $x = 5.946 \text{ m}$

Fig. 2.4 Comparisons of wave profiles and horizontal velocities at $t = 5.0 \text{ s}$ when $\varepsilon = 0.24$
Fig. 2.4 (continued) Comparisons of wave profiles and horizontal velocities at $t = 5.0$ s when $\varepsilon = 0.24$

Fig. 2.5 Comparisons of uplift force between different studies
The results show that there is a negligible difference between the maximum results, but considerable differences of the minimum results between the experimental study by French (1969) and the turbulent flow model adopted in the present study are observed. Explanations of the large difference between the minimum results may be that the inertia effects play an important role for the turbulent flow. The minimum results happen when the wave passes underneath the platform, and are affected by the inertia effects in the turbulent flow. The maximum results may be dominated by the wave speed when the wave impacts the front of the platform.

It can also be observed that a larger value of $\varepsilon$ leads to closer results to those by French (1969). The height of the water above the platform to calculate $F_s$ is 2.0 cm, 4.572 cm and 7.62 cm, for $\varepsilon = 0.24, 0.32$ and 0.40, respectively. Therefore, the smaller $\varepsilon$ makes the $F_s$ smaller, which results in more sensitive comparisons. In addition, the Iso-Surface used to separate the air phase and the water phase can be more accurate if more vertical grids are adopted in the wave height. However, in the current study the same grid mesh is employed for different $\varepsilon$, which may lead to this phenomenon. Generally speaking, the comparisons show that the uplift forces generated by the current method agree well with the other similar studies, indicating that the present wave models could be further employed for predicting wave forces on bridge decks.

### 2.3 Numerical Results of the Wave Forces

In simulating the wave forces induced by solitary waves, the geometric parameters of the bridge deck model shown in Fig. 2.1 are used. The computation domain is 200 m (length) $\times$ 13 m (height). Fig. 2.6 shows an example of the model grid mesh adopted in the computational domain. The grid resolutions are: $dx=0.05$ m and $dy=0.05$ m for the zone nearby the bridge model; $dx=0.2$ m and $dy=0.05$ m for the near water surface zone at the far field from the bridge model; $dx=0.2$ m and $dy=0.1$ m for the deep water zone; and $dx=0.2$ m and $dy=0.2$ m for the air zone at the far field from the bridge model.

The meshes near the walls of the bridge model satisfy the requirement that the height of the first grid should be in the logarithmic layer. Structured meshes are mainly used. The total meshed cells are around 240,000.
In the present study, eight different bridge elevations and six different wave heights are analyzed by employing the numerical wave models as seen in Tables 2.1 and 2.2. In Table 2.1, the submersion coefficient $C_s$ is defined as the ratio of $S$ (the distance between the bottom of the superstructure to SWL (negative if the structure is submerged in the water)), to $H_b$ (the height of the bridge superstructure). The momentum center is the moment center due to the vertical force and horizontal force, and it is located at the middle height of the deck for each case. The still water depth $d$ is 7.22 m and the range of the bridge elevations (distance from the seabed to the bottom of the superstructure) is from 4.52 m to 7.89, representing a large variety of bridge elevations that can be normally seen in coastal areas. In Table 2.2, it is noticed that the higher the wave height is, the faster the wave travels and the less calculation time needed for one simulation.

Table 2.1 Structure elevations and corresponding coefficients

<table>
<thead>
<tr>
<th>Case</th>
<th>$S$(m)</th>
<th>$C_s = S/H_b$</th>
<th>Momentum Center</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.67</td>
<td>0.5</td>
<td>35.225</td>
</tr>
<tr>
<td>2</td>
<td>0.3</td>
<td>0.22</td>
<td>35.225</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>0</td>
<td>35.225</td>
</tr>
<tr>
<td>4</td>
<td>-0.67</td>
<td>-0.5</td>
<td>35.225</td>
</tr>
<tr>
<td>5</td>
<td>-1.35</td>
<td>-1</td>
<td>35.225</td>
</tr>
<tr>
<td>6</td>
<td>-1.65</td>
<td>-1.22</td>
<td>35.225</td>
</tr>
<tr>
<td>7</td>
<td>-2.02</td>
<td>-1.5</td>
<td>35.225</td>
</tr>
<tr>
<td>8</td>
<td>-2.7</td>
<td>-2</td>
<td>35.225</td>
</tr>
</tbody>
</table>

Table 2.2 Wave cases and related parameters for numerical simulations

<table>
<thead>
<tr>
<th>Wave</th>
<th>$H$(m)</th>
<th>$\varepsilon = H/d$</th>
<th>$L_e$(m)</th>
<th>$c$(m/s)</th>
<th>$t_0 = L_{\text{min}}/c$(s)</th>
<th>calculation time $t$(s)</th>
<th>$\Delta t$(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>3.00</td>
<td>0.416</td>
<td>40.63</td>
<td>9.95</td>
<td>5</td>
<td>16</td>
<td>0.002</td>
</tr>
<tr>
<td>II</td>
<td>2.60</td>
<td>0.36</td>
<td>43.65</td>
<td>9.77</td>
<td>6</td>
<td>17</td>
<td>0.002</td>
</tr>
<tr>
<td>III</td>
<td>2.20</td>
<td>0.305</td>
<td>47.5</td>
<td>9.6</td>
<td>6</td>
<td>18</td>
<td>0.002</td>
</tr>
<tr>
<td>IV</td>
<td>1.74</td>
<td>0.241</td>
<td>53.4</td>
<td>9.4</td>
<td>7</td>
<td>18</td>
<td>0.002</td>
</tr>
<tr>
<td>V</td>
<td>1.30</td>
<td>0.18</td>
<td>61.7</td>
<td>9.1</td>
<td>8</td>
<td>19</td>
<td>0.002</td>
</tr>
<tr>
<td>VI</td>
<td>0.87</td>
<td>0.12</td>
<td>75.45</td>
<td>8.90</td>
<td>9</td>
<td>22</td>
<td>0.002</td>
</tr>
</tbody>
</table>

2.3.1 Time Histories of Wave Forces

In this section, the wave forces of the selected bridge decks due to solitary waves are analyzed. All the forces are net forces acting on the bridge superstructure model. Fig. 2.7 demonstrates the results of the horizontal forces and the vertical forces for all the eight cases when the wave height is 1.74 m.

For the horizontal forces as shown in Fig. 2.7(a), the peak value varies with the change of the submersion depth. However, in the study by Jin and Meng (2011), the peak horizontal forces (landward) seem to be constant with all different deck elevations, which is different from what is observed from the current numerical simulations. This is probably due to the different wave types adopted in the current study and in the study by Jin and Meng (2011) who used Stoke 5th order waves. In the present study, the maximum positive horizontal force occurs at Case 6 (Fig. 2.7(a)); that is when the top of the superstructure is
submerged into the water by 0.3 m. The maximum negative horizontal force (seaward) occurs at Case 4. The positive peak horizontal force is about 1.5 times to 2.0 times of the negative peak horizontal force for each case.

To resist the horizontal forces, many practical countermeasures, such as the shear keys, are commonly adopted in the coastal bridges. Douglass et al. (2006) predicted that the total resistance provided by the bolt system per span is about 890 kN (200 kips) to 1779 kN (400 kips), much larger than the predicted result from the current study, 676 kN per span (42.66 kN/m, for Case 6 with the wave height 3.00 m). It can be concluded that the horizontal force generated by a 3.00m solitary wave only cannot cause much damage to the bridge bolt system and then the superstructure. However, the positive peak vertical force (upward) for Case 6 with the corresponding wave height is higher than the self-weight of the bridge (see Fig. 2.10 later, \( F_b = 95.3 \) kN), 1.8 times of the bridge’s self-weight, which could easily displace or move the superstructure. The maximum one of the positive peak vertical forces occurs when the bottom of the superstructure is around the SWL.

![Fig. 2.7 Demonstration of the time histories of solitary wave forces](image)

(a) \( H=1.74 \) m, Horizontal  
(b) \( H=1.74 \) m, Vertical

Fig. 2.7 Demonstration of the time histories of solitary wave forces

![Fig. 2.8 Bridge deck-wave interaction for Case 2 with the solitary wave height 1.74 m.](image)

(a) \( t = 0.0 \) s; (b) \( t = 8.0 \) s; (c) \( t = 9.0 \) s; (d) \( t = 10.0 \) s; (e) \( t = 11.0 \) s; (f) \( t = 12.0 \) s.

30
Interactions of the wave and bridge deck are shown in Fig. 2.8 for Case 2 with the solitary wave height 1.74 m. In this case, the clearance $S$ is 0.3 m and the wave height is 0.09 m above the top of the bridge deck (the elevation of the top of the deck is 7.22 m+0.3 m+1.35 m). With the propagation of the wave from the left side, the vertical force and horizontal force reach their positive peak values at around 9.4 s and 10s as shown in Fig. 2.7 (b) and (a), respectively. The wave crest reaches at the seaward beam at about 10.2 s ($30/9.4+7$), where 30 m is the distance of the seaward beam to the inlet boundary, 9.4 m/s is the wave celerity $c$, and 7s is the time calculated from $t_0 = L_{\text{min}}/c$), which shows that the positive peak vertical force and horizontal force occur when the wave front strikes the bridge superstructure. At about 11.5 s (when the wave crest just passes the landward beam), the horizontal forces appear to be negative (Fig. 2.7 (a)) and then back to zero. Because the air phase is incompressible, the entrapped air is obvious between the beams as shown in Fig. 2.8.

2.3.2 Effects of Submersion Depths on Wave Forces

The results of the positive peak horizontal forces and vertical forces with different submersion depths for different wave heights are shown in Figs. 2.9 and 2.10, respectively. Here, only the positive peak horizontal forces require attention because the absolute value of the negative peak horizontal forces, generally speaking, are smaller than the corresponding positive peak horizontal force. The maximum horizontal forces occur at Case 6, i.e., $C_s = -1.22$, for the six wave heights studied as shown in Fig. 2.9, the same as observed in Fig. 2.7(a).

![Fig. 2.9 Variation of positive peak horizontal forces per unit length with submersion coefficient for different wave heights](image)

In addition, the maximum positive peak vertical force appears at Case 3 ($C_s = 0$) for $H = 2.20$ m, 2.60 m, and 3.00 m, and at Case 5 ($C_s = -1.0$) for $H = 0.87$ m, 1.30 m, and 1.74 m. It is found that when the submersion coefficient falls in the range from -1.0 to 0, the positive peak vertical forces are relatively larger. For the trend of the positive peak vertical force, a similar phenomenon was found by Xiao et al. (2010). However, Xiao et al. (2010) just analyzed the effect of submersion depths with one fixed wave
height of linear waves. In their study, the maximum positive peak vertical force is noticed when the submersion coefficient is -1.0.

It is also interesting to notice that the positive peak vertical forces surpass the bridge’s self-weight when $H = 3.00$ m, 2.60 m, and 2.20 m for the eight cases studied. For the cases when $H = 0.87$ m, only in one case when the submersion coefficient is -1.0 that the positive peak vertical force surpasses the bridge’s self-weight. For Case 3, i.e., when the bottom of the superstructure is just at the SWL, the positive peak vertical force is 2.11, 1.93, 1.67, 1.40, and 1.16 times of the bridge’s self-weight, when $H = 3.00$ m, 2.60 m, 2.20 m, 1.74m, and 1.30 m, respectively.

![Fig. 2.10 Variation of positive peak vertical forces per unit length with submersion coefficient for different wave heights. ($F_v$ refers to the positive peak vertical force and $F_b$ refers to the self-weight of the bridge deck per unit length)](image)

### 2.3.3 Effects of the Railing Height on Wave Forces

Further comparisons are conducted by including the effects of railing. The railing heights of 0.3m and 0.6m are added to the original bridge model for cases 1, 2 and 3 with the solitary wave height 2.20 m. In Table 2.3, taking the value when the railing height is 0 m as the referenced value, the force ratios are listed accordingly. This table shows that the positive peak vertical forces and horizontal forces tend to increase with the increase of the railing height. In addition, the railing has larger effects on the horizontal forces than on the vertical forces.

### 2.4 Comparisons with Previous Empirical Methods

As discussed earlier, the bridge deck-wave interaction problem is a very complicated process which involves various wave conditions and different structure geometries. While a few empirical formulae regarding wave forces on bridge decks are established (Douglass et al. 2006; McPherson 2008; AASHTO 2008; Boon-intra 2010), many methods acquired through laboratory models and numerical models based on other types of coastal structures are also proposed and hence can be further utilized to assess the bridge deck-wave interaction problems (Kaplan 1992; Kaplan et al. 1995; Bea et al. 1999; McConnell et al. 2004; Cuomo et al. 2007). As such, it is necessary to examine the
developed empirical methods in the cases of solitary wave induced loadings on the bridge decks in order to provide guidelines for practical engineering projects.

Table 2.3 Results of different railing height by current method

<table>
<thead>
<tr>
<th>Case</th>
<th>Railing height</th>
<th>Vertical force</th>
<th>Ratio</th>
<th>Horizontal force</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>unit: kN</td>
<td></td>
<td>unit: kN</td>
<td></td>
</tr>
<tr>
<td>Case 1</td>
<td>0</td>
<td>97.143</td>
<td>1</td>
<td>18.67</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>0.3 m</td>
<td>100.4</td>
<td>1.034</td>
<td>20.274</td>
<td>1.086</td>
</tr>
<tr>
<td></td>
<td>0.6 m</td>
<td>109.638</td>
<td>1.129</td>
<td>22.908</td>
<td>1.227</td>
</tr>
<tr>
<td>Case 2</td>
<td>0</td>
<td>145.78</td>
<td>1</td>
<td>20.52</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>0.3 m</td>
<td>152.268</td>
<td>1.045</td>
<td>22.663</td>
<td>1.104</td>
</tr>
<tr>
<td></td>
<td>0.6 m</td>
<td>156.188</td>
<td>1.071</td>
<td>25.725</td>
<td>1.254</td>
</tr>
<tr>
<td>Case 3</td>
<td>0</td>
<td>158.003</td>
<td>1</td>
<td>19.93</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>0.3 m</td>
<td>160.885</td>
<td>1.018</td>
<td>23.195</td>
<td>1.164</td>
</tr>
<tr>
<td></td>
<td>0.6 m</td>
<td>167.151</td>
<td>1.058</td>
<td>27.134</td>
<td>1.361</td>
</tr>
</tbody>
</table>

In this section, comparisons are made between the current numerical results and those calculated from some as-established formulae (McConnell et al. 2004; Douglass et al. 2006; Cuomo et al. 2007; McPherson 2008; Boon-intra 2010) in order to verify the applicability and capability of these formulae in predicting the solitary wave forces on a typical coastal bridge deck with girders. Some other methods, such as Coastal Engineering Manual (U.S. Army Corps of Engineers 2002) and ASCE/SEI7-05 (2006), are also found to predict wave forces on coastal structures; however, they are not utilized here in the following comparison process with the following reasons: (a) for Coastal Engineering Manual (U.S. Army Corps of Engineers 2002), it is indicated that physical model tests are needed to recalibrate the corresponding coefficients adopted in the prediction equations; (b) ASCE/SEI7-05 (2006) deals with wave forces on wall kind of coastal structures and they may be not suitable to be employed here in assessing the wave forces on bridge decks that have relatively narrow horizontally projected areas.


Based on a series of experimental studies on jetties (Tirindelli et al. 2002; McConnell et al. 2003; Allsop and Cuomo 2004), McConnell et al. (2004) provided the following empirical equations to predict wave forces on structure elements for jetty structures.

\[
\frac{F_{\text{vqs}}(+\text{or}-)}{F_v^*} = \frac{a}{\left[\frac{\eta_{\max} - S}{H}\right]^b} \tag{2.5a}
\]

\[
\frac{F_{\text{hqs}}(+\text{or}-)}{F_h^*} = \frac{a}{\left[\frac{\eta_{\max} - S}{H}\right]^b} \tag{2.5b}
\]

where \(F_{\text{vqs}}\) and \(F_{\text{hqs}}\) are quasi-static forces to be determined; \(F_v^*\) and \(F_h^*\) are the basic vertical and horizontal forces, respectively; \(S\) is the clearance between the bottom of the structure to SWL; and \(a\) and \(b\) are empirical coefficients. In their experimental studies of the jetty structures, the considered ratio of the wave height to the clearance, \(H/S\), was from 1.1 to 18.2 and the ratio of the wave height to the water depth, \(H/d\), was from 0.13
to 0.33. Since no submerged conditions were considered in developing this method, the equations may not be suitable to the cases when the bridge decks are below the SWL.

### 2.4.2 Douglass et al.’s (2006) Interim Approach

Based on the previous observations that the wave loads are linearly proportional to the difference between the wave crest and the elevation of the bottom of the structure (Wang 1970; French 1979; Overbeek and Klabbers 2001), Douglass et al. (2006) developed an interim approach to predict the wave forces on typical coastal bridges using the following equations:

\[ F_v = c_{v-va} \gamma (\Delta z_v) A_v \]  
\[ F_h = [1 + c_r(N - 1)] c_{h-va} \gamma (\Delta z_h) A_h \]

where \( F_v \) = vertical wave load component; \( F_h \) = horizontal wave load component; \( c_{v-va} \) and \( c_{h-va} \) = empirical coefficients for vertical and horizontal “varying” loads, respectively; \( c_r \) = reduction coefficient for reduced horizontal load on the internal girders; \( N \) = number of girders supporting the bridge deck; \( A_v \) = area of the horizontal projection of the bridge deck; \( A_h \) = area of the vertical projection of the deck span; \( \Delta z_v \) = difference between the elevation of the crest of the maximum wave and the elevation of the bottom of the bridge deck; \( \Delta z_h \) = difference between the elevation of the crest of maximum wave and the elevation of the centroid of \( A_h \); and \( \gamma \) = unit weight of saltwater. The definition sketch for these parameters is shown in Fig. 2.11. In this figure, \( \eta_{max} \) is the maximum wave crest elevation. In the calculations employing the equations by Douglass et al.’s (2006), the parameters are defined as follows: \( A_v = 10.45 \text{ m}^2 \), \( c_{v-va} = 1 \), \( A_h = 1.35 \text{ m}^2 \), \( c_{h-va} = 1 \), \( \gamma = 9.792 \text{ kN/m}^3 \), \( N = 6 \), and \( c_r = 0.4 \). The height of the girder is 1.05m and the thickness of the bridge deck is 0.3m.

![Fig. 2.11 Definition sketch for the interim approach proposed by Douglass et al. (2006)](image)

### 2.4.3 Cuomo et al.’s (2007) Empirical Method

Similar to the guidance for evaluating wave forces on exposed jetties by McConnell et al. (2003, 2004) and Tirindelli et al. (2002, 2003a, 2003b), Cuomo et al. (2007) provides a prediction method based on separated structural elements using the filtered experimental data (Tirindelli et al. 2003a, 2003b) by wavelet analysis (Cuomo et al. 2003) to account for the dynamic effects in the experimental setup. Both the horizontal
and vertical wave forces are plotted against \((\eta_{\text{max}} - S)/d\) and non-dimensionalized by \(\gamma HA\), where \(A\) is the area of the element, normal to the wave forces applied. The generalized prediction equation is given as:
\[
\frac{F_v \text{ or } F_h}{\gamma HA} = a \left( \frac{\eta_{\text{max}} - S}{d} \right) + b
\]
where the coefficients \(a\) and \(b\) are provided by empirical fitting.

### 2.4.4 McPherson’s (2007) Empirical Method

Taking the Douglass et al.’s (2006) interim approach as a starting point, McPherson (2008) developed a method to examine the wave forces on bridge decks. The equations for predicting the horizontal and vertical wave forces by McPherson (2008) are given as follows:

- \(F_v = F_{\text{hydrostatic}} + F_{\text{Bridge}} + F_{\text{AirEntrapment}}\)
  \[F_v = \gamma \delta_Z A_v - F_w + \gamma V o l_{\text{Bridge}} + (N - 1)0.5\gamma \delta_G A_G\] (2.8a)
- \(F_h = F_{\text{Hydrostatic\_Front}} - F_{\text{Hydrostatic\_Back}}\)
  \[F_h = 0.5\gamma \delta A_v + \gamma (h - h_{\text{model}}) A_v\] (2.8b)

If \(h \leq h_{\text{model}}\),

- \(F_w = 0.5\gamma \delta A_v\) (2.8c)

and if \(h > h_{\text{model}}\),

- \(F_w = 0.5\gamma \delta A_v + \gamma (h - h_{\text{model}}) A_v\) (2.8d)

If \(\eta_{\text{max}} < h_{\text{deck}}\),

- \(F_{\text{Hydrostatic\_Front}} = 0.5(\eta_{\text{max}} + h - h_{\text{girder}})H_{\text{bridge}} L_{\text{bridge}} \gamma\) (2.8e)

and if \(\eta_{\text{max}} > h_{\text{deck}}\),

- \(F_{\text{Hydrostatic\_Front}} = 0.5[(\eta_{\text{max}} + h - h_{\text{girder}}) + (\eta_{\text{max}} - h_{\text{deck}})]H_{\text{bridge}} L_{\text{bridge}} \gamma\) (2.8f)

If \(h < h_{\text{girder}}\),

- \(F_{\text{Hydrostatic\_Back}} = 0\) (2.8g)

and if \(h > h_{\text{girder}}\),

- \(F_{\text{Hydrostatic\_Back}} = 0.5(h - h_{\text{girder}})^2 L_{\text{bridge}} \gamma\) (2.8h)

where \(\delta_Z\) is distance from the top of the deck to the wave crest, \(\eta_{\text{max}}\); \(\delta_G\) is the height of the bridge girders; \(A_G\) is the cross sectional area of trapped air between girders; \(\delta\) is the height of wave overtopping the bridge deck; \(h\) is the height from the ground elevation to the SWL; \(h_{\text{model}}\) is the distance from the ground elevation to the top of the deck; \(h_{\text{girder}}\) is the height from the ground elevation to the bottom of the bridge girders; \(h_{\text{deck}}\) is the height from the ground elevation to the bottom of the deck; \(H_{\text{bridge}}\) is the height of the bridge impacted by lateral wave forces; \(L_{\text{bridge}}\) is the length of the bridge impacted by lateral wave forces; \(A_v, \gamma, N\), and \(\eta_{\text{max}}\) are the same as those adopted in Douglass et al. (2006).
2.4.5 Boon-intra’s (2007) Method

Based on the tsunami time-history loads calculated from finite-element models and the studies by Douglass et al. (2006), Yeh (2007), and FEMA P646 (2008), Boon-intra (2010) proposed a method to estimate tsunami impact forces on bridge superstructures by combining the hydrostatic and hydrodynamic water pressure on deck-girder bridge sections. The proposed method was developed to be used as a preliminary guideline for design purpose due to the lack of laboratory experiments on physical bridge models. The equations are described as follows:

\[ F_h = F_{\text{hydrostatic}} + F_{\text{hydrodynamic}} \]
\[ = [1 + c_r(N - 1)]\gamma(\Delta z)A_h + 0.5 \cdot C_d \rho (\Delta h \cdot u^2)_{\text{max}} \]  
\[ (2.9a) \]
\[ F_v = F_{\text{buoyant}} + F_{\text{uplift}} = [\gamma \cdot (\Delta z) + 0.5 \cdot \rho u^2_{x,\text{max}}]A_v \]  
\[ (2.9b) \]

where \( F_h \) is the horizontal force, consists of two parts, hydrostatic horizontal force and hydrodynamic horizontal force; \( F_v \) is the vertical force, consists of two parts, buoyant force (hydrostatic vertical force) and uplift force (hydrodynamic vertical force); \( \Delta z \) is the distance from the bottom of girders to the instantaneous water-surface elevation (to \( \eta_{\text{max}} \) as used in the current study); \( (\Delta h \cdot u^2)_{\text{max}} \) is the maximum flux momentum; \( u_{x,\text{max}} \) is the adjusted horizontal wave velocity \( (u_{x,\text{max}} = 3.5u^*_x,\text{max} \) when the bridge deck is subjected to less inundation and \( u_{x,\text{max}} = u^*_x,\text{max} \) when the bridge deck is facing large inundation); \( u^*_x,\text{max} \) is horizontal wave velocity; \( C_d \) is the empirical drag coefficient \( (C_d = 1.0 \) when the bridge deck is subjected to less inundation and \( C_d = 2.0 \) when the bridge deck is facing large inundation); \( A_v, A_h, \gamma, N, \) and \( c_r \) are the same as those adopted in Douglass et al. (2006). In the current study, \( u \) and \( u^*_x,\text{max} \) are considered as the horizontal velocities of the water particles at the SWL in order to accommodate to the solitary wave conditions (rather than breaker bores).

2.4.6 Comparisons of the above Reviewed Methods

Generally, the total wave forces are divided into several components, i.e. hydrostatic force (e.g., water on deck force and buoyancy force), velocity related force (e.g., drag force and slamming force), and acceleration related force (e.g., inertia force). For the above discussed studies (McConnell et al. 2004; Douglass et al. 2006; Cuomo et al. 2007; McPherson 2008; Boon-intra 2010), they share some commonalities, i.e., no inertia forces are considered and the parameters of the wave period and wave lengths are not expressed. Since it is recognized that the wave forces are closely related to the wave period (El Ghamry 1963; McPherson 2008) and the inertia forces may play an important role in the total forces (Bea et al. 1999; AASHTO 2008; Sheppard and Marin 2009), the drawbacks or shortcomings for employing above methods to make predictions of solitary wave forces on bridge decks can be expected. However, these existing methods may be further utilized to predict reasonable wave loads on the bridge superstructures for some cases. In fact, the verification of the applicability of these methods is significantly important.

The numerical results of the positive peak horizontal forces and vertical forces are compared with those through the studies by McConnell et al. (2004), Douglass et al. (2006), Cuomo et al. (2007), McPherson (2008), and Boon-intra (2010), and are
demonstrated for two wave heights in Fig. 2.12 and Fig. 2.13, respectively. For the horizontal forces as shown in Fig. 2.12, the predicted wave forces by Douglass et al. (2006), Cuomo et al. (2007), and Boon-intra 2010 are significantly conservative at most times. Apparently, Douglass et al.’s (2006) interim approach has an intrinsic shortcoming that higher wave forces can be estimated when the bridge superstructure is much submerged due to the increased water level. The reason is that this interim approach is not proposed for much submerged cases but rather for the conditions when the bridge superstructure is well around or suspended above the SWL. In addition, this interim equation does not distinguish the difference of wave types, e.g. Stokes waves, cnoidal wave and solitary wave. Different wave types have different horizontal and vertical velocity components, which can be reflected in the numerical simulations, but not in the empirical formulas. Moreover, the linear increase of horizontal wave forces with the increase of the submersion depth contradicts with the numerical observations reported in the literature (Jin and Meng 2011; Xiao et al. 2010; Huang and Xiao 2009). Boon-intra’s (2010) method adds a hydrodynamic force component to the overall force based on the study by Douglass et al. (2006). As such, the predicted horizontal forces follow the same general trend as observed for the results based on Douglass et al.’s (2006) interim approach. The predicted forces by McConnell et al. (2004) also show remarkable differences as compared with the current numerical results.

Generally, McPherson’s (2008) method predicts much closer horizontal wave forces to those by the current numerical method as compared with other methods; however, the predicted forces are slightly larger than those by the current numerical method when the submersion coefficient is negative and smaller when positive. It is assumed that there is no water (pressure) on the trailing end (backside) of the studied structures for other four studies (McConnell et al. 2004; Douglass et al. 2006; Cuomo et al. 2007; Boon-intra 2010), and this makes the prediction process unstable and errors can occur, especially for fully submerged conditions. Apparently, McPherson’s (2008) method demonstrates its improvement based on the study by Douglass et al. (2006) to better assess the solitary wave forces on the bridge decks since the hydrostatic force on the backside is taken into account.
For the comparisons of the vertical forces between the current numerical method and other five methods as shown in Fig. 2.13, the predicted forces by McConnell et al. (2004) and Cuomo et al. (2007) almost follow the same pattern that smaller forces (compared to the current numerical results) are estimated when the submersion coefficient is larger than -1.0 and larger forces when the submersion coefficient is much negative (smaller than -1.0). Douglass et al.’s (2006) method predicts smaller vertical forces when the submersion coefficient is positive and more conservative vertical forces when the bridge superstructure is beyond fully submerged.

Similarly, Boon-intra (2010)’s method inherits the characters of Douglass et al.’s (2006) interim approach that larger vertical forces are predicted with greater levels of submergence for the bridge superstructures, but with more conservative predicted results since the hydrodynamic force component is considered in the method. It is noticed that the predicted uplift force, \( F_{\text{uplift}} = 0.5 \cdot \rho u_{x,\text{max}}^2 A \), is very conservative when \( u_{x,\text{max}} = 3.5u_{x,\text{max}}^* \) as compared with the equation proposed by Bea et al. (1999) that \( F_i = 0.5 \cdot \rho C_l A u^2 \), where \( C_l \) is the lifting coefficient, \( A \) is the vertical deck area subjected to wave impinging, and \( u \) is the horizontal fluid velocity of the wave crest. This predicted uplift force tends to be larger with higher wave heights and may occupy a larger portion in the total vertical forces.

McPherson’s (2008) method predicts relatively close results of the vertical forces with those by the current numerical method when the submersion coefficient is positive and around -1.0. It is noticed that remarkable difference between the predicted values and current numerical results is found when the submersion coefficient is -0.5 (i.e., the bridge superstructure is half submerged) and the predicted forces by McPherson’s (2008) method are relatively smaller. This is mainly due to the inappropriate treatment of \( \delta_Z \) in the force component of \( F_{\text{hydrosstatic}} = \gamma \delta_Z A v - F_w \) for this submergence condition, where \( \delta_Z \) is distance from the top of the deck to the wave crest. The actual hydrostatic force may be underestimated by taking this way. It may be more reasonable to take account in the effects of the compressed pressure due to the entrapped air on the above deck elements. It is observed that the air pressure in each air chamber between the girders
underneath the deck is uniformly distributed (Hayatdavoodi et al. 2014). Hence, the definition of \( \delta_Z \) needs to be adjusted and this will be discussed later.

To conclude, the applicability of the methods by McConnell et al. (2004) and Cuomo et al. (2007) to predict solitary wave forces on the typical coastal bridge deck with girders is questionable. Discrepancies are found in the discussed comparisons, indicating that the prediction equations originally developed for the jetty structures cannot be directly adopted to estimate solitary wave forces on bridge decks. Several distinct factors are analyzed in contributing to the differences in the above comparisons, such as the entrapped air effects, no submerged conditions considered, and different wave types studied. These factors result in critical differences in both the phenomena and mechanisms of the wave-structure-interaction between the experimental studies for the jetty structures and the current study for a bridge deck. As such, the coefficients, i.e., \( a \) and \( b \), provided by McConnell et al. (2004) and Cuomo et al. (2007) need to be recalibrated and more studies are needed in this direction. Douglass et al.’s (2006) interim approach predicts more conservative wave forces at most times and it is more acceptable to assess the vertical forces when the bridge superstructure is near the SWL. Boon-intra’s (2010) method is too much conservative since additional hydrodynamic force component is considered based on Douglass et al.’s (2006) interim approach. McPherson’s (2008) method performs much better since the water on deck force (weight of the overtopping water) and the hydrostatic force due to the existing water at the backside of the bridge superstructure are considered.

### 2.5 Suggested Method

It is confirmed that the hydrostatic force component is dominant in the total forces for partially and fully submerged bridge decks (Douglas et al. 2006; McPherson 2008; Bozorgnia et al. 2010). Based on the above comparisons and discussions, the methods by McConnell et al. (2004), Douglass et al.’s (2006), Cuomo et al. (2007), and McPherson (2008) are provided based on the analysis at the hydrostatic force level and thus they are expressed only including the hydrostatic force components. However, it is recognized that the velocity related force (named hydrodynamic force) should be considered by taking into account the effects of wave periods, wave types, and water particle velocities near the structure, though may not be straightforward, in order to acquire more realistic results. This is one reason that McPherson’s (2008) method underestimates the horizontal forces at cases when the bridge superstructure is above the SWL (Fig. 2.12) and the vertical forces at cases when the bridge superstructure is around the SWL (Fig. 2.13). Thus, an improved prediction method is suggested to include the hydrodynamic force by modifying the McPherson’s (2008) method, and the equations are expressed as follows:

\[
F_v = F_{\text{Hydrostatic}} + F_{\text{Bridge}} + F_{\text{AirEntrainment}} + F_t
= \gamma \delta_Z A_v - F_w + \gamma V \omega_{\text{Bridge}} + (N - 1)0.5y \delta G A_G + 0.5 \cdot \rho C_l A_y u^2
\]  
(2.10a)

\[
F_h = F_{\text{Hydrostatic, Front}} - F_{\text{Hydrostatic, Back}} + F_D
\]  
(2.10b)

If the SWL is below the top of the deck and no overtopping water exists, i.e., \( h + \eta_{\text{max}} \leq h_{\text{model}} \),
\[
F_w = 0
\]  
(2.10c)

if the SWL is below the top of the deck but overtopping water exists, i.e., \( h \leq h_{\text{model}} < h + \eta_{\text{max}} \).
\( F_v = C_w \gamma \delta A_v \) 
and if the SWL is above the top of the deck, i.e., \( h > h_{\text{model}} \),
\( F_v = C_w \gamma \delta A_v + \gamma (h - h_{\text{model}}) A_v \)  \hspace{1cm} (2.10d)
If the front girder is partially submerged, i.e., \( h_{\text{girder}} < h + \eta_{\text{max}} < h_{\text{model}} \),
\( F_{\text{Hydrostatic, Front}} = 0.5(\eta_{\text{max}} + h - h_{\text{girder}}) H_{\text{bridge}} L_{\text{bridge}} \gamma \)  \hspace{1cm} (2.10f)
and if the front girder is fully submerged, i.e., \( h + \eta_{\text{max}} > h_{\text{model}} \),
\( F_{\text{Hydrostatic, Front}} = 0.5(\eta_{\text{max}} + h - h_{\text{girder}} + (\eta_{\text{max}} + h - h_{\text{model}}) H_{\text{bridge}} L_{\text{bridge}} \gamma \)  \hspace{1cm} (2.10g)
If the back girder is above the water, i.e., \( h + h_{\text{Back}} < h_{\text{girder}} \),
\( F_{\text{Hydrostatic, Back}} = 0 \)  \hspace{1cm} (2.10h)
if the back girder is partially submerged, i.e., \( h_{\text{girder}} < h + h_{\text{Back}} < h_{\text{girder}} + H_b \),
\( F_{\text{Hydrostatic, Back}} = 0.5(h + h_{\text{Back}} - h_{\text{girder}})^2 L_{\text{bridge}} \gamma \)  \hspace{1cm} (2.10i)
and if the back girder is fully submerged, i.e., \( h + h_{\text{Back}} > h_{\text{girder}} + H_b \),
\( F_{\text{Hydrostatic, Back}} = 0.5(2h + 2h_{\text{Back}} - h_{\text{girder}} - h_{\text{model}}) H_{\text{bridge}} L_{\text{bridge}} \gamma \)  \hspace{1cm} (2.10j)
\( F_D = 0.5 \cdot \rho C_D A_h u^2 \)  \hspace{1cm} (2.10k)
where \( F_i \) is the uplift force; \( F_D \) is the drag force; \( C_D \) and \( C_l \) are the drag and lift coefficients, respectively, and they are tentatively defined as 1.0 in the current study; \( C_w \) is closely related to the weight of the overtopping water and the effective wave length, defined as 0.6 when \( h \leq h_{\text{model}} \) and 0.7 when \( h > h_{\text{model}} \) in the current study; \( u \) is the horizontal velocity of the water particle at the SWL taken at the section of the wave crest; \( \delta_z \) is the distance from the bottom of the deck to the wave crest; \( h_{\text{Back}} \) is the possible water height above the SWL at the trailing edge of the bridge deck and it is related to the effective wave length, the wave height, the water depth, and the comparable width of the bridge superstructure. The value of \( h_{\text{Back}} \) ranges from 0.4 m (for smaller wave heights) to 0.8 m (for higher wave heights) tentatively based on the observations in the current study; all other parameters remain the same as those defined in the study by McPherson (2008).

Further comparisons are made by incorporating the results from the suggested method, as shown in Fig. 2.14. The results through the methods by Douglass et al. (2006) and Boon-intra (2010) are excluded. It shows that the suggested method makes relatively better predictions than McPherson’s (2008) method. It also shows that the suggested method predicts relatively conservative results at most times. The comparisons of the wave forces between the numerical results and those predicted by the current suggested method for all the cases studied in the current study are plotted in Fig. 2.15. As a result, we can conclude that this suggested method can be taken as an alternative way to make reasonable predictions of the solitary wave forces on deck-girder bridge superstructures.

2.6 Technical Justification for Suggested Method

In this part, several key parameters, such as the value of \( u, C_w \), and \( h_{\text{Back}} \), in the suggested method are justified. Based on the observations from Fig. 2.7 that the vertical force and horizontal force reach their positive peak values at different time for different cases but around 10s, Fig. 2.16 are plotted as the “freezing frames” with the wave height 1.74 m at the simulation time \( t = 10 \) s in order to make the technical justification for the suggested method.
Fig. 2.14 Comparisons with the results from the suggested method

(a) Horizontal force, $H = 1.74$ m
(b) Horizontal force, $H = 2.20$ m
(c) Vertical force, $H = 1.74$ m
(d) Vertical force, $H = 2.20$ m

Fig. 2.15 Comparisons of the positive peak wave forces between the numerical results and the predicted results

(a) Horizontal force
(b) Vertical force
Fig. 16 Analysis of water particle velocities for different bridge elevations with the wave height 1.74 m at the simulation time $t = 10.0$ s. Note: the bridge elevation in (b), (c), and (d) refers to the elevation at the middle of the bridge superstructure.
In Fig. 2.16, the wave surface profiles are drawn based on the analytical solutions for demonstration purpose only and practically, they are different from the realistic wave profiles that should be disturbed in the bridge deck-wave interaction. Three cases of different bridge elevations (Case 1, Case 3, and Case 6) are presented here as typical scenarios with three fixed sections during the solitary wave propagation (Section I, Section II, and Section III) plotted in each case. Section II is where the wave crest is located. Section I and III are considered as 5m behind and ahead of the location of the wave crest, respectively, and Section III is almost the location where the positive peak vertical force occurs when this section reaches to the front of the bridge deck. The water particle velocities at each section and the bridge elevations for each case are also presented in Fig. 2.16 (b), (c), and (d), respectively. For solitary waves, the horizontal velocities of the water particles are always positive, while the vertical velocities become negative when the wave crest passes that point, such as the water particles at Section I.

It is interpreted that both the horizontal and vertical velocities of the water particles contribute to the positive peak vertical force when section III reaches to the front of the bridge deck, as shown in Fig. 2.16 (d). Afterwards, the bottom of the bridge superstructure will have less area that is subjected to the water particles with the upward (positive) vertical velocity and more area for those with downward (negative) vertical velocity, and the vertical velocities of the water particles on the vertical projected area of the bridge deck becomes smaller gradually along with the wave propagation till to be negative (downward); this makes the vertical force fall off from the peak value (i.e., when Section I reaches the front of the bridge deck). Apparently, the horizontal velocities of the water particles at Section II for the three cases are larger than those at Section III and Section I for the corresponding cases. This assures that the positive peak horizontal force probably takes place when Section II reaches the front of the bridge deck.

For the suggested method, \( u \) is represented by the horizontal velocity of the water particle at the SWL for Section II. Thus, this may not reflect the actual situation very well in the term of \( F_l = 0.5 \cdot \rho C_l A_v u^2 \) that both the horizontal and vertical velocities of the water particles make the joint efforts to the occurrence of the positive peak vertical force. In addition, for the positive peak horizontal forces \( F_D = 0.5 \cdot \rho C_D A_h u^2 \), this leads to slightly conservative estimations for more submerged cases, such as Case 6 as shown in these sections, where the horizontal particle velocities at the elevation of Case 6 is relative smaller than those for Case 3 (i.e., \( u \) is acquired at this position).

Fig. 2.17 shows the schematic of the estimation of the overtopping water in order to predict the value of \( F_w \). In the prediction of the fraction of the water that is above the SWL in the overall overtopping water \( (h > h_{model}) \), \( C_w \) is an empirical coefficient and it is closely related to the effective wave length for solitary waves. Practically, \( C_w \) can be defined as a value as 0.7 or even larger for more submerged cases when the effective length is much longer than the width of the bridge deck, and this can be demonstrated in an example as shown in Fig. 2.18. Although it is noticed in this example that the analytical (without the bridge model in the computational domain) water surface profile is slightly disturbed with the presence of the bridge model, it can be treated that the fraction of the water above the SWL in the overall overtopping water under these two conditions are the same for simplicity. However, \( C_w \) can be a smaller value when the
effective wave length is comparable with the width of the bridge deck. It is the same criteria for choosing appropriate values for $C_w$ when $h \leq h_{model}$.

![Schematic of the estimated overtopping water](image1)

**Fig. 2.17** Schematic of the estimated overtopping water

(a) Snapshot at $t = 9s$ for Case 8 with the wave height 3.00m

![Snapshot](image2)

(b) Comparisons of the wave surface profiles corresponding to the snapshot in (a)

**Fig. 2.18** Demonstration of an example to estimate the overtopping water when the positive peak horizontal force occurs

It needs to be noticed that the hydrostatic force at the trailing edge of the bridge deck, named $F_{Hydrostatic\_Back}$, is considered as 0 when $h + h_{Back} < h_{girder}$. This matches the infield situation quite well when the bridge elevation is much above the SWL. However, an appropriate value of $h_{Back}$ needs to be determined when the bridge elevation is partially submerged or fully submerged. The value of $h_{Back}$ is related to the effective wave length, the wave height, the water depth, the wave speed, and the geometry of the bridge superstructure. It is observed that higher wave heights are
accompanied with more intense bridge deck-wave interaction, as shown in Fig. 2.19, where snapshots around the occurrence of the positive peak horizontal forces for Case 3 with different wave heights are captured. In this regard, the value of $h_{Back}$ for the corresponding wave height can be empirically determined accordingly. However, this value may subject to error due to that the solitary waves may undergo significant scattering or diffraction in the bridge deck-wave interaction, especially when the bridge elevation is located around the SWL.

![Fig. 2.19 Snapshots for empirically determining the value of $h_{Back}$ for Case 3 with different wave heights](image)

Another concern is that the value of $\delta_Z$ in the force component of $F_{hydrostatic} = \gamma \delta_Z A_{v} - F_w$ is the distance from the bottom of the deck to the wave crest as defined in the suggested method. Actually, it may not be appropriate to take the same pressure ($\gamma \delta_Z$) on the whole projected area ($A_{v}$), as can be observed in Fig. 2.20. In this figure, the gauge pressure (with respect to the operating pressure, 101325 Pa) for cases with the wave height 1.74 m at the simulation time $t = 10s$ is plotted. For each of the three cases considered here (Case 1, Case 3, and Case 6), the pressure varies at different chambers (partitioned by the deck elements and the girder elements). Hence, the hydrostatic force may be overestimated for some cases.

Currently, since research on the contribution of the inertial force to the total force is at the early stage for bridge deck-wave interaction problems, especially on the topic
regarding solitary wave induced loadings on bridge decks (McPherson 2008; AASHTO 2008; Jin and Meng 2011), it is seldom included in these reviewed prediction equations. Though Kaplan (1992), Kaplan et al. (1995), and Bea et al. (1999) considered the inertial force for the horizontal cylinders and offshore platforms, these formulae may be not appropriately for bridge decks due to that the cross section geometries of bridge decks are significantly different from the offshore platforms. However, it is feasible to more appropriately address the inertia force in the prediction equations for bridge deck-wave interactions under solitary wave conditions based on some previous studies (AASHTO 2008; Gullett et al. 2012). This is left for future studies.

![Fig. 2.20 Snapshots for the gauge pressure (with respect to the operating pressure) for cases with the wave height 1.74 m at the simulation time \( t = 10 \) s. (a) Case 1; (b) Case 3; (c) Case 6.](image)

2.7 Conclusions and Remarks

The review of previous studies demonstrates the gap that more convenient and practical guidelines for solitary wave forces on coastal bridge decks are urgently needed based on the as-obtained observations. This need for the potential guidelines originally motivated this research. In the present study, solitary wave forces on a typical coastal bridge deck with girders are numerically investigated using the SST \( k-\omega \) turbulent flow. The obtained simulation results of the wave forces on the typical coastal bridge deck with girders are compared with those by previous empirical formulae and then an improved method is suggested to include the hydrodynamic force on the basis of the examined
methods. Based on the parameters set in the current study: the water depth 7.22 m, the range of the ratio of the wave height to the water depth from 0.12 to 0.42, and the submersion coefficient from -2 to 0.5, the following conclusions can be drawn:

(1) The maximum positive peak vertical forces per unit length are closely related to the wave heights and submersion depths. For higher wave heights, when the bottom of the superstructure is around the SWL, the positive peak vertical forces tend to be larger than those at other elevations. However, the maximum positive horizontal force occurs when the superstructure is fully submerged. It is observed that the positive peak vertical force decreases rapidly with an increase of deck clearance above the SWL. As a result, increasing the bridge deck clearance above the SWL could be a good countermeasure when designing and retrofitting coastal bridges vulnerable to solitary waves, though resulting in higher cost.

(2) For the cases considered (Cases 1, 2, and 3 with the wave height 2.20 m), increasing the railing height results in an increase of the horizontal force and the vertical force. It is concluded that the railing has larger effects on the horizontal force than the vertical force.

(3) It is not practicable to employ the methods by McConnell et al. (2004) and Cuomo et al. (2007) to predict solitary wave forces on typical coastal bridge decks. Discrepancies are found in the studied comparisons, indicating that the prediction equations originally developed for the jetty structures cannot be directly adopted to estimate solitary wave forces on bridge decks. Douglass et al.’s (2006) interim approach predicts more conservative wave forces at most times. Boon-in-tra’s (2010) method is too much conservative since additional hydrodynamic force component is considered based on the Douglass et al.’s (2006) interim approach. McPherson’s (2008) method performs much better since the water on deck force and the hydrostatic force at the backside of the bridge superstructure are considered.

(4) A suggested method for calculating solitary wave forces based on reviewed studies is proven to be a practical and simple way to predict the wave forces induced by solitary waves on the typical coastal bridge deck with girders. Generally, the suggested method predicts slightly conservative but reasonable results.

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

(1) In the present study, 2D numerical simulations have been conducted. However, 3D models may provide more reliable results, but maybe much more computational cost.

(2) In this study, large ratios of the wave height to the water depth (i.e., 0.36 and 0.42) are considered since good wave profiles of the numerical results as compared with analytical ones are obtained at the location where the bridge model is placed. However, additional care should be taken for the results since the analytical solution is based on relatively small ratios.

(3) Larger wave heights that are close to the breaking wave height need to be further studied.

(4) Further studies are needed to appropriately address the inertia force and the entrapped air effects in the prediction equations for bridge deck-wave interactions under solitary wave conditions.
(5) The suggested method can be further adapted to make convenient assessment of the wave forces induced by periodical waves. As such, several coefficients need to be adjusted accordingly, such as $C_w$ and $A_v$ in the equations to predict $F_w$ and the hydrostatic force ($F_{hydrostatic\_back}$) at the backside of the bridge deck. Moreover, additional research is necessary to advance the present understanding of the solitary wave forces on coastal bridge decks with different number of girders and on slab only bridge decks.

2.8 References


3.1 Introduction

Recent tsunamis and hurricanes have caused devastating impact on many coastal communities and significant damages to the coastal infrastructures, including many low-lying coastal bridges, demonstrating an urgent need to investigate the failure mechanisms of these coastal bridges. During these natural events, the coastal bridges play a significant role in the evacuation and in the recovery process. However, from the post disaster survey, many low-lying coastal bridges were damaged in these events (overturned or displaced), causing detrimental influence on many aspects, including the transportation of the communities in disaster areas and providing supplies for the affected areas (Graumann et al. 2005; Shoji and Moriyama 2007; Yeh et al. 2007; Ghobarah et al. 2006; FHWA 2008; Bricker et al. 2012).

The importance of the bridge deck-wave interaction problems under solitary waves attracts a widespread concern and many efforts have been made on this topic, especially in the last decade (Araki et al. 2008; Lao et al. 2010; Shoji et al. 2011; Hayatdavoodi et al. 2014; Bozorgnia et al. 2010; Bricker et al. 2012; Bricker and Nakayama 2014; Xu and Cai 2014). For the physical model studies, Araki et al. (2008) investigated the characteristics of the fluid actions on a six-girder PC bridge in a 2D wave tank. The horizontal and lift forces were measured and analyzed and they were found with the same order in magnitude. Lao et al. (2010) estimated the horizontal forces on two kinds of bridge deck configurations, with solid and perforated parapets. It was found that the bridge deck with perforation in parapets has less damage as compared with the one with solid parapets due to that perforated parapets has a smaller water impact area. Shoji et al. (2011) conducted hydraulic experiments on a prototype bridge, the Lueng Ie Bridge, with 1:79.2 and 1/100 geometrical scales under plunging breaker bores and surging breaker bores of the solitary wave. In this study, only horizontal wave force was analyzed with the experimental data, and the drag coefficients under these two conditions were investigated. Most recently, Hayatdavoodi et al. (2014) conducted solitary wave forces on several bridge superstructure configurations with a 1:35 scale experimentally and five ratios of the wave height to the still water depth were considered. In addition, comparisons with the results by CFD simulations were made and the entrapped air effects were also investigated. As for numerical approaches, Bozorgnia et al. (2010) studied the wave forces on a bridge superstructure with the geometry similar to the I-10 Bridge across Mobil Bay by employing solitary wave model with various wave heights for one fixed bridge elevation. The air venting slots were also studied with much reduction of the vertical forces, while the horizontal forces were witnessed without significant change in magnitude. Bricker et al. (2012) and Bricker and Nakayama (2014) investigated the wave force characteristics of the Utatsu concrete girder highway bridge damaged in the 2011 Great East Japan Tsunami. The effects of the entrapped air, the deck inclinations, and the nearby structures were considered. Xu and Cai (2014) mainly focused on the solitary wave forces on the typical coastal bridge decks with inclinations. Based on the extensive study under the prescribed conditions, the characterizations of the force development along with variable inclination angles of the bridge superstructure were presented.
The wave forces collected in the physical experiments are mainly recorded through the force or pressure transducers that are installed in the model setups and hence the presented data in previous studies are always the overall forces, including horizontal and vertical forces. Similarly, the wave forces given in the previous numerical studies are also evaluated on the whole bridge superstructures. As such, it is necessary to investigate the wave forces on each component of the bridge deck in order to deepen our understanding of the wave forces in each locality. In addition, there are rare studies focusing on the overturning moments on the bridge decks induced by solitary waves, if any (Xu and Cai 2014). It is generally considered that the overall horizontal and vertical wave forces are applied on the centroid of the bridge superstructure (Douglass et al. 2006). AASHTO (2008) code addresses this problem in a relatively indistinct way that two different locations are proposed where the wave forces should be applied in the bridge design. While one location is at the trailing edge of the bridge decks (at the bottom of the most landward girder), the other one is that the prorated horizontal and vertical forces are applied to the corresponding exposed surfaces of the overhangs. Therefore, the understanding of the overturning moment on the bridge decks needs to be further clarified and the advantage of the component level based analysis for such a purpose is prominent.

Regarding the air venting holes for releasing the entrapped air in the air chamber between the girders underneath the bridge deck, it is generally considered this countermeasure is a very practical and efficient way for mitigating the wave on deck forces (Cuomo et al. 2009; Bozorgnia et al. 2010; Hayatdavoodi et al. 2014). However, the air venting holes (or slots) in these studies are with fixed areas. Hence, the efficiency of venting holes with different sizes needs to be further clarified. Moreover, the effects of this countermeasure on the overturning moments induced by solitary waves are rare studied, if any. It is extremely important to have a general understanding on this specific topic.

Hence, the primary objective in this work is to present a component level based analysis of the solitary wave forces, including the overturning moment, on a coastal bridge deck with girders. The secondary objective is to further explore the countermeasure of air venting holes with different sizes and its effects on the wave loadings. To fulfill these objectives, a numerical method is adopted by employing the commercial software ANSYS Fluent (v15.0, Academic Version) in the current study. The numerical simulations are carried by employing the Shear Stress Transport (SST) $k$-$\omega$ model. The solitary wave model, governing equations, and the numerical calculation domain are introduced firstly. Then, the solitary wave model is validated with the analytical results and with other related experimental studies. Three fixed still water levels (SWLs) with a variation of different structure elevations and four wave heights for each SWL are considered to investigate the characterizations of the overall wave forces on the bridge deck. The time histories of the horizontal force, vertical force, and moment are analyzed based on a component level approach. Afterwards, the countermeasure of air venting holes is investigated with different venting areas. The conclusions are finally presented.
3.2 Methodology and Validation

It is generally accepted that the wave forces on the physical models follow the Froude Similarity law. However, there are very limited studies related to the comparisons between the results on the full scale prototype and the scaled specimen. This is due to that full scale prototype model tests require much more resources as compared with scaled model tests and there are rare data recorded pertaining to the wave forces on the bridge decks in the extreme events. As such, the bridge deck-wave interaction problems are seldom verified with the data on the prototype bridges through the Froude Similarity law by experimental methods. Regarding this, the numerical methods exhibit their advantages as compared with the conventional methodologies (laboratory studies) such that variable scales and more scenarios can be conveniently realized once the numerical methodology is validated with the previous experimental studies. Hence, numerical approaches are more widely adopted in the bridge deck-wave interaction problems most recently.

Currently, very few 3D models have been studied due to the high computational cost (Bozorgnia and Lee 2012). As such, 2D numerical simulations of wave loadings on coastal bridge superstructures have been conducted in the present study. In the simulation process, the water is taken as an incompressible, viscous fluid. The fluid motion is described based on Navier-Stokes equations, which are shown as follows:

\[
\rho \left( \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} \right) = S_m
\]  
\[
\rho \left( \frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} \right) = -\frac{\partial p}{\partial x} + \mu \left( \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) + S_x
\]  
\[
\rho \left( \frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} \right) = -\frac{\partial p}{\partial y} - \rho g + \mu \left( \frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) + S_y
\]

Where \( \rho \) is the mass density; \( u, v \) are the velocity components in \( x \) and \( y \) directions, respectively; \( p \) is the pressure; \( \mu \) is the viscosity; \( g \) is the gravitational acceleration; \( S_m \) is the mass source; \( S_x, S_y \) are the momentum sources in the \( x \) direction and \( y \) direction, respectively. As a two-phase flow problem, the VOF (Volume of Fluid) method (Hirt and Nichols 1981) is employed to prescribe the dynamic free surface.

For the setups of the SST \( k-\omega \) turbulent model in Fluent, the pressure-based solver (segregated) is chosen for the transient flow, the Pressure-Implicit with Splitting of Operators (PISO) scheme (FHWA 2009; Bricker et al. 2012) is utilized for the pressure-velocity coupling method, and the PRESTO (PREssure STaggering Option) scheme is set for the pressure spatial discretization. Least squares cell based scheme is used for the gradient discretization, second order upwind for momentum advection terms, and Geo-Reconstruct for the volume fraction equations. The turbulence damping is turned on. For the velocity inlet boundary (AC) shown in Fig. 3.1(a), the turbulence intensity is 2% and the turbulence viscosity ratio is 10%. For the top (AB) and outlet (BD) of the calculation domain, the backflow turbulence intensity and the backflow turbulence viscosity ratio are the same as that set for the velocity inlet boundary. Second order upwind is used for the spatial discretization of the turbulence kinetic energy and the specific dissipation rate.

The 2nd order solitary wave model is chosen here to theoretically represent the tsunamis, where the wave length and period is deemed as infinite. The water particle
velocities $u$, $v$, the water pressure $p$, and the free surface profile $\eta$ of the solitary wave of the 2nd order (Sarpkaya and Isaacson 1981) are expressed as follows:

$$\frac{\eta}{d} = \varepsilon \text{sech}^2 q - \frac{3}{4} \varepsilon^2 \text{sech}^2 q \tanh^2 q$$  \hspace{1cm} (3.2a)

$$\frac{p}{\rho gd} = \frac{\eta}{d} + 1 - \frac{s}{d} - \frac{3}{4} \varepsilon^2 \text{sech}^2 q \left\{ \left( \frac{s}{d} \right)^2 - 1 \right\} (2 - 3 \text{sech}^2 q)$$  \hspace{1cm} (3.2b)

$$\frac{u}{\sqrt{gd}} = \varepsilon \text{sech}^2 q + \varepsilon^2 \text{sech}^2 q \left\{ \frac{1}{4} - \text{sech}^2 q - \frac{3}{4} \left( \frac{s}{d} \right)^2 (2 - 3 \text{sech}^2 q) \right\}$$  \hspace{1cm} (3.2c)

$$\frac{v}{\sqrt{gd}} = \varepsilon \sqrt{3\varepsilon} \left( \frac{s}{d} \right) \text{sech}^2 q \tanh q \left\{ 1 - \varepsilon \left[ \frac{3}{8} + 2 \text{sech}^2 q + \frac{1}{2} \left( \frac{s}{d} \right)^2 (1 - 3 \text{sech}^2 q) \right] \right\}$$  \hspace{1cm} (3.2d)

Where $\varepsilon = \frac{H}{d}$, $q = \frac{\sqrt{3\varepsilon}}{2d} \left( 1 - \frac{5}{8} \varepsilon \right) (x - ct)$, $s = y + d$, $d$ is the still water depth, and $H$ is the wave height, $y$ is the distance from and normal to the still water level, negative if with the same vector as the gravitational acceleration. The wave celerity $c$ is $c = \frac{c}{\sqrt{gd}} = 1 + \frac{1}{2} \varepsilon - \frac{3}{20} \varepsilon^2$. It is noted that the solitary wave crest is located at $x = 0$ when $t = 0$ s, i.e., the wave crest is just located at the inlet boundary. Hence, the incident solitary wave needs to be adjusted to make the water surface increase gradually at the inlet boundary (Dong and Zhan 2009). The numerical computation domain for the 2D simulations is shown in Fig. 3.1, where the line EF is the SWL, separating the regions of the air and water at the initial condition.

(a) Numerical domain for the 2D simulations

(b) Schematic of bridge deck components for the current numerical method

Fig. 3.1 Schematic diagram for computational domain with a typical coastal bridge deck with girders
For the boundary conditions, line AB is set as the pressure outlet with the pressure of air the same as the operating pressure (101325 pascal). Line AC is the velocity inlet with the equations of $u$ (3.2c), $v$ (3.2d) and the free surface profile $\eta$ (3.2a) compiled into Fluent. Line CD is the no slip stationary wall condition. Line BD is the pressure outlet to keep the balanced pressures for the air and the water zones. The geometries of the model bridge superstructure are firstly introduced here for convenience and will be employed later. This prototype bridge designed to carry 2-lane on the deck consists of a slab and six AASHTO type III girders supporting the slab and can be commonly found connecting island communities (Hayatdavoodi et al. 2014). The width of the superstructure is 10.45 m, the deck depth is 0.3 m, and the height of the six evenly distributed girders, simplified as rectangles, is 1.05 m.

The generated solitary waves are validated with the analytical ones near the location of the bridge model as shown in Fig. 3.2. In this figure, six different values of $\varepsilon$, 0.12, 0.18, 0.24, 0.30, 0.36, and 0.42, are considered based on the mesh sensitivity studies conducted in our previous study by Xu and Cai (2014). From Fig. 3.2 (f), a phase difference and a height difference are observed for $\varepsilon = 0.42$. The reasons are mainly attributed to: (a) higher orders neglected in the expressions of the 2nd order solitary wave theory may become more important for high ratios of $\varepsilon$; (b) the energy dissipates during the wave propagation; (c) numerical errors accumulate during the calculating process. The difference of the wave crest between the current method and the analytical solution is 5.3% when $\varepsilon = 0.42$, and it is 3.2% when $\varepsilon = 0.36$. However, the phase difference and height difference are much smaller with smaller $\varepsilon$ values. In summary, good results are obtained, especially when $\varepsilon \leq 0.24$. The accuracy of the solitary wave models are further validated with one experimental study by McPherson (2008) in our previous study (Xu and Cai 2014), and very good results are witnessed through the comparisons with the time histories of the wave forces that are obtained in the experimental study.

### 3.3 Characteristics of Solitary Wave Forces without Air Venting Holes

The physics of wave loadings on coastal bridges are complex due to the complicated structure geometries, various wave models (non-breaking or breaking, linear or nonlinear), and different topographic environments. In order to have a better understanding of the wave forces on typical coastal bridges, idealized conditions are prescribed with three different SWLs and twelve cases for each SWL, see Table 3.1. As shown in Table 3.1, the submersion coefficient $C_s$ is defined as the ratio of $S$, the distance between the bottom of the superstructure to SWL (here negative if the structure is submerged in the water), to $H_b$, the height of the bridge superstructure. The momentum center is the moment center due to the vertical force and horizontal force and it is located at the middle height of the deck. The $y$ coordinate of the momentum center changes with different submersion coefficients and different SWLs. It should be noted that the three SWLs with the same submersion coefficient shares the same case name, say, the cases when the submersion coefficient is 0 for SWL of 9.0 m, 7.2 m, and 5.4 m are all named “Case 3”. However, it is unique when discussing the case of “$d=9.0$ m, Case 4”.

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Fig. 3.2 Comparisons of the free surface profiles for solitary waves near the location of the bridge model. The bridge model is located at around 35m in the x direction from the inlet boundary.
Table 3.1 Structure elevations and corresponding coefficients

<table>
<thead>
<tr>
<th>Case</th>
<th>$S$(m)</th>
<th>$C_s = S/H_0$</th>
<th>Momentum Center</th>
<th>$x$ (m)</th>
<th>$y$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>0.60</td>
<td>0.444</td>
<td>35.225</td>
<td>10.8</td>
<td></td>
</tr>
<tr>
<td>Case 2</td>
<td>0.30</td>
<td>0.222</td>
<td>35.225</td>
<td>10.5</td>
<td></td>
</tr>
<tr>
<td>Case 3</td>
<td>0</td>
<td>0</td>
<td>35.225</td>
<td>10.2</td>
<td></td>
</tr>
<tr>
<td>Case 4</td>
<td>-0.30</td>
<td>-0.222</td>
<td>35.225</td>
<td>9.9</td>
<td></td>
</tr>
<tr>
<td>Case 5</td>
<td>-0.60</td>
<td>-0.444</td>
<td>35.225</td>
<td>9.6</td>
<td></td>
</tr>
<tr>
<td>Case 6</td>
<td>-0.90</td>
<td>-0.667</td>
<td>35.225</td>
<td>9.3</td>
<td></td>
</tr>
<tr>
<td>Case 7</td>
<td>-1.20</td>
<td>-0.889</td>
<td>35.225</td>
<td>9.0</td>
<td></td>
</tr>
<tr>
<td>Case 8</td>
<td>-1.50</td>
<td>-1.111</td>
<td>35.225</td>
<td>8.7</td>
<td></td>
</tr>
<tr>
<td>Case 9</td>
<td>-1.80</td>
<td>-1.333</td>
<td>35.225</td>
<td>8.4</td>
<td></td>
</tr>
<tr>
<td>Case 10</td>
<td>-2.10</td>
<td>-1.556</td>
<td>35.225</td>
<td>8.1</td>
<td></td>
</tr>
<tr>
<td>Case 11</td>
<td>-2.40</td>
<td>-1.778</td>
<td>35.225</td>
<td>7.8</td>
<td></td>
</tr>
<tr>
<td>Case 12</td>
<td>-2.70</td>
<td>-2.0</td>
<td>35.225</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>$d=9.0$m</td>
<td>Case 6</td>
<td>-0.90</td>
<td>35.225</td>
<td>7.2</td>
<td></td>
</tr>
<tr>
<td>Case 7</td>
<td>-1.20</td>
<td>-0.889</td>
<td>35.225</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>Case 8</td>
<td>-1.50</td>
<td>-1.111</td>
<td>35.225</td>
<td>7.2</td>
<td></td>
</tr>
<tr>
<td>Case 9</td>
<td>-1.80</td>
<td>-1.333</td>
<td>35.225</td>
<td>6.9</td>
<td></td>
</tr>
<tr>
<td>Case 10</td>
<td>-2.10</td>
<td>-1.556</td>
<td>35.225</td>
<td>6.6</td>
<td></td>
</tr>
<tr>
<td>Case 11</td>
<td>-2.40</td>
<td>-1.778</td>
<td>35.225</td>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td>Case 12</td>
<td>-2.70</td>
<td>-2.0</td>
<td>35.225</td>
<td>5.7</td>
<td></td>
</tr>
<tr>
<td>$d=7.2$m</td>
<td>Case 6</td>
<td>-0.90</td>
<td>35.225</td>
<td>5.7</td>
<td></td>
</tr>
<tr>
<td>Case 7</td>
<td>-1.20</td>
<td>-0.889</td>
<td>35.225</td>
<td>5.7</td>
<td></td>
</tr>
<tr>
<td>Case 8</td>
<td>-1.50</td>
<td>-1.111</td>
<td>35.225</td>
<td>5.4</td>
<td></td>
</tr>
<tr>
<td>Case 9</td>
<td>-1.80</td>
<td>-1.333</td>
<td>35.225</td>
<td>5.1</td>
<td></td>
</tr>
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<td>Case 10</td>
<td>-2.10</td>
<td>-1.556</td>
<td>35.225</td>
<td>4.8</td>
<td></td>
</tr>
<tr>
<td>Case 11</td>
<td>-2.40</td>
<td>-1.778</td>
<td>35.225</td>
<td>4.2</td>
<td></td>
</tr>
<tr>
<td>Case 12</td>
<td>-2.70</td>
<td>-2.0</td>
<td>35.225</td>
<td>3.9</td>
<td></td>
</tr>
</tbody>
</table>

Four wave heights for each SWL are chosen with the parameters listed in Table 3.2. In the table, $L_e$ is the effective wave length, $L_e = 2\pi d/\sqrt{3H/d}$, and $t_0$ is the adjusted time, calculated by the effective wave length, $L_e$, divided by the wave celerity, $c$. The
simulation time, \( t \), is set long enough to let the solitary wave pass through the bridge superstructure and the time step is set as 0.005 s for all the cases studied here.

Table 3.2 Wave heights and related parameters for numerical simulation

<table>
<thead>
<tr>
<th>( d ) (m)</th>
<th>( H ) (m)</th>
<th>( \varepsilon = H/d )</th>
<th>( L_e ) (m)</th>
<th>( c ) (m/s)</th>
<th>( t_0 )</th>
<th>simulation time ( t ) (s)</th>
<th>( \Delta t ) (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.0</td>
<td>2.20</td>
<td>0.244</td>
<td>66.03</td>
<td>10.46</td>
<td>7</td>
<td>17</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>1.80</td>
<td>0.2</td>
<td>73.0</td>
<td>10.28</td>
<td>8</td>
<td>19</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>1.40</td>
<td>0.156</td>
<td>82.78</td>
<td>10.09</td>
<td>9</td>
<td>21</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>0.111</td>
<td>97.95</td>
<td>9.90</td>
<td>10</td>
<td>24</td>
<td>0.005</td>
</tr>
<tr>
<td>7.2</td>
<td>2.20</td>
<td>0.306</td>
<td>47.25</td>
<td>9.57</td>
<td>5</td>
<td>15</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>1.80</td>
<td>0.25</td>
<td>52.23</td>
<td>9.38</td>
<td>6</td>
<td>16</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>1.40</td>
<td>0.194</td>
<td>59.23</td>
<td>9.17</td>
<td>7</td>
<td>18</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>0.139</td>
<td>70.08</td>
<td>8.96</td>
<td>8</td>
<td>21</td>
<td>0.005</td>
</tr>
<tr>
<td>5.4</td>
<td>2.20</td>
<td>0.407</td>
<td>30.69</td>
<td>8.58</td>
<td>4</td>
<td>13</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>1.80</td>
<td>0.333</td>
<td>33.93</td>
<td>8.37</td>
<td>4.5</td>
<td>14</td>
<td>0.005</td>
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<tr>
<td></td>
<td>1.40</td>
<td>0.259</td>
<td>38.47</td>
<td>8.15</td>
<td>5</td>
<td>16</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>1.00</td>
<td>0.185</td>
<td>45.52</td>
<td>7.91</td>
<td>6</td>
<td>18</td>
<td>0.005</td>
</tr>
</tbody>
</table>

For the mesh resolutions in the computational domain with the bridge superstructure, \( \Delta x \) is set as 0.2 m, 0.1 m, and 0.2 m for the near velocity inlet zone, main computational zone, and far field from the main computational zone, respectively; and \( \Delta y \) is set as 0.4 m, 0.1 m, and 0.2 m for the air zone, the near water zone, and the deep water zone, respectively. Structured meshes are mainly used and fine meshes are considered near the walls of the bridge model. One example of the grid mesh nearby the bridge model is shown in Fig. 3.3.

Fig. 3.3 Grid mesh nearby the bridge model

### 3.3.1 Time Histories of Wave Forces

All the wave forces on the bridge superstructure are monitored and recorded since the simulation begins. Fig. 3.4 demonstrates one example of wave structure interaction for the case of “\( d = 7.2 \) m, \( H = 1.40 \) m, Case 3”. For this case, the elevation of the bottom of the superstructure is located at the SWL. It is observed that there exists a large amount of entrapped air between the girders due to the incompressible flow in the current 2D simulations. The time histories of the solitary wave forces for this case are shown in Fig. 3.5, where the letters (a) to (f) refer to the snapshots as plotted correspondingly in Fig. 3.4.

From Fig. 3.5, it is shown that the vertical force and the horizontal force reach their positive peak values at around 10s. But the magnitude of the positive peak vertical force is about 11.7 times of that of the positive peak horizontal force. Apparently, the positive peak vertical force (117.2 kN per unit length) surpasses the self-weight of the bridge span (about 95.3 kN per unit length or 1510 kN per span) under this condition
The moment reaches its negative peak value (clockwise, normal to the paper) at around 9 s and the positive peak value (counterclockwise, normal to the paper) at around 12 s, which varies with the changing vertical and horizontal forces when the wave is propagating. In the wave propagation, it is predicted that when the wave front strikes the seaward girder (snapshot (b) in Fig. 3.4), the uplift force causes the negative moment on the bridge deck. Then, when the wave crest reaches around the middle of the bridge deck, the vertical force may be evenly distributed along the deck (snapshot (c) in Fig. 3.4), resulting in smaller moments. Further, when the landward decks (Deck 4 and Deck 5) have relatively larger vertical uplift forces (snapshot (e) in Fig. 3.4), the positive peak moment is hence induced. As the wave propagates away, the wave forces on the bridge deck decrease to zero. In the above discussion, the moment induced by the vertical force is expected larger than that induced by the horizontal force owing to that the vertical force is much larger than the horizontal force and the vertical force has a longer arm to the moment center.

Fig. 3.4 Wave structure interaction for the case of “$d = 7.2$ m, $H = 1.40$ m, Case 3”. (a) $t = 8.0$ s; (b) $t = 9.0$ s; (c) $t = 10.0$ s; (d) $t = 11.0$ s; (e) $t = 12.0$ s; (f) $t = 13.0$ s.

Fig. 3.5 Time histories of the wave forces for the case of “$d=7.2$ m, $H = 1.40$ m, Case 3”
3.3.2 Effects of the Submersion Coefficient on the Wave Forces

The positive (landward) and negative (seaward) peak horizontal forces for the studied cases are plotted in Fig. 3.6, where the values vary with the submersion coefficient for each SWL considered. Generally, the positive peak horizontal forces are following similar trends in such a way that the maximum positive horizontal force occurs when the bridge superstructure is just fully submerged ($C_s = -1.111$ when $d = 9.0$ m and 7.2 m, and $C_s = -1.333$ when $d = 5.4$ m), i.e., the top of the bridge superstructure is 0.15 m and 0.45 m below the SWL, respectively. While the negative horizontal forces are comparatively smaller than the positive horizontal forces, they should not be neglected in designing and retrofitting bridges.

![Fig. 3.6](image)

Fig. 3.6 Variation of the positive and negative peak horizontal forces with the submersion coefficient
Fig. 3.6 (continued) Variation of the positive and negative peak horizontal forces with the submersion coefficient

For the vertical forces as shown in Fig. 3.7, the normalized vertical forces \( (F_v/F_b) \) increase very fast from Case 1 \( (C_s = 0.444) \) to Case 3 \( (C_s = 0) \) for each wave height at different SWLs, and then decrease slowly with the submersion coefficient decreases (to be more negative), where \( F_v \) is the positive (upward) peak vertical force for each case considered and \( F_b \) is the bridge self-weight. At most times, the maximum values of the normalized vertical forces occur at Case 4 \( (C_s = -0.222) \) for different SWLs such that the bottom of the superstructure is submerged in the water by 0.3m.

Fig. 3.7 Variation of the positive peak vertical forces with the submersion coefficient
Fig. 3.7 (continued) Variation of the positive peak vertical forces with the submersion coefficient

The peak moment on the bridge superstructure for the studied cases is shown in Fig. 3.8. Generally speaking, the positive peak moment is larger than the negative peak moment when the submersion coefficient $C_s$ is larger than -0.667. While when the submersion coefficient $C_s$ is smaller than -1.111, the positive peak moment is much smaller than the negative peak moment and hence they can be neglected. It is observed that when the bridge superstructure is located around the SWL, both the positive and negative peak moment tends to have relatively larger values as compared with those fully submerged cases.

3.3.3 Effects of Different SWLs on the Wave Forces

When considering the horizontal forces with different SWLs for one chosen wave height ($H = 1.00$ m) as shown in Fig. 3.9, it is observed that the positive horizontal forces with a lower SWL are larger than those with a higher SWL. This may be attributed to that
the horizontal velocities of the water particles at around the crest section (i.e., the positive peak horizontal force occurs when the wave crest arrives at the front of the bridge superstructure) with a lower SWL are larger than those with a higher SWL based on Eqn. (2c), see Fig. 3.10.

Fig. 3.8 Variation of the positive and negative peak moment with the submersion coefficient

Fig. 3.9 Variation of the horizontal forces with the submersion coefficient for different SWLs ($H = 2.20$ m)
Fig. 3.10 Horizontal velocities of the water particles at the crest section when $H=1.00$ m

Fig. 3.11 shows the comparison of the vertical forces with the submersion coefficient for one chosen wave height ($H = 1.40$ m) between different SWLs, where the vertical forces with a higher SWL tends to be larger than those with a lower SWL when the submersion coefficient $C_s$ is larger than -0.444. It is predicted that the larger wave celerity $c$ with a higher SWL leads to a rapider rising pressure for the entrapped air between the girders underneath the bridge superstructure, resulting in relatively larger vertical forces. Meanwhile during the process, the effects of the green water on deck should be considered. For example, when the submersion coefficient is -0.444, i.e., the bottom of the superstructure is submerged into the water by 0.6m and the top of the superstructure is above the SWL by 0.75 m. At this time, the green water on the deck is observed for the four wave heights adopted and the amount of the green water (resulting in the green water load, downward) differs with different wave heights.

Fig. 3.11 Variation of the vertical forces with the submersion coefficient for different SWLs ($H = 1.40$ m)

65
When the bridge superstructure is fully submerged and beyond, i.e., the submersion coefficient $C_s$ is smaller than -1.0, the vertical forces with a higher SWL are smaller than those with a lower SWL. The reason may be that the separation of the wave flow by the bridge superstructure leads to a larger portion of the green water on deck due to the larger wave celerity $c$ with a higher SWL. Thus, the smaller vertical forces are induced with a higher SWL.

The general trend of the moment with different SWLs for one chosen wave height ($H = 1.80$ m) is demonstrated in Fig. 3.12. As shown in this figure, when the submersion coefficient $C_s$ is larger than -0.667, both the positive and negative moments with a lower SWL are larger than those with a higher SWL. However, after the superstructure is fully submerged, the negative moment with a higher SWL is larger than that with a smaller SWL, and the positive moment can be neglected with a much smaller value.

![Fig. 3.12](image)

**Fig. 3.12** Variation of the moment with the submersion coefficient for different SWLs ($H = 1.80$ m)

### 3.3.4 Characteristics of the Wave Forces Based on Component Level

In order to strengthen the understandings of the loading process in local areas, the wave forces are analyzed on the component level (the components of the bridge superstructure are shown in Fig. 3.1(b)). Three typical cases are discussed here for the demonstration purpose, “$d = 7.2$ m, $H = 1.00$ m, Case 4”, “$d = 7.2$ m, $H = 1.80$ m, Case 3”, and “$d = 7.2$ m, $H = 2.20$ m, Case 9”, and the results are shown in Fig. 3.13, 3.15, and 3.16, respectively.

In Fig. 3.13 (a), it is observed that Girder 6 bears a larger positive horizontal force than the other girders and Girder 1 has a larger negative one. Generally speaking, the pressure in the air chambers partitioned by the girders underneath the bottom of the bridge deck mainly causes the forces normal to the surfaces of the objects, say, the horizontal forces on the girder components (Girder 1 to Girder 6) and vertical forces on the deck components (Deck 1 to Deck 5), as shown in Fig. 3.13 (a) and (b), respectively. It is observed that the water at the trailing edge of the bridge superstructure rises to some
extent when the positive peak horizontal force occurs (similar to the snapshot as shown in Fig. 3.4 (d)). However, the portion of the horizontal force on Girder 6 due to the pressure in the most landward air chamber (landward) overcomes that due to the existence of the raised up water at the trailing edge (seaward), resulting in the positive horizontal force on Girder 6, as shown in Fig. 3.14 (c), where the total pressure for this case at different time is plotted. It is also observed that for the girders located in between two air chambers (Girder 2 to Girder 5), the pressure induced horizontal force on the seaward side of the girder is larger than that on the landward side of the girder such that the positive horizontal forces are acquired but with smaller magnitude when compared with that on Girder 6. The horizontal force on Girder 1 varies along with the wave propagation. At about $t = 9$ s (Fig. 3.14 (a)), the integrated value of the total pressure on the landward side of Girder 1 is obviously larger than that on the seaward side and this causes the occurrence of the negative peak horizontal force on Girder 1. The deck components (Deck 1 to Deck 5) located at the bottom of the deck between the girders, actually, bear no horizontal force owing to that the entrapped air tends to exert the pressure normal to the deck components without the force component in the horizontal direction.

In Fig. 3.13 (b), the uplift force (refers to the positive peak vertical force) on Deck 1 is larger than those on the other four deck components due to the energy dissipation with the propagation of the wave, while they are almost of the same magnitude. The uplift force on the girders shows the similar phenomenon that Girder 1 bears larger uplift force than the other girder components. In addition, the green water on the Deck top causes a negative vertical force, but with a much smaller magnitude when compared with the uplift forces on the other five deck components. Generally speaking, the pressure at the bottom of the girders is larger than that at the bottom of the decks (see Fig. 3.14), though the uplift forces on the deck components are larger than those on the girder components. This is because the area of the deck components is about 6 times that of the girder components in the vertical direction. Actually, during the bridge deck-wave interaction process for this case, the water particles do not touch the bottom of the deck components such that the uplift forces are mainly caused by the entrapped air. We expect that the assumption of the incompressible air in the numerical simulations would have no significant effects on the pressure distribution in the air chambers.

In Fig. 3.13 (c), it is noted that Deck 5 provides the largest positive moment and Deck 1 the largest negative moment, but the peak values occur at different time. The moment on the deck components (except Deck 3, where the momentum center is located) are larger than that on the girder components. Since there are no horizontal forces on the deck components, the expectation that the moment induced by the vertical force is larger than that induced by the horizontal force can be confirmed. For this case, the moment by the green water on deck is not a concern since it is comparatively small. The time the peak moment on one component occurs may coincide with the time the peak force (horizontal or vertical) on that component occurs. However, on the overall wave forces, there may be no full correlation between the positive peak moment and the positive peak horizontal or vertical force (see Fig. 3.5).
Fig. 3.13 Time histories of solitary wave forces based on the component level for the case of “$d = 7.2$ m, $H = 1.00$ m, Case 4”

Fig. 3.14 Total pressure (with respect to the operating pressure, 101325 Pa) for the case of “$d = 7.2$ m, $H = 1.00$ m, Case 4” at different time. (a) $t = 9$ s; (b) $t = 10$ s; (c) $t = 11$ s; (d) $t = 12$ s.
For the results of the case of “$d = 7.2$ m, $H = 1.80$ m, Case 3” as shown in Fig. 3.15, the wave forces on the component level share much similarity with the case of “$d = 7.2$ m, $H = 1.00$ m, Case 4” as discussed above. However, one difference should be noted that the green water on the Deck top causes a large negative vertical force and hence a large moment.

![Diagram showing horizontal, vertical forces, and moment](image)

Fig. 3.15 Time histories of solitary wave forces based on the component level for the case of “$d = 7.2$ m, $H = 1.80$ m, Case 3”

Since the bridge superstructure is fully submerged in the case of “$d = 7.2$ m, $H = 2.20$ m, Case 9”, the green water on the Deck top is of significant magnitude, as shown in Fig. 3.16. However, the moment on the Deck top counterweighs the positive moment caused by other components, especially on Deck 4 and 5. As a result, the positive peak moment on the whole bridge superstructure is negligible when the submersion coefficient is negative, see Fig. 3.8 (b).
Fig. 3.16 Time histories of solitary wave forces based on the component level for the case of “$d = 7.2$ m, $H = 2.20$ m, Case 9”

3.4 Characteristics of Solitary Wave Forces with Air Venting Holes - the Countermeasure

It is recognized that the countermeasure of the air venting holes is an effective way for retrofitting the bridge superstructure that are vulnerable to tsunamis or hurricanes (Cuomo et al. 2009; Bozorgnia et al. 2010; Hoshikuma et al. 2013; Hayatdavoodi et al. 2014). Hence, the current study makes further explorations on this subject aiming to provide an insight view in this direction. In the following discussions, four different venting ratios (based on the deck area) are considered, the total pressure in the air chamber is monitored, the flow rate of the air and water through the air venting holes under different wave heights are recorded, and the characteristics of solitary wave forces with the air venting holes are analyzed.
In order to accommodate the 2D numerical simulations, the air venting holes are equivalently modeled as slots in the deck that are continuous along the unit length of the bridge superstructure (normal to the paper). In practical projects, circular holes may be considered as long as the venting areas are the same as the long slots per unit length. Thus, the process of the air releasing may follow the same way. This will enable us to obtain the general observations regarding the performance of the wave loading when this countermeasure is adopted.

Currently, five air venting holes are considered and they are located in the middle of the deck components (Deck 1 to Deck 5), see Fig. 3.17. In this figure, nine points (P 1 to 9) are well distributed in the air chamber and they are used to monitor the change of the total pressure in the air chamber, where the water will replace part of the air in the chamber with the presence of the air venting holes during the wave propagation. Lines 1 and 2 are located at the top and bottom of the air venting hole in Deck 1, respectively, and they are employed to acquire the information of the flow rate of the air and water through the air venting holes as they escape from the chamber.

The cases of “d = 7.2 m, Case 3” with four wave heights (1.00 m, 1.40 m, 1.80 m, and 2.20 m) are considered here for demonstration purpose, where the bottom of the bridge superstructure is just located at the SWL. At this time, the vertical force and the moment are of relative large values (see Figs. 3.7 (b) and 3.8 (b)), and it is interesting to assess the performance of the wave loadings under these prescribed conditions. Four different venting ratios (based on the deck area) are considered, as shown in Table 3.3. In fact, Vent 0 is the case that no air venting holes are considered, while it is listed here for the comparison purpose. One example of the grid mesh nearby the bridge model in the computational domain (Vent 2) is demonstrated in Fig. 3.18.

<table>
<thead>
<tr>
<th>Ratio (%)</th>
<th>Deck area (m²)</th>
<th>Number of Slots</th>
<th>Slot width, required (m)</th>
<th>Slot width, designed in the model (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vent 0</td>
<td>0</td>
<td>10.45</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Vent 1</td>
<td>1</td>
<td>10.45</td>
<td>5</td>
<td>0.0209</td>
</tr>
<tr>
<td>Vent 2</td>
<td>2</td>
<td>10.45</td>
<td>5</td>
<td>0.0418</td>
</tr>
<tr>
<td>Vent 3</td>
<td>3</td>
<td>10.45</td>
<td>5</td>
<td>0.0627</td>
</tr>
</tbody>
</table>

Fig. 3.17 Schematic of the air venting holes in the deck. P 1 refers to Point 1.
3.4.1 Total pressure in the Chamber

The total pressure value is the absolute pressure with respect to the operating pressure (101325 Pa) and for incompressible fluid, it is defined as

\[ P_0 = P_s + \frac{1}{2} \rho |\vec{v}|^2 \]  

where \( P_0 \) is the Total pressure, \( P_s \) is the static pressure, and \( \vec{v} \) is the particle velocity. The monitored total pressure for the nine points in the case of “\( d = 7.2 \) m, \( H = 1.40 \) m, Case 3” with the four venting ratios is recorded as shown in Fig. 3.19. In order to facilitate the discussions pertaining to the total pressure in Fig. 3.19, the snapshots for the bridge deck-wave interaction at the appropriate time the positive peak horizontal and vertical forces occur are shown in Fig. 3.20 and the contours of the total pressure for Vent 0 and Vent 1 at the corresponding time are demonstrated in Fig. 3.21.

Fig. 3.18 Grid mesh nearby the bridge model for Vent 2

Fig. 3.19 Time histories of the Total pressure monitored at the target points
It is interesting to notice in Fig. 3.19 that: (a) the total pressure for the nine points in the case of Vent 0 arise along the wave propagation and the peak values are of the same magnitude, indicating a uniformly distributed pressure field is held on (see Fig. 3.21 (a)), and this confirms the observations found in other studies (Evans 1981; Hayatdavoodi et al. 2014); (b) for the other three venting ratios (Vents 1, 2, and 3), the first and large spike in each time histories of the total pressure is probably caused by the impact when the rapidly rising water (in the chamber) immediately touches the bottom of the deck (Deck 1) and then, it is followed by slowly varying quasi-static pressure, which is much smaller than the corresponding one for Vent 0; (c) the curves of the total pressure for Vents 1 to 3 present three distinguishable clusters before the pressure values become negative, say, Points 1 to 3 bear the same magnitude of the total pressure with the values smaller than those for Points 4 to 6 and Points 7 to 9 (see Fig. 3.21 (b)); (d) there is a slight difference of the occurrence time of the first spike in the total pressure curves among Vents 1 to 3, indicating that a larger venting ratio may make the air releasing relatively faster, resulting in a smaller spike in the pressure field (and hence the impact force on the deck); (e) when the wave crest passes the chamber (the monitored one) for Vents 0 to 3, the total pressure begins to fall off from the peak values. The witness of the negative total pressure with oscillating values represents that the water level descends in the chamber and during the process, the green water on deck drops through the air venting holes. As a result, the negative vertical force will be induced and this will be discussed later.

Fig. 3.20 Snapshots for the bridge deck-wave interaction at the appropriate time the positive peak horizontal and vertical forces occur (“$d = 7.2$ m, $H = 1.40$ m, Case 3”, $t = 9.8$ s). (a) Vent 0; (b) Vent 1; (c) Vent 2; (d) Vent 3.

The water shooting out through the air venting holes is captured for Vents 1 to 3, as shown in Fig. 3.20. However, this is closely related with the difference of the total pressure between the top (Line 1) and bottom (Line 2) of the air venting holes. As such, the averaged total pressure at Lines 1 and 2 are recorded and plotted in Fig. 3.22 in order to gain an insight of this process. The area-weighted average of the total pressure is
calculated by dividing the summation of the product of the total pressure and the facet area (per mesh) at Line 1 or 2 by the total area of Line 1 or 2 (actually, it is the length of Line 1 or 2 since it is in 2D simulations), and the equation is given as follows

\[ \frac{1}{A} \int P_0 \, dA = \frac{1}{A} \sum_{i=1}^{n} P_{0i} \left| A_i \right| \]

where \( A \) is the total area, \( P_{0i} \) is the total pressure at the \( i \)th facet area, and \( A_i \) is the \( i \)th facet area.

Fig. 3.21 Total pressure (with respect to the operating pressure, 101325 Pa) for the bridge deck-wave interaction at the appropriate time the positive peak horizontal and vertical forces occur (“\( d = 7.2 \) m, \( H = 1.40 \) m, Case 3”, \( t = 9.8 \) s). (a) Vent 0; (b) Vent 1.

Fig. 3.22 Time histories of the averaged total pressure at the top and bottom of the air venting hole (the monitored one)

As shown in Fig. 3.22, it is confirmed that the peak value of the first spike at Line 2 for Vent 1 is much larger than that for Vent 3, similar to that as observed in Fig. 3.19 with the occurrence of the first spike, indicating that the impact of the water on the bottom of the deck (Deck 1) results in a sudden increase for the pressure field in the whole chamber. Moreover, the averaged total pressure at Line 2 is obviously larger than
that at Line 1 in a time period from when the first spike occurs to about 10.3s. The
difference of the averaged total pressure during this period indicates that the water will be
pushed out if almost all the air is already escaped from the chamber. This results in a
need to monitor the air and water flow through the air venting holes during this process
and it is discussed next.

3.4.2 Flow Rate of the Air and Water through the Air Venting Holes

The flow rate of the air through a prescribed area is computed by summing the
value of the VOF (volume of fluid) factor multiplied by the density of air and the dot
product of the facet area and the facet velocity vector, and the equation is

$$\int \Phi_{\text{air}} \vec{v} \cdot \vec{A} = \sum_{i=1}^{n} \Phi_{i} \rho_{\text{air}} \vec{v}_{i} \cdot \vec{A}_{i}$$

(3.5)

where $\Phi_{i}$ is the VOF factor pertaining to the $i$th facet area and it ranges from 0 to 1 (1
refers full of air and 0 refers no air) and $\vec{v}_{i}$ is the velocity vector at the $i$th facet area. It is
treated in the same way for the calculation of the flow rate of the water.

(a) air phase
(b) Water phase

Fig. 3.23 Time histories of the flow rate through the top and bottom of the air venting
hole for Vent 1 in the case of “$d = 7.2$ m, $H = 1.40$ m, Case 3”

The flow rate of the air and water through Lines 1 and 2 for Vent 1 in the case of
“$d = 7.2$ m, $H = 1.40$ m, Case 3” are plotted in Fig. 3.23 and a few snapshots for this case
are captured as shown in Fig. 3.24 to strengthen the understanding. At $t = 9$ s, there only
left a small portion of air in the monitored chamber, as shown in Fig. 3.24 (a) and the air
stops to escape from the chamber at about $t = 9.2$ s. It is at the same time (the first spike
takes place) that the water begins to shoot out through the air venting holes due to the
significant difference of the averaged total pressure between Lines 1 and 2, as shown in
Fig. 3.22. At about $t = 10.4$ s, the flow rate of the water turns to be negative, which means
that the green water on the deck begins to drop off through the air venting hole, and the
green water will keep on falling for a long time. At about $t = 12$ s, the air is drawn back
into the chamber accompanying with the fallen water and this probably induces the
oscillations in the time histories of the total pressure at the monitored nine points as
shown in Fig. 3.19.
Fig. 3.24 Snapshots to illustrate the flow rate through the top and bottom of the air venting hole for Vent 1 in the case of “$d = 7.2 \text{ m}, H = 1.40 \text{ m, Case 3}$”. (a) $t = 9 \text{ s}$; (b) $t = 10 \text{ s}$; (c) $t = 11 \text{ s}$; (d) $t = 12 \text{ s}$.

The comparisons of the flow rate of the air and water among Vents 1 to 3 are shown in Fig. 3.25. In this figure, it is worth noting that there is no significant difference for the volume of the escaped air among Vents 1 to 3 (based on the integration of the flow rate of air), indicating that when the wave crest arrives at the monitored chamber, the entrapped air can escape through the air venting hole of Vent 1 in a timely manner. It is also noted that for a larger venting ratio (Vent 3), more water will be pushed out and drawn back through the air venting hole.

Fig. 3.25 Comparisons of the flow rate of the air and water through Line 2 between different venting ratios in the case of “$d = 7.2 \text{ m}, H = 1.40 \text{ m, Case 3}$”
3.4.3 Overall Wave Forces

Based on the time histories of wave forces for the considered venting ratios in the case of “$d = 7.2$ m, $H = 1.40$ m, Case 3” as shown in Fig. 3.26, the horizontal forces with air venting holes are larger than that without the air venting holes and this confirms the similar observation documented by Hayatdavoodi et al. (2014). It is noteworthy that the vertical forces with air venting holes display a favorable reduction as compared with that without the air venting holes. In addition, several spikes are recognized as the slamming forces that are induced by the rapidly rising water impinging the bottom of the deck. Plus, negative vertical forces are recorded in the time histories and this is reflected from the presence of the negative total pressure for the nine points as shown in Fig. 3.19.

Fig. 3.26 Time histories of wave forces for different venting ratios in the case of “$d = 7.2$ m, $H = 1.40$ m, Case 3”
As for the moment, the positive peak ones are of the same magnitude, but the negative peak ones (based on the quasi-static level and the slamming force are excluded in order to make convenient comparisons) have smaller values as compared with that for Vent 0. The characteristics of the slamming forces were studied extensively in the study by Cuomo et al. (2009).

The results of the wave forces (slamming forces excluded) for the prescribed wave heights (1.00 m, 1.40 m, 1.80 m, and 2.20 m) are shown in Fig. 3.27. As is evident from the figure, the countermeasure of the air venting holes have detrimental effects on the horizontal forces for the four wave heights and on the positive moment when $H = 2.20$ m, while it significantly benefits the bridge deck with much smaller loadings of the vertical forces and the negative moment for the four wave heights. It can be concluded that Vent 1 is good enough to produce favorable results.

The calculation of the portion of uplift force that directly related to the hydrostatic force of the trapped air is based on the following equations (Hayatdavoodi et al. 2014)

\[ F_{\text{hydro}} = \rho g V_{\text{air}} \]  
\[ V_{\text{air}} = C_r n S_G h_c \]

where $V_{\text{air}}$ is the volume that is originally occupied by the air and then replaced by the water in the chambers when the air venting holes are adopted; $n = 5$ is the number of the chambers; $C_r$ is the reduction factor since there will still be some air in the chambers at the time the positive vertical forces occurs and $C_r = 0.90$ based on the snapshots in the case when $H = 1.40$ m, as shown in Fig. 3.20. Further checks are made for other wave heights at the time the positive vertical forces take place and the acquired values of $C_r$ are listed in Table 3.4; $S_G = 1.73$ m is the horizontal space between two girders; $h_c$ is measured from the SWL to the bottom of the deck when the bottom of girders is below
the SWL (submerged cases) or from the bottom of girders to the bottom of the deck when the bottom of girders is above the SWL (suspended cases), and $h_c = 1.05 \text{ m}$ in this specific case. Since the uplift forces for Vents 1 to 3 are of the same magnitude, the analysis of the air pockets caused the hydrostatic forces for the four wave heights is discussed between the results of Vent 0 and Vent 1 and the results are listed in Table 3.4.

![Graphs showing variation of wave forces](image)

Fig. 3.27 Variation of the wave forces considering different wave heights for the studied venting ratios

The results show that the hydrostatic force due to the entrapped air occupies a larger portion of the total uplift force when the wave height is smaller, while the portion of the hydrodynamic force (the total uplift force minus the hydrostatic force) in the total uplift force increases with the increase of the wave height (see the last row in Table 3.4). This confirms the same observation in the study by Hayatdavoodi et al. (2014). As such, the entrapped air in the chamber has effects on both of the hydrostatic force and the hydrodynamic force.
3.5 Concluding Remarks

In the present study, the solitary wave models are employed to estimate the wave loadings on a typical coastal bridge deck with girders under three different SWLs. This parametric study shows that for each wave height with a viable submersion coefficient, the maximum horizontal forces occur when the bridge superstructure is just fully submerged and the maximum vertical ones occur when the bottom of the bridge superstructure is at around the SWL. The positive moments tend to be larger than the negative moments for one specific case before the bridge superstructure is fully submerged; however, it goes to the opposite when the bridge superstructure is fully submerged.

Table 3.4 Analysis of the air pockets caused hydrostatic forces (slamming forces excluded)

<table>
<thead>
<tr>
<th>Wave heights</th>
<th>$H = 1.00$ m</th>
<th>$H = 1.40$ m</th>
<th>$H = 1.80$ m</th>
<th>$H = 2.20$ m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total uplift force, Vent 0 (kN)</td>
<td>91.4</td>
<td>115.3</td>
<td>138.5</td>
<td>156.3</td>
</tr>
<tr>
<td>Reduced uplift force, Vent 1 (kN)</td>
<td>16.2</td>
<td>42.8</td>
<td>69.6</td>
<td>92.7</td>
</tr>
<tr>
<td>Force reduction (kN)</td>
<td>75.2</td>
<td>72.5</td>
<td>68.9</td>
<td>63.6</td>
</tr>
<tr>
<td>Ratio of the reduction to the total uplift force</td>
<td>82%</td>
<td>63%</td>
<td>50%</td>
<td>41%</td>
</tr>
<tr>
<td>$C_r$</td>
<td>0.78</td>
<td>0.90</td>
<td>0.93</td>
<td>0.95</td>
</tr>
<tr>
<td>$F_{hydrostatic}$ due to the entrapped air (kN)</td>
<td>68.9</td>
<td>79.5</td>
<td>82.1</td>
<td>83.9</td>
</tr>
<tr>
<td>Ratio of the $F_{hydrostatic}$ to the total uplift force</td>
<td>75%</td>
<td>69%</td>
<td>59%</td>
<td>54%</td>
</tr>
</tbody>
</table>

When considering the maximum wave forces with different still water depths, interesting phenomena were found: (a) the positive horizontal forces at lower water depth are larger than those at higher water depth. This is probably due to the reason that the horizontal water particle velocities at the crest section at lower water depth are larger than those at higher water depth; (b) the uplift forces at higher water depth are larger than those at lower water depth before the superstructure is fully submerged, while the uplift forces at higher water depth are smaller than those at lower water depth when fully submerged; (c) the positive and negative moments at lower water depth are larger than the corresponding ones at higher water depth.

As for the countermeasure of the air venting holes, several observations are documented as follows: (a) the venting ratio of 1% with five evenly distributed venting holes based on the whole area is enough to mitigate the vertical forces on the bridge decks; (b) the hydrostatic force due to the entrapped air contributes a larger portion of the total uplift force when the wave height is smaller, while the portion of the hydrodynamic force in the total uplift force increases with the increase of the wave height.
3.6 References


CHAPTER 4. AN INVESTIGATION OF WAVE FORCES ON BILOXI BAY BRIDGE DECKS WITH INCLINATIONS UNDER SOLITARY WAVES

4.1 Introduction

Hurricanes and tsunamis, accompanied with the high storm surge and waves, have caused devastating impact on coastal communities. A tsunami in 1946 killed 150 people in Hilo, Hawaii and a tsunami in 1964 stroked the Pacific coast of the United States with 12 casualties and millions of dollars loss in northern California and Oregon. One of the most disastrous tsunami, the 2004 Indian Ocean Tsunami, caused more than 225,000 lives, and destroyed entire cities and communities (FHWA 2008). The 2005 hurricane Katrina and the 2011 Great East Japan Tsunami caused thousands of casualties and catastrophes to coastal structures, and demonstrated an urgent need to predict wave forces on coastal bridges that are the backbones of the transportation lines, especially in these extreme cases (Graumann et al. 2005, USAID 2005, Shoji and Moriyama 2007, Yeh et al. 2007, Ghobarah et al. 2006, Bricker et al. 2012).

During these natural events, low-lying coastal bridges are vulnerable to the wave attacks. From the post disaster survey, it is concluded that hurricane induced wave forces or tsunami forces tend to cause damage to both bridge superstructures and substructures. Bridge failures due to unseating and displacement of bridge decks are mainly caused by the combination of the lift forces, drag forces, and moments. Curved bridges (bridge decks with inclinations) were also experienced similar failures, see Fig. 4.1 (Gilberto Mosqueda 2005). These coastal bridges are crucial to coastal communities during and after the disastrous event for the evacuation purpose. Due to the complex geometries of coastal bridge structures, it is difficult to analyze wave loads, including hydrodynamic loads, current induced scour, and flood borne debris, theoretically using current design methods. The failure mechanisms of curved bridges under wave-induced forces are still unclear now and no current design codes are available for this problem. Therefore, in order to correctly estimate the wave forces on curved bridges, effects of different inclinations must be carefully studied.

Many experimental studies and numerical simulations of tsunami or hurricane induced wave forces on coastal bridges have been conducted during the last several decades. In these studies, Denson (1980) investigated the characteristics of five angles of wave incidences, i.e, 30°, 45°, 60°, 75° and 90°, on a slab/beam bridge model and a box girder bridge model through experimental analysis. Both bridge models include two lanes, landward and seaward. The trapezoidal box girder has a 1:10 (5.71° inclination) super-elevation to seaward, and the results were plotted in dimensionless form. However, no comparisons of the results of inclined bridge decks and level bridge decks were made because a further study on a corresponding level box girder bridge was not conducted. Cuomo et al. (2009) conducted laboratory experiments on a 1:10 Froude scale curved bridge to investigate the wave-in-deck loads. However, the effects of deck inclinations on the wave forces were not studied.

* Material reprinted from Xu and Cai (2014) with permission from Journal of Performance of Constructed Facilities on the behalf of American Society of Civil Engineers (ASCE).
Bricker et al. (2012) evaluated the possible failure mechanisms of many bridges due to the 2011 Great East Japan Earthquake by field surveys and numerical methods. A concrete girder bridge, the Utatsu Bridge, with the bridge deck inclined up to $3^\circ$ (seaward side upward), was analyzed based on 2D numerical simulations and the effects of buoyancy, lift, drag, and moment were investigated with both broken and unbroken waves. As discussed in their study, several parameters contribute to the bridge failures, i.e., deck inclination, flow speed, trapped air, entrained sediment, and tsunami surge. It is concluded that the lift force, drag force, and overturning moment vary with different flow speeds, and the resulted overturning moment is much easier to cause bridge failures than the lift force alone. In addition, the bridge deck inclination angle plays a very important role for the bridge stability problems, where a deck inclined on the seaward side can cause a significant increase of lift forces compared with a level deck and the inclined deck tends to be unstable at lower flow speeds. For example, the inclined deck is considered as unstable when the water flow is as slow as 3 m/s, while the level deck is stable even up to 5 m/s for the surge case. However, in their study, only one deck inclination angle is considered under the specific condition that the bridge deck is fully submerged into the water. Therefore, more studies are needed for the understanding of the effects of the bridge deck inclinations under different bridge elevations.

![Image of failed bridge](image_url)

**Fig. 4.1 Failure of US 90 to I-10 Ramp Bridge over Mobile Bay**

As Riggs (2007) mentioned, wave forces due to tsunamis and hurricanes have similar requirements in terms of structural designs. In the current study, a solitary wave model is selected to represent the tsunami wave occurred frequently along the Western Pacific Ocean, the Eastern Pacific Ocean and the Indian Ocean (the 2004 Indian Ocean Tsunami, the 2011 Great East Japan Tsunami). One bridge model with the geometry similar to the Biloxi Bay Bridge, which was damaged during hurricane Katrina in 2005, is chosen to conduct numerical simulations. One still water level (SWL) is considered and seven structure elevations with different deck inclinations and four wave heights are chosen.
In the following part of the present study, the numerical method and the computational domain are illustrated first. Then, typical time histories of the horizontal force, uplift force, and moment are presented. In addition, descriptions and comparisons are made between the inclined and level decks for different bridge superstructure elevations considering various deck inclinations. Conclusions are presented finally.

4.2 Methodology

Compared with the time consuming and high cost laboratory experiments, numerical approaches are becoming more widely adopted in engineering communities. As for the problem of wave induced forces on coastal low-lying bridges, very few 3D simulations have been conducted until now since it typically takes months for a simulation. Bozorgnia et al. (2012) conducted 3D cases and 2D cases on the bridge-wave problem and compared the results with the experimental data reported by Bradner et al. (2011). The differences of the maximum vertical forces between the Test 1 (2D) and Test 5(3D) are only 11% for $H/d = 0.45$ ($H$ is the wave height and $d$ is the still water depth), 6% for $H/d = 0.36$, and even less for other 3 situations, indicating that 2D simulations may predict relatively reasonable results. Therefore, 2D simulations are conducted in the present study to save the computational cost.

Fig. 4.2 shows the schematic diagram for the computational domain of the 2D cases (200m in length x 15m in height), where line EF is the SWL (7.2m from the bottom bed) that separates the regions of the air and water at the initial condition. For the boundary conditions, AB is defined as pressure outlet boundary condition maintained as the atmospheric pressure (101,325 pascal), AC the velocity inlet, CD the no slip stationary wall condition, and BD the pressure outlet boundary condition.

The geometric parameters of the Biloxi Bay Bridge decks with some simplifications are also shown in Fig. 4.2. The width of the structure is 10.45 m, the girder height is 1.05 m, and the deck depth is 0.3 m. All the six girders, each with a width of 0.3 m, are simplified as rectangles and evenly distributed. While the bridge model has a deck inclination, all the girders will be adjusted according to the deck inclination without changing the total height of 1.05 m. The inclination angle of the bridge deck is defined positive when there is a super elevation at the sea-ward side of the bridge deck. Bridge models with the same geometry (level deck) were also studied by Xiao et al. (2010) and Huang and Xiao (2009).
Commercial software Fluent is used in the present study with a laminar flow model. The pressure-based solver (segregated) is chosen for the transient flow, the Pressure-Implicit with Splitting of Operators (PISO) scheme is utilized for the pressure-velocity coupling method, and the PRESTO! (PREssure STaggering Option) scheme is set for the pressure spatial discretization. Least squares cell based scheme is used for the gradient discretization, second order upwind for momentum advection terms, and geo-reconstruct for the volume fraction equations.

The accuracy of the solitary wave models were validated in our previous study by comparisons with the analytical results and the laboratory experiments of French (1969), accompanied with the mesh sensitivity studies (Xu et al. 2015). While the study by French (1969) was focused on a flat deck, one study by McPherson (2008) was found to consider solitary wave forces on a 1:20 scaled bridge deck, with a section of U.S. 90 Bridge as the prototype bridge. The study by McPherson (2008) was conducted in the Haynes Coastal Engineering Laboratory 3D shallow water wave basin at the Texas A&M University. The railing effects were considered with the perforated railings on the seaward side of the bridge deck. In order to eliminate 3 dimensional effects, dummy bridge sections were placed on both sides of the bridge model. Bridge pilings were used to support the dummy bridge sections to simulate in field scenarios.

In the current study, 2D simulations are considered for the further validation purpose with the bridge model geometry shown in Fig. 4.3. It should be noted that the perforated railings cannot be fully realized due to the limitation of the 2D model. To accommodate the accuracy of the bridge model, a railing height of 3cm is considered above the bridge deck with a 2cm clearance. The bridge model is fixed and the bottom of the girder is located 0.41 m above the bed. Four water depths, 0.39 m, 0.41 m, 0.48 m and 0.54 m, are considered. Only one wave height, 0.14 m, is used.

![Fig. 4.3 Schematic diagram for the bridge model adopted by McPherson (2008)](image)

The computational domain is 13 m in length x 0.9 m in height. Based on the mesh sensitivity studies in our previous study, the grid resolutions for this validation 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; \(dx=0.005\) 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. The total meshed cells are about 320,000 and the grid mesh in the computational domain is shown in Fig. 4.4. The simulation time is 6 s and the time step is set as \(dt=0.0025\) s.
Fig. 4.4 Grid mesh for the bridge model adopted by McPherson (2008)

Comparisons between the results by the current method and by McPherson (2008) are shown in Fig. 4.5 and Fig. 4.6. As shown in Fig. 4.5, when $d = 0.54$ m, a small difference between the maximum horizontal forces is found. The main reason may be that the simplified 2D railing has shortcomings when compared with the 3D perforated railings in the laboratory experiments. It also should be noted in Fig. 4.6 that differences are found between the maximum vertical forces when $d = 0.39$ m and 0.41 m. We attribute this to the effects of the entrapped air. In 2D simulations, the entrapped air cannot escape in a timely manner. In general, good agreements are witnessed, indicating that the current method can be further employed in the bridge deck-wave interaction problem.

Fig. 4.5 Comparisons of the horizontal forces

Fig. 4.7 shows an example of the model grid mesh adopted in the current study for inclined sections. Structured mesh method is employed. The grid resolutions are: $\Delta x = 0.1$ m, $\Delta y = 0.1$ m for the zone nearby the bridge model; $\Delta x = 0.2$ m, $\Delta y = 0.1$ m for the near water surface zone at the far field from the bridge model; $\Delta x = 0.2$ m, $\Delta y = 0.2$ m for the deep water zone, and $\Delta x = 0.2$ m, $\Delta y = 0.4$ m for the air zone at the far field from the bridge model.
Fig. 4.6 Comparisons of the vertical forces

(a) Grid mesh in the computational domain

(b) Grid mesh nearby the bridge model

Fig. 4.7 One example of the grid mesh

4.3 Characteristics of Solitary Wave Forces on Biloxi Bay Bridge Decks with Inclinations

To investigate the effects of deck inclinations on the wave forces, different bridge elevations with a fixed SWL are introduced and analyzed by employing the numerical wave models as shown in Table 4.1. The submersion coefficient $C_s$ is the ratio of $S$, the
distance between the bottom of the superstructure to the SWL (negative if the structure is submerged in the water), to $H_b$, the height of the bridge structure. The momentum center is the moment center due to the vertical force and horizontal force, and it is located at the middle height of the deck. The bridge elevation is the elevation of the bottom of the superstructure. It is noted that for convenience, when calculating both the submersion coefficient and bridge elevation, the inclined bridge deck is levelled by taking the momentum center as the reference point and rotating center. Each case is named according to both the momentum center in the $y$ direction and the submersion coefficient; for example, E8.4/CS(0.444) stands for the case when the momentum center is 8.4m in the $y$ direction and the submersion coefficient is 0.444. Four wave heights were chosen as seen in Table 4.2, where $\varepsilon = H/d$, the ratio of wave height $H$ to the water depth $d$; $L_e = 2\pi d / \sqrt{3H/d}$, the effective wave length; $c$ is the wave speed; $t_0 = L_{min}/c$, and $L_{min}$ is defined as the minimum length to allow the wave crest to reach the inlet boundary after a certain time $t_0$ (Xu et al. 2015).

Table 4.1 Deck inclinations and other parameters

<table>
<thead>
<tr>
<th>Cases</th>
<th>Water depth (m)</th>
<th>Deck inclination</th>
<th>Bridge elevation (m)</th>
<th>$C_s = S/H_b$</th>
<th>Momentum Center</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$x$ (m) $y$ (m)</td>
</tr>
<tr>
<td>E9.0/CS(0.444)</td>
<td>7.2</td>
<td>-10*</td>
<td>7.8</td>
<td>0.444</td>
<td>9.0</td>
</tr>
<tr>
<td>E8.7/CS(0.222)</td>
<td></td>
<td>-8*</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E8.4/CS(0)</td>
<td></td>
<td>-6</td>
<td>7.8</td>
<td>0.222</td>
<td>8.7</td>
</tr>
<tr>
<td>E8.1/CS(-0.222)</td>
<td></td>
<td>-4</td>
<td>7.5</td>
<td>0</td>
<td>8.4</td>
</tr>
<tr>
<td>E7.8/CS(-0.444)</td>
<td></td>
<td>-2</td>
<td>7.2</td>
<td>-0.222</td>
<td>8.1</td>
</tr>
<tr>
<td>E7.5/CS(-0.667)</td>
<td></td>
<td>0</td>
<td>6.9</td>
<td>-0.444</td>
<td>7.8</td>
</tr>
<tr>
<td>E7.2/CS(-0.889)</td>
<td></td>
<td>2</td>
<td>6.6</td>
<td>-0.667</td>
<td>7.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>6.3</td>
<td>-0.889</td>
<td>7.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>6.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>8*</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>10*</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: * these four deck inclinations are only considered when the bridge elevation is 7.2m.

Table 4.2 Wave cases and related parameters

<table>
<thead>
<tr>
<th>$H$ (m)</th>
<th>$\varepsilon = H/d$</th>
<th>$L_e$ (m)</th>
<th>$c$ (m/s)</th>
<th>$t_0 = L_{min}/c$</th>
<th>calculation time $t$ (s)</th>
<th>$dt$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.20</td>
<td>0.306</td>
<td>47.25</td>
<td>9.57</td>
<td>5</td>
<td>15</td>
<td>0.005</td>
</tr>
<tr>
<td>1.80</td>
<td>0.25</td>
<td>52.23</td>
<td>9.38</td>
<td>6</td>
<td>16</td>
<td>0.005</td>
</tr>
<tr>
<td>1.40</td>
<td>0.194</td>
<td>59.23</td>
<td>9.17</td>
<td>7</td>
<td>18</td>
<td>0.005</td>
</tr>
<tr>
<td>1.00</td>
<td>0.139</td>
<td>70.08</td>
<td>8.96</td>
<td>8</td>
<td>21</td>
<td>0.005</td>
</tr>
</tbody>
</table>

4.3.1 Time Histories of Wave Forces

The wave forces on the bridge deck with two selected deck inclination angles under solitary waves are analyzed and discussed here for demonstration. All the forces on the bridge superstructure are recorded since the simulations begin. In the process of the bridge deck-wave interaction, the wave profile is disturbed when the wave impacts the
front of the bridge deck, and overtopping may occur, resulting in the changes of the pressure domain for each time step. The total force component (the horizontal force, the vertical force, and the moment) on the bridge superstructure is computed by integrating the pressure and viscous forces on each cell face along the walls of the bridge model after each time step the whole computational domain is solved. The reference pressure, i.e., the atmospheric pressure (101,325 pascal), is used for computing the pressure force acting on each wall zone of the bridge model. The buoyancy force and the green water force (water on deck) are included in the vertical force component. Fig. 4.8 shows an example of the bridge deck-wave interaction for a deck inclination of 6° with the solitary wave height 1.80 m and the bridge elevation 7.2 m, and Fig. 4.9 is for the deck inclination of −6°. The time-history wave forces of these two cases are shown in Fig. 4.10.

Fig. 4.8 Bridge deck-wave interaction for deck inclination 6° with the solitary wave height 1.80 m and the bridge elevation 7.2 m

Fig. 4.10 shows significant difference for the two different deck inclination angles in terms of the vertical force, horizontal force, and moment. For the deck inclination 6°, the peak vertical force and positive peak horizontal force is 145.947 kN and 22.182 kN, about 1.34 times and 1.64 times of the value with the deck inclination−6°, 109.209 kN and 13.498 kN, respectively. For the moment, the positive and negative peak moments with the deck inclination 6° are 108.535 kN*m and -7.858 kN*m, respectively, versus the values 63.639 kN*m and -108.596 kN*m, respectively, for the deck inclination−6°. The dominating moment changes its sign from positive (108.535 kN*m) to negative (-108.596 kN*m) value due to the change of inclination angles, to which much attention should be paid during the designing processes.
Fig. 4.9 Bridge deck-wave interaction for deck inclination $-6^\circ$ with the solitary wave height 1.80 m and the bridge elevation 7.2 m

Fig. 4.10 Time-history of wave forces for deck inclination $6^\circ$ and $-6^\circ$ with the wave height 1.80 m and the bridge elevation 7.2 m
4.3.2 Effects of the Deck Inclinations on Horizontal Forces

As shown in Fig. 4.10, the positive peak horizontal force with a positive deck inclination $6^\circ$ is larger than that with a negative one. More results of the effects of the deck inclinations on the peak horizontal forces are presented in detail in Fig. 4.11. In each plot of the figure, the left part shows the positive and negative peak horizontal forces of each case (from each time-history horizontal force curve). It is noticed that the horizontal forces with higher wave height are larger, especially for the positive horizontal forces and, generally speaking, the positive peak horizontal forces are larger than the negative peak horizontal forces. As a result, only the positive peak horizontal forces are further presented on the right part in each plot showing the ratios or the normalized values of $\frac{F_{\text{horizontal}}}{F_{h0}}$, where $F_{\text{horizontal}}$ is the positive peak horizontal force of each case and $F_{h0}$ is the corresponding positive peak horizontal force when the inclination angle is zero.

It can be observed that when the height of water depth plus the wave height is higher than the top of the seaward girder (cases of E7.2/CS(-0.889), E7.5/CS(-0.667), E7.8/CS(-0.444), and E8.1/CS(-0.222)), the horizontal force tends to increase as the bridge deck inclination increase from negative to positive. For a negative deck inclination $-6^\circ$, the $\frac{F_{\text{horizontal}}}{F_{h0}}$ ratios are around 0.78 to 0.93. However, for a positive deck inclination $6^\circ$ the ratios are around 1.4 to 1.7. The values vary with the wave heights. There is a drop from the general curve for the case of E8.1/CS(-0.222) with the wave height 1.00m and the deck inclination $4^\circ$. This may be due to that the height of water depth plus the wave height is lower than the top of the seaward girder ($7.2 + 1.0 < 8.1 + (10.45/2) \times \tan(4^\circ)$). This phenomena can be also observed in the case of E8.4/CS(0). When the height of water depth plus the wave height is lower than the top of the seaward girder, the wave front cannot surpass the whole bridge deck, and the current speed may be disturbed, resulting in a drop of the horizontal force ratios from the general trend. In general, for the other two cases (cases of E8.7/CS(0.222) and E9.0/CS(0.444)), the ratios with positive deck inclinations are larger than 1.0, and the ratios with positive deck inclination $6^\circ$ are round 1.05 to 1.45.

![Fig. 4.11 Horizontal forces considering bridge deck inclinations](image-url)
Fig. 4.11 (continued) Horizontal forces considering bridge deck inclinations
Fig. 4.11 (continued) Horizontal forces considering bridge deck inclinations

For the case of E8.4/CS(0), the general trends of the horizontal forces decrease, then increase as the bridge deck inclination increases. In this plot (Fig. 4.11(e)), it shows that the ratios with negative deck inclination $-10^\circ$ are larger than 1.0, around 1.1 to 1.2; when the deck inclination is $-6^\circ$, the ratios are smaller than 1. This may be due to that
more projected horizontal area will appear when the bridge deck inclines more. In addition, for the cases of E8.7/CS(0.222) and E9.0/CS(0.444), the general trends of the horizontal forces are not as clear as those of case E8.4/CS(0).

4.3.3 Effects of the Deck Inclinations on Vertical Forces

The effects of the deck inclinations on the peak vertical forces are presented in Fig. 4.12 (only the positive vertical forces are considered here). Similarly, the left part of each plot shows the peak vertical forces, with larger ones for the higher wave heights. The ratios or the normalized values of \( \frac{F_{vertical}}{F_{v0}} \) are shown in the right part of each plot, where \( F_{vertical} \) is the positive peak vertical force of each case, and \( F_{v0} \) is the corresponding positive peak vertical force when the inclination angle is zero.

For the cases of E7.2/CS(-0.889), E7.5/CS(-0.667), E7.8/CS(-0.444), and E8.1/CS(-0.222), the vertical forces tend to increase as the bridge deck inclination increases from negative to positive values. The vertical forces with a negative deck inclination \(-6^\circ\) are smaller than those when the inclination angle is zero, with ratios around 0.76 to 0.84, varying with different wave heights. The vertical forces with a positive deck inclination \(6^\circ\) are larger than those with a zero inclination, with ratios less than 1.12. The phenomena were also observed by Bricker and Nakayama (2014), where the lift force and moment acting on the deck are significantly weaker for the level deck than those for the inclined deck.

For the case of E8.4/CS(0), the general trends of the vertical forces for the four wave heights increase, then decrease as the bridge deck inclination increases. The ratios with a negative deck inclination \(-10^\circ\) range from 0.59 to 0.65. The ratios start to decrease when the bridge deck inclination is larger than \(2^\circ\). While only the ratio of the case with a wave height of 1.00m is less than 1.0 when the bridge deck inclination is \(6^\circ\), the ratios are less than 1.0 under three wave heights (1.0m, 1.4m and 1.8m) when the bridge deck inclination is \(10^\circ\).

![Fig. 4.12 Vertical forces considering bridge deck inclinations](image-url)
Fig. 4.12 (continued) Vertical forces considering bridge deck inclinations
Fig. 4.12 (continued)  Vertical forces considering bridge deck inclinations

While the general trend of case E8.7/CS(0.222) is similar to the case of E8.4/CS(0), the trend of the case E9.0/CS(0.444) is not clear. However, it can be concluded that when the bridge deck inclination is negative, the ratios are less than 1.0,
and are larger than 1.0 when the inclination is positive. This conclusion is based on the bridge deck inclinations ranging from $-6^\circ$ to $6^\circ$.

4.3.4 Effects of the Deck Inclinations on Moment Forces

The effects of the deck inclinations on the peak moments (both positive and negative moments) are presented in Fig. 4.13. Again, while the left part of each plot shows the positive and negative peak values, the right part shows the ratios or the normalized values, i.e., $\frac{M_{\text{moment}}}{|M_0|}$, where $M_{\text{moment}}$ and $M_0$ are similarly defined as the cases of the horizontal and vertical forces. It is noted that for some inclination angles, there is only either positive or negative moment, not both.

For the case of E7.2/CS(-0.889), higher waves do not always generate larger moments, and the trends of the ratios are not clear. For the case of E7.5/CS(-0.667), the ratios of the positive moments decrease, then increase with the increase of the bridge deck inclinations from negative to positive values. However, the ratios of the negative moments increase, then decrease with the increase of the bridge deck inclinations.

![Fig. 4.13 Moments considering bridge deck inclinations](image-url)
Moments considering bridge deck inclinations

For the other five cases, E7.8/CS(-0.444), E8.1/CS(-0.222), E8.4/CS(0), E8.7/CS(0.222) and E9.0/CS(0.444), the general trends of the positive and negative moments are much clear. It can be observed that larger moments, positive or negative, are accompanied with higher wave heights. For the cases of E8.7/CS(0.222) and E9.0/CS(0.444), the trends of both the positive and negative moments increase as the
bridge deck inclinations increase; and the ratios vary with the wave heights. For the cases of E8.4/CS(0), E8.7/CS(0.222) and E9.0/CS(0.444), the dominating moments with negative bridge deck inclinations are negative moments, and are positive moments with positive bridge deck inclinations.

![Graphs](f) E8.7/CS(0.222)

![Graphs](g) E9.0/CS(0.444)

Fig. 4.13 (continued) Moments considering bridge deck inclinations

The values of the moments depend on the corresponding horizontal forces and vertical forces. In the calculation process, all the horizontal and vertical forces are integrated by the pressure along the whole bridge model surface. While some local pressure changes may not affect the total horizontal or vertical forces, they influence the moments. As a result, the normalized ratios of the moments either increase or decrease with the increase of the bridge deck inclinations. In addition, both the bridge deck inclination and the wave heights play important roles on the moments.

### 4.4 Concluding Remarks

From this research concerning wave forces due to solitary waves on bridge decks with inclinations, conclusions can be drawn as follows:

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(1) Higher waves are accompanied with larger wave forces on the bridge decks, especially for the wave-induced horizontal forces and vertical forces. However, there are some exceptions for the moment forces when the bridge deck is fully submerged into the water. Generally speaking, the wave forces on the inclined bridge decks under solitary waves are affected by the following factors: wave heights, bridge deck inclinations, and the relative position between the wave peak (the height of water depth plus the wave height) and the top of the seaward girder. The wave heights and bridge deck inclinations play more significant roles than the other factors.

(2) Generally speaking, the increment of the normalized ratios of the horizontal forces for the bridge deck inclinations from $-6^\circ$ to $0^\circ$ is smaller than that for the bridge deck inclination from $0^\circ$ to $6^\circ$ when the wave peak is higher than the top of the seaward girder. The ratios generally increase with the increase of the bridge deck inclinations from $-6^\circ$ to $6^\circ$.

(3) The increment of the normalized ratios of the vertical forces for the bridge deck inclination from $-6^\circ$ to $0^\circ$ is larger than that for the bridge deck inclination from $0^\circ$ to $6^\circ$. The normalized ratios increase as the bridge deck inclinations increase, especially when the bridge deck is partially or fully submerged into the water.

(4) The ratios of the moments do not have consistent trends for the seven different bridge elevations studied in the present study. The dominating moments with negative bridge deck inclinations are negative moments, and are positive moments with positive bridge deck inclinations when the leveled bridge decks are above the SWL. However, the dominating moments with negative bridge deck inclinations are positive moments when the leveled bridge decks are under the SWL.

The limitations of the current study and future work are described as follows: (1) In the present study, 2D numerical simulations have been conducted. However, 3D models may provide more reliable results, but maybe much more computational expensive. (2) The bridge models employed in the present study are simplified without considering the railing and the diaphragm. Hence, more studies are needed to further investigate the wave forces due to solitary waves on coastal bridge decks with inclinations. (3) In this study, laminar flow is adopted. In the future work, effects of the turbulence need to be considered.

4.5 References


5.1 Introduction

Tsunamis, especially in the last decade, have devastated many coastal communities, including many low-lying coastal bridges (Sugimoto and Unjoh 2006; Shoji and Moriyama 2007; FHWA 2008; Akiyama et al. 2012; Kosa 2012; Maruyama et al. 2013). Post-disaster reports show that these coastal bridge decks under wave actions during these extreme natural disasters were subjected to huge wave loads that are acknowledged as the main reason for these bridges’ failures.

Similar to mitigating aerodynamic effects in long span bridges (Cai et al. 1999), there are a few commonly used practices for mitigating hydraulic forces, such as by changing a solid railing system into an open one or cutting slots or venting holes on a bridge deck to release the trapped air (Bozorgnia et al. 2010; Lao et al. 2010; Hayatdavoodi et al. 2014). Additional mitigation ideas may be learnt from earthquake engineering where base isolations, cable restraints, shear keys, and shape memory alloys are commonly used to adjust the interface stiffness between the superstructures and substructures to reduce the damage to structures. For example, when the restraining force reaches to some extent, the shear key will be sheared off as intended so that the substructures will be protected from damage. A good mitigation strategy would be to balance both the superstructure and substructure performance. While a weak connection/restraining system may not be good enough in protecting the superstructure, a too strong one would put too much force on the substructure that tends to be very expensive for repair. Therefore, before developing a good mitigation strategy, the dynamic analysis regarding the general lateral restraining stiffness (representing the substructures and interface connections such as restraining cables as discussed later) effects on the bridge deck-wave interaction needs to be fully understood.

Many studies for the solitary wave (representing the incident waves in tsunamis, Lin 2008) forces on coastal bridge decks were conducted in order to predict the wave forces on the rigidly supported bridge decks (rigid setups) (McPherson 2008; Lao et al. 2010; Seiffert et al. 2014; Hayatdavoodi et al. 2014). However, very few studies focused on the dynamic characteristics of the bridge deck-wave interaction problems (flexible setups), if any (Sugimoto and Unjoh 2006). There were some discussions that using flexible connections between the superstructure and substructures may reduce the interaction forces (Okeil and Cai 2008), which was based on the assumption that a larger displacement of the superstructure in the horizontal direction would dissipate more energy in the bridge deck-wave interaction process. However, experimental results by Bradner et al. (2011) did not support this assumption and a general consensus has not been reached. As a matter of fact, a dynamic analysis is essential for the design of coastal bridges, similar to the requirements for other nearshore and offshore structures (Sarpkaya and Isaacson 1981; Chakrabarti 2005).

The dynamic analysis for the bridge deck-wave interaction is recognized as an extremely complex problem not only because of the limitations for adequately describing
the bridge deck/superstructure system but also because of the difficulties of realizing the procedure for the bridge deck-wave interaction with sufficient accuracy, experimentally or numerically. A schematic diagram for the bridge deck-wave interaction under solitary waves is demonstrated in Fig. 5.1, where the interface between the superstructure and the substructure is not shown for clarity. It is noted that the lateral restraining stiffness of the superstructure represents the combined effects of the substructure stiffness and the interface stiffness between the substructure and superstructure. The substructure stiffness depends on the soil condition, the structural stiffness of the piers/piles, etc. The interface stiffness depends on the connections between the superstructure and substructure, such as bearing types, shear keys, restraining cables, and shape memory alloys (Song et al. 2006; Dong et al. 2011). In the present study, only the total lateral restraining stiffness of the bridge deck is concerned, without distinguishing the stiffness from the interface or substructure, similar to that adopted in the study by Bradner et al. (2011). While a very large restraining stiffness represents a case that the bridge deck is almost not moving under wave loading, a very small restraining stiffness (such as cases with very slender piers or weak connections between the super and substructures) will result in a large movement of the bridge deck, which, in turn, results in hydrodynamic interaction between the bridge deck and wave.

![Fig. 5.1 Schematic diagram for the bridge deck-wave interaction under solitary waves. H refers to the wave height; δ is the structural displacement for the bridge deck; and SWL refers to the still water level.](image-url)

Numerical modeling and simulation is undergoing fast development and is widely adopted in the efforts to study the effects of tsunami or hurricane impacts on coastal bridge decks (Huang and Xiao 2009; Xiao et al. 2010; Bozorgnia et al. 2010; Jin and Meng 2011; Bozorgnia et al. 2012; Bricker et al. 2012; Hayatdavoodi et al. 2014). The advantages of numerical simulations are that full scale models can be easily realized; model geometries and positions can be adjusted conveniently; and experimental cost and time can be saved. In order to achieve an appropriate balance between the computational cost, model sophistication, and physical realities, 2D numerical simulations that are
usually used in the literature for this topic are adopted here and the bridge deck is considered as a single degree of freedom system (SDOF) to accommodate the comparison with the experimental study by Bradner et al. (2011).

The objective of the present study is to investigate the lateral restraining stiffness effect on the bridge deck-wave interaction under solitary waves in order to obtain a good understanding of the bridge deck performance with different flexible setups. To this end, a CFD based numerical methodology is first proposed with a mass-spring-damper system implemented in a commercial computer program (Fluent, Education Version, V15.0). Then, the methodology is verified with experimental measurements in the literature. Finally, through a parametric study, the general dynamic characteristics of the structure vibration and the wave forces in the bridge deck-wave interaction are documented. The numerical results illustrate that increasing the lateral restraining stiffness in the transverse/horizontal direction results in larger horizontal forces on the interface between the bridge deck/superstructure and the substructure and larger dynamic amplification factors for the horizontal forces on the bridge deck. Therefore, rigidifying the superstructure by increasing the lateral restraining stiffness is generally beneficial to reducing the hydrodynamic wave forces.

5.2 Numerical Methodology and Experimental Verification

5.2.1 Wave Generation and Verification with Analytical Results

To simulate the turbulent characters for the in-coming waves and those generated from the bridge deck-wave interaction, the shear stress transport (SST) k-ω model is used as the turbulence closure for the RANS equations. This turbulent model has its advantages over the k-ε model, one of the most common turbulence models, 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. The equations for the SST k-ω model are given as follows:

\[
\frac{\partial}{\partial t} (\rho k) + \frac{\partial}{\partial x_j} (\rho k u_j) = \frac{\partial}{\partial x_j} \left( \Gamma_k \frac{\partial k}{\partial x_j} \right) + \bar{G}_k - Y_k + S_k \quad (5.1a)
\]

\[
\frac{\partial}{\partial t} (\rho \omega) + \frac{\partial}{\partial x_j} (\rho \omega u_j) = \frac{\partial}{\partial x_j} \left( \Gamma_\omega \frac{\partial \omega}{\partial x_j} \right) + G_\omega - Y_\omega + D_\omega + S_\omega \quad (5.1b)
\]

where \(\Gamma_k\) and \(\Gamma_\omega\) are the effective diffusivity of \(k\) and \(\omega\); \(\bar{G}_k\) represents the generation of turbulence kinetic energy due to the mean velocity gradients, calculated from \(\bar{G}_k\); \(G_\omega\) is the generation of \(\omega\); \(Y_k\) and \(Y_\omega\) are the dissipation of \(k\) and \(\omega\), respectively; \(D_\omega\) is the cross-diffusion term; and \(S_k\) and \(S_\omega\) are user-defined source terms.

The free surface profile \(\eta\), water pressure \(p\), and water particle velocities \(u\) and \(v\) for the 2nd-order solitary waves (Sarpkaya and Isaacson 1981) are expressed as follows:

\[
\frac{\eta}{d} = \varepsilon \text{sech}^2 q - \frac{3}{4} \varepsilon^2 \text{sech}^2 q \text{tanh}^2 q \quad (5.2a)
\]

\[
\frac{p}{\rho gd} = \frac{\eta}{d} + 1 - \frac{s}{d} - \frac{3}{4} \varepsilon^2 \text{sech}^2 q \left( \frac{s}{d} \right)^2 - 1 \left( 2 - 3 \text{sech}^2 q \right) \quad (5.2b)
\]

\[
\frac{u}{\sqrt{gd}} = \varepsilon \text{sech}^2 q + \varepsilon^2 \text{sech}^2 q \left\{ \frac{1}{4} - \text{sech}^2 q - \frac{3}{4} \left( \frac{s}{d} \right)^2 \left( 2 - 3 \text{sech}^2 q \right) \right\} \quad (5.2c)
\]
\[ \frac{v}{\sqrt{gd}} = \varepsilon \sqrt{3\varepsilon} \left( \frac{s}{d} \right) \text{sech}^2 q \tanh q \left\{ 1 - \varepsilon \left[ \frac{3}{8} + 2\text{sech}^2 q + \frac{1}{2} \left( \frac{s}{d} \right)^2 (1 - 3\text{sech}^2 q) \right] \right\} \]  

(5.2d)

where \( \varepsilon = \frac{H}{d} \), \( q = \frac{\sqrt{3\varepsilon}}{2d} \left( 1 - \frac{5}{8} \varepsilon \right) (x - ct) \), \( s = y + d \), \( d \) is the still water depth, \( H \) is the wave height, \( g \) is the gravitational acceleration, and \( y \) is the distance from the SWL and is negative if it is in the same direction with the gravitational acceleration. The wave celerity \( c \) is calculated as:

\[ \frac{c}{\sqrt{gd}} = 1 + \frac{1}{2} \varepsilon - \frac{3}{20} \varepsilon^2 \]  

(5.3)

It is calculated from Eq. (5.2a) that the solitary wave crest is located at \( x = 0 \) when \( t = 0 \) s, namely, the wave crest is just at the inlet boundary. To more appropriately simulate the wave profile, the incident solitary wave should be shifted leftward by replacing \( t \) with \( t - t_0 \), where \( t_0 = L_{\text{min}}/c \) and \( L_{\text{min}} \) is defined as the minimum length to allow the wave crest to reach the inlet boundary after a certain time. In this way the water surface will increase gradually at the inlet boundary to ensure that a fully developed wave profile will be generated and propagate from the inlet to the location of the structure model. \( L_{\text{min}} \) should be greater than the effective wave length \( L_e \), where \( L_e = 2\pi d / \sqrt{\frac{3H}{d}} \).

This method was adopted from Dong and Zhan (2009).

For this simulation problem, water is assumed as an incompressible, viscous fluid and the Volume of Fluid (VOF) method (Hirt and Nichols 1981) is employed to prescribe the dynamic free surface. For the setups of the SST \( k-\omega \) model in Fluent, the pressure-based solver (segregated) is chosen for the transient flow, the Pressure-Implicit with Splitting of Operators (PISO) scheme (FHWA 2009; Bricker et al. 2012) is utilized for the pressure-velocity coupling method, and the PRESTO (PREssure STaggering Option) scheme is set for the pressure spatial discretization. The turbulence damping is turned on and the damping factor is 50. For the velocity inlet boundary, the turbulent intensity is 2% and turbulent viscosity ratio is 10%. For the top and outlet of the calculation domain (see Fig. 5.2), the backflow turbulent intensity and the backflow turbulent viscosity ratio are assumed to be the same as that set for the velocity inlet boundary. Least squares cell based scheme is used for the gradient discretization, second order upwind for momentum advection terms, and Geo-Reconstruct for the volume fraction equations. Second order upwind is also used for the spatial discretization of the turbulent kinetic energy and the specific dissipation rate.

![Fig. 5.2 Schematic diagram for the computational domain and bridge deck model](image-url)
The numerical calculation domain and the boundary conditions are illustrated in Fig. 5.2 and discussed here since they are used for all the simulations in the present study. The line EF is the SWL, separating the regions of the air phase and water phase at the initial point; 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 equations of $\eta$ (5.2a), $u$ (5.2c), and $v$ (5.2d) are compiled into Fluent synchronously as the free surface profile and the velocity inlet components in the x and y directions, respectively, by the User Defined Functions (UDF); the bottom line CD is modeled with a no slip stationary wall condition; and the line BD is also set as pressure outlet.

The geometry of a typical coastal bridge deck model is introduced here firstly for the convenience of discussion, and the numerical simulations employing this bridge model will be further discussed in the parametric study. This prototype bridge, 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 island communities (Hayatdavoodi et al. 2014). The width of the superstructure is 10.45 m, the girder height is 1.05 m, and the deck depth is 0.3 m. All the six girders, each with a width of 0.3 m, are simplified as rectangles and evenly distributed.

Based on the above discussions, the wave generation and verification with analytical results are conducted in a computational domain with a section of 200 m long and 13 m high. For the mesh sensitivity study, a value of 0.3 for $\varepsilon$ (the ratio of the wave height, 2.20 m, to the still water depth, 7.20 m) is chosen; different mesh resolutions, $dx = 0.05$ m and 0.025 m in the x direction and $dy = 0.05$ m and 0.025 m in the y direction are used, respectively; and the time steps of 0.001 s and 0.005 s are considered based on the requirements of the Courant Number. The obtained results show that there are no significant differences on the achieved wave profiles. Therefore, the final grid meshes used with the structured mesh method are $dx = 0.05$ m in the x direction and $dy = 0.05$ m in the y direction and the time step $dt = 0.005$ s is adopted. The wave parameters used for the verification with the analytical results are as follows: the water depth d is 7.20 m and the wave heights are 1.74 m, 2.20 m, and 2.60 m with the $\varepsilon$ values of 0.24, 0.30, and 0.36, respectively, and with the $t_0$ values of 7 s, 6 s, and 6 s, respectively. It is noted that this verified wave height 2.20 m will be further used in the parametric study. Fig. 5.3 shows the comparisons of the free surface profiles between the numerical results and the analytical results at two different simulation times (corresponding $t_0$ values subtracted), where the bridge model is located at around 95 m from the inlet boundary. This figure shows that the numerical wave profile agree well with the analytical one.

**5.2.2 Verification of the Wave Forces on a Bridge Deck**

The capability of the developed wave model to predict the wave forces is ensured by the verification of the wave forces on a bridge deck with girders and side railings conducted by McPherson (2008) in a wave basin. The bridge model with the Biloxi Bay Bridge as the prototype one was built with a 1:20 scale and tested in the Haynes Coastal Engineering Laboratory 3D shallow water wave basin at the Texas A&M University. Perforated railings were considered on the seaward side of the bridge model. Two dummy bridge sections, supported by the bridge pilings, were placed on each side in
order to eliminate the three dimensional bridge deck end (3D) effects and to simulate in field scenarios.

Fig. 5.3 Comparisons of the free surface profiles for solitary waves at two different simulation times. (a) $\epsilon = 0.24$; (b) $\epsilon = 0.30$; and (c) $\epsilon = 0.36$.

In this verification process, the bridge model geometry is shown in Fig. 5.4 for the 2D simulations, where the perforated railings cannot be fully realized owing to the limitation of the 2D computational domain. To accommodate the accuracy of the bridge model, a railing height of 3 cm is considered above the bridge deck with a 2 cm clearance. Only one elevation of the bridge model is considered with the bottom of the girders located 0.41 m above the bed. Four water depths, 0.39 m, 0.41 m, 0.48 m and 0.54 m, are considered. Only one wave height, 0.14 m, is used with the $\epsilon$ values falling in the range from 0.24 to 0.36 that are already verified with the analytical wave profile.

Fig. 5.4 Schematic diagram for the bridge model adopted by McPherson (2008)

To accurately capture the near-wall features (the velocity field and pressure field) and hence the predicted wave forces, the wall boundaries of the structure model should be paid great attention. Based on the log-law for the “law-of-the-wall” used for identifying the viscous layer, blending layer, and the fully turbulent layer, very fine
meshes should be adopted near these wall boundaries. To take full advantage of the SST k-ω model, the wall-coordinate (dimensionless) y+ should be less than 2, where y+ is used to calculate the height of the first grid cell along the walls of the structure model in the turbulent flow. While it is very difficult to satisfy this requirement, the height of the first grid should be in the logarithmic layer and valid results can still be guaranteed. In the literature, y+ is desirable to be set in a range from 11.6 to 300 in order to achieve an acceptable accuracy in bridge engineering (Bredberg 2000; Xiong et al. 2014). In the current study, the range of y+ is from 30 to 50 in order to ensure that reliable pressure field and velocity field around the near wall regions can be obtained and to avoid the extensive computation.

For this verification, the computational domain is 13 m in length and 0.9 m in height. The grid resolutions 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; 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. A structured mesh method is employed and the grid mesh in the computational domain is shown in Fig. 5.5. The simulation time is 6 s with the time step of dt=0.0025 s.

![Grid mesh for the bridge model adopted by McPherson (2008)](image)

The total force component along the specified force vector \( \vec{a} \) (horizontal force or vertical force) on the wall zones of the bridge deck model is computed by summing the dot product of the pressure and viscous forces on each face with the specified force vector. The terms in this summation represent the pressure and viscous force components in the direction of the vector \( \vec{a} \) as:

\[
F_a = \vec{a} \cdot \vec{F}_p + \vec{a} \cdot \vec{F}_v
\]  

(5.4)

where \( \vec{a} \) is the specified force vector, \( \vec{F}_p \) is the pressure force vector, and \( \vec{F}_v \) is the viscous force vector. A reference pressure, \( p_{ref} \), (the operating pressure, 101, 325 Pa) is used to normalize the cell pressure for computation of the pressure force to reduce the round-off error:

\[
\vec{F}_p = \sum_{i=1}^n (p - p_{ref}) \hat{A}\hat{n}
\]  

(5.5)

where \( n \) is the number of faces, \( A \) is the area of the face, and \( \hat{n} \) is the unit normal to the face. The associated force coefficients (\( cl \) and \( cd \), uplift and drag force coefficients, respectively) are computed for the selected wall zones and are recorded since the simulation begins in the monitor setups. The force coefficient is defined as the force
divided by $\frac{1}{2} \rho v^2 A$, where $\rho$, $v$ and $A$ are the referenced density, velocity, and area, respectively, and these values are set in the “Reference Values” in Fluent.

Comparisons between the numerical results and the experimental measurements by McPherson (2008) are shown in Figs. 5.6 and 5.7. Fig. 5.6 shows a small difference between the maximum horizontal forces when $d = 0.54$ m, which may be due to that the simplified 2D railing has shortcomings when compared with the 3D perforated railings in the laboratory experiments and this was expected. Similarly, Fig. 5.7 shows some small differences between the maximum vertical forces when $d = 0.39$ m and 0.41 m. This is probably due to the effects of the entrapped air since the entrapped air cannot escape in a timely manner in 2D simulations. In general, good agreements are obtained, indicating that the current wave simulation and force prediction procedure can be further employed in the bridge deck-wave interaction problem.

![Fig. 5.6 Comparisons of the horizontal forces](image1)

![Fig. 5.7 Comparisons of the vertical forces](image2)
### 5.2.3 Realization of the Mass-Spring-Damper System

The bridge model considering the lateral restraining stiffness can be simulated using a mass-spring-damper system. In order to accommodate the SDOF system, it is assumed that the bridge deck is kept as intact in the bridge deck-wave interaction process. However, exceptional conditions were witnessed that many bridges were displaced or washed away due to the wave actions with/without the storm surge, where the wave loads surpassed the structure capacities, especially the bearing/interface capacities. These conditions are not in the scope of the present study.

The schematic diagram for the mass-spring-damper system in the computational domain is shown in Fig. 5.8, where the bridge model can vibrate in the $x$/horizontal direction. In this system, $m$ is the unit length weight of the bridge deck, $k$ is the lateral restraining stiffness, and $c$ is the damping coefficient. The motion of the bridge model can be described as the following equation:

$$\ddot{x} + 2\xi \omega_0 \dot{x} + \omega_0^2 x = \frac{F(t)}{m}$$  \hspace{1cm} (5.6)

where $x$ is the instantaneous displacement of the bridge model in the $x$ direction, $\xi$ is the damping ratio, $\omega_0$ is the natural frequency of the bridge superstructure and $F(t)$ is the instantaneous horizontal force integrated from the hydraulic pressure along the bridge model surface. In the present numerical study, the Froude similarity criteria are automatically satisfied since a full scale bridge model is chosen here.

From the mass-spring-damper system, the following equations are used to calculate the corresponding lateral restraining stiffness and the damping coefficient based on known mass, vibration period, and damping ratio of the bridge structure.

$$k = m\omega_0^2$$  \hspace{1cm} (5.7)
\[
\omega_0 = \frac{2\pi}{T_s} \quad \text{(5.8)}
\]
\[
c = 2\xi \omega_0 m \quad \text{(5.9)}
\]
where \(T_s\) is the structural vibration period.

In the calculation process, while the mesh in the Fixed zones (zone 1 and zone 2) remains the same as its original mesh, the Remeshing zone (with the mass-spring-damper system incorporated as a rigid body) is dynamically updated using the layering mesh method. For the setups of the layering mesh method, the height based method is chosen with the split factor 0.4 and the collapse factor 0.2. For the realization of the mass-spring-damper system in the simulation of the bridge deck-wave interaction, a dynamic updating technique is developed and implemented in Fluent, as shown in Fig. 5.9 (Xu et al. 2009; Ou et al. 2009).

![Flow chart for the bridge deck-wave interaction using a mass-spring-damper system](image)

The procedure for the dynamic mesh updating technique in the simulation of the bridge deck-wave interaction is described as follows. Firstly, in the beginning of each time step, the velocity and the pressure fields can be obtained by the CFD calculation. The total force components (the horizontal force \(F(t)\), the vertical force, and the moment) on the bridge deck can then be obtained for the current time step and saved both to the internal memory and external data profiles. Subsequently, by substituting the obtained horizontal force \(F(t)\) into equation (5.6), the velocity of the bridge deck is predicted by using the Newmark-\(\beta\) method for the structural dynamic analysis. Finally, this velocity is attributed to the rigid body that will move to a new position in one time step and hence, the remeshing zone is updated correspondingly. Once the mesh is updated, the whole fluid domain is ready for the CFD calculation in the next time step. This loop will continue to the final time step.
During the dynamic mesh updating procedure, UDFs are compiled in the Fluent program to allow users to realize the data capture and data calculation in each time step, such as the structural dynamic analysis using the Newmark-β method. The macros of Compute_Force_And_Moment are employed to real-timely calculate the wave forces on the bridge deck and the macros of DEFINE_CG_MOTION are used to control the movement of the rigid body at the end of each time step.

5.2.4 Verification of the Mass-Spring-Damper System

To verify the capabilities of the mass-spring-damper system, the general observations for the wave forces were compared with those from a laboratory experiment (Bradner 2008; Schumacher et al. 2008a; Schumacher et al. 2008b; Bradner et al. 2011). In the experimental study, a 1:5 scaled bridge deck model was tested in a large wave flume at the O. H. Hinsdale Wave Research Laboratory at the Oregon State University. The wave flume is 104 m in length, 3.66 m in width, and 4.57 m in depth. The span length of the bridge deck model is 3.45 m (corresponding to the width of the wave flume) and the width is 1.94 m. Six scaled AASHTO type III girders are evenly distributed underneath the deck slab. This model was placed on two linear guide rails with each one supporting one end of the span. Both the rigid setup and flexible setup (including soft springs setup and medium springs setup) were considered in the experimental study. For the flexible setup, in order to determine the restraining stiffness, a finite element analysis was conducted by typifying several different elevations of the bridge deck. Then, the structural vibration period was obtained for the corresponding deck elevation and a suitable support stiffness was subsequently chosen (two springs installed between the tested bridge model and the supporting frames) to match this period value.

The flexible setup for the bridge deck-wave interaction in the experimental study can be realized using the proposed mass-spring-damper system. However, several differences should be noted in the verification procedure: (a) 2D numerical simulations were conducted in the current study which may not fully capture all the characteristics observed in the experimental study; (b) since the span length of the bridge deck model is 3.45 m which is only slightly smaller than the width of the wave flume, 3.66 m, the 3D end effects may play significant roles in the experimental study; (c) the friction force between the bridge deck model and the supporting guide rails was not desired but cannot be avoided in the experimental study. However, this force was neglected and taken as 0 in the numerical simulations; and (d) all the AASHTO type III girders were simplified as rectangles in the numerical simulations. While these differences were noticed, it was expected that reasonable predictions of the general observations would be obtained. Other than the above discussed differences, all other parameters considered in the verification are exactly the same as those used in the experimental study, as listed in Table 5.1. Due to the reason that Bradner et al. (2011) only presented one figure (Fig. 12 in Bradner et al. 2011) to illustrate the results of the flexible setup and they normalized the time histories of the wave forces without giving the actual wave force and the corresponding wave height information, a direct comparison of the wave forces between the current method and the experimental measurements is not possible.
Table 5.1 Parameters considered in the verification of the mass-spring-damper system with Bradner et al. (2011)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder height (m)</td>
<td>0.23</td>
</tr>
<tr>
<td>Girder spacing (m)</td>
<td>0.37</td>
</tr>
<tr>
<td>Deck thickness (m)</td>
<td>0.05</td>
</tr>
<tr>
<td>Overall height (m)</td>
<td>0.28</td>
</tr>
<tr>
<td>Span mass per unit length (kg)</td>
<td>562.3</td>
</tr>
<tr>
<td>Structural vibration period, $T_s$, (s)</td>
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</tr>
<tr>
<td>Lateral restraining stiffness, $k$, (N/m)</td>
<td>31318</td>
</tr>
<tr>
<td>Damping ratio, $\xi$</td>
<td>0</td>
</tr>
<tr>
<td>Damping coefficient, $c$, (N$\cdot$s$^2$/m)</td>
<td>0</td>
</tr>
<tr>
<td>Still water depth, $d$, (m)</td>
<td>1.89</td>
</tr>
<tr>
<td>Wave height, $H$, (m)*</td>
<td>0.50</td>
</tr>
<tr>
<td>Wave period, $T$, (s)</td>
<td>2.5</td>
</tr>
<tr>
<td>Wave length (m)</td>
<td>8.6</td>
</tr>
</tbody>
</table>

Note: * Only one wave height of 0.50 m is considered here for the verification.

Based on the wave steepness $H/gT^2$ and relative depth $d/gT^2$ (Sarpkaya and Isaacson 1981), Stokes 2nd order wave theory is proper for the wave height of 0.50 m and its analytical expressions for the water particle velocities $u$ and $v$, and the free surface profile $\eta$ are as follows (Lin 2008):

$$u = \frac{H}{2} \frac{gk \cosh k(d+y)}{\cosh kd} \cos(kx - \omega t) + \frac{3H^2 \omega k \cosh 2k(d+y)}{16 \sinh^4(kd)} \cos2(kx - \omega t)$$  \hspace{1cm} (5.10a)

$$v = \frac{H}{2} \frac{gk \sinh k(d+y)}{\cosh kd} \sin(kx - \omega t) + \frac{3H^2 \omega k \sinh 2k(d+y)}{16 \sinh^4(kd)} \sin2(kx - \omega t)$$  \hspace{1cm} (5.10b)

$$\eta = \frac{H}{2} \cos(kx - \omega t) + \frac{H^2 k \cosh(kd)}{16 \sinh^2(kd)} (2 + \cosh 2kd) \cos2(kx - \omega t)$$  \hspace{1cm} (5.10c)

where $k$ is the wave number, $\omega$ is the wave frequency, $t$ is the simulation time, and $x$ is the distance from the inlet boundary.

The numerical calculation domain is 40 m in length and 2.5 m in height. Using the wave properties listed in Table 5.1, the numerical wave profiles are obtained and compared with analytical results as shown in Fig. 10, demonstrating a good agreement with each other.

![Fig. 5.10 Comparisons of the wave profiles for Stokes 2nd order waves](image-url)
The comparisons of the general characteristics of the numerically predicted wave forces between the flexible setup and rigid setup are demonstrated in Fig. 5.11, where three notable characteristics are observed: (a) while much smaller negative horizontal forces are found for the rigid setup, significant negative horizontal forces are observed for the flexible setup (soft springs setup). This is due to the consideration of the inertia forces of the bridge deck and will be discussed later; (b) a phase lag can be observed between the positive peak horizontal forces of the rigid setup and the flexible setup (soft springs setup) and this is also due to the inertia forces that are taken account in; and (c) there is no significant difference on the positive peak vertical forces. These observations follow the same trends as those documented by Bradner et al. (2011), indicating that the proposed mass-spring-damper system has a good capability to capture the general dynamic characteristics of the bridge deck-wave interaction problems. This system may be further adopted for other near shore and offshore structures, such as elastically mounted cylinder in a flowing fluid domain (Xu et al. 2014).

![Fig. 5.11 Comparisons of the numerical wave forces between the flexible setup (soft springs setup) and rigid setup](image)

In summary, a close match between the generated solitary waves and the prescribed ones demonstrated above is a premise for a reliable prediction of the wave forces. In addition, it has demonstrated that a simple mass-spring-damper model can capture the general dynamic characteristics in the bridge deck-wave interaction process. Therefore, this methodology is used in the following parametric study to systematically investigate the lateral restraining stiffness effect on the bridge deck-wave interaction.
5.3 Parametric Study

In this parametric study, seven sets of lateral restraining stiffnesses corresponding to seven structural vibration periods and a damping coefficient $\xi$ of 0.05 are chosen, as shown in Table 5.2. These structural vibration periods represent a considerable range of cases for the bridge decks with and without mitigation countermeasures such as restraining cables. By adjusting the properties of these interface connections, more specific structural vibration periods can be obtained. In the simulations, the mass is primarily taken as 9716 kg per unit length according to the study by Xiao et al. (2010) \((154000 \text{ kg} / 15.85 \text{ m} = 9716 \text{ kg per unit length})\) and this provides a close estimation for the present bridge model.

<table>
<thead>
<tr>
<th>Cases</th>
<th>$T_s$ (s)</th>
<th>$m$ (kg)</th>
<th>$\xi$</th>
<th>$k$ (kN/m)</th>
<th>$c$ (N·s/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>k1534</td>
<td>0.5</td>
<td>9716</td>
<td>0.05</td>
<td>1534</td>
<td>12209</td>
</tr>
<tr>
<td>k383</td>
<td>1.0</td>
<td>9716</td>
<td>0.05</td>
<td>383</td>
<td>6105</td>
</tr>
<tr>
<td>k170</td>
<td>1.5</td>
<td>9716</td>
<td>0.05</td>
<td>170</td>
<td>4070</td>
</tr>
<tr>
<td>k96</td>
<td>2.0</td>
<td>9716</td>
<td>0.05</td>
<td>96</td>
<td>3052</td>
</tr>
<tr>
<td>k61</td>
<td>2.5</td>
<td>9716</td>
<td>0.05</td>
<td>61</td>
<td>2442</td>
</tr>
<tr>
<td>k43</td>
<td>3.0</td>
<td>9716</td>
<td>0.05</td>
<td>43</td>
<td>2035</td>
</tr>
<tr>
<td>k15</td>
<td>5.0</td>
<td>9716</td>
<td>0.05</td>
<td>15</td>
<td>1221</td>
</tr>
</tbody>
</table>

As discussed above, the computational domain is 200 m long and 13 m high, the water depth is 7.20 m, and the wave height is 2.20 m. Seven structure elevations and the corresponding coefficients are chosen and shown in Table 5.3, 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; $S$ is the distance from the bottom of the bridge superstructure to the SWL; and $H_b$ is the superstructure depth. The momentum center is the moment center due to the vertical force and 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 deck elevation (the value refers to the elevation of the bottom of the girder) and the submersion coefficient; for example, E7.20/CS(0) stands for the case when the bottom of the bridge model is 7.20 m from the sea bed and the corresponding coefficient of submersion depth $C_S$ is 0.

<table>
<thead>
<tr>
<th>Cases</th>
<th>Bridge elevation (Bottom of the girder) (m)</th>
<th>$S$ (m)</th>
<th>$C_S = S/H_b$</th>
<th>$C_S$</th>
<th>Momentum Center $x$ (m)</th>
<th>$y$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E8.55/CS(1.0)</td>
<td>8.55</td>
<td>1.35</td>
<td>1</td>
<td>95.225</td>
<td>9.75</td>
<td></td>
</tr>
<tr>
<td>E7.88/CS(0.5)</td>
<td>7.875</td>
<td>0.675</td>
<td>0.5</td>
<td>95.225</td>
<td>9.075</td>
<td></td>
</tr>
<tr>
<td>E7.20/CS(0)</td>
<td>7.20</td>
<td>0</td>
<td>0</td>
<td>95.225</td>
<td>8.40</td>
<td></td>
</tr>
<tr>
<td>E6.53/CS(-0.5)</td>
<td>6.525</td>
<td>-0.675</td>
<td>-0.5</td>
<td>95.225</td>
<td>7.725</td>
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<tr>
<td>E5.85/CS(-1.0)</td>
<td>5.85</td>
<td>-1.35</td>
<td>-1</td>
<td>95.225</td>
<td>7.05</td>
<td></td>
</tr>
<tr>
<td>E5.18/CS(-1.5)</td>
<td>5.175</td>
<td>-2.025</td>
<td>-1.5</td>
<td>95.225</td>
<td>6.375</td>
<td></td>
</tr>
<tr>
<td>E4.50/CS(-2.0)</td>
<td>4.50</td>
<td>-2.7</td>
<td>-2</td>
<td>95.225</td>
<td>5.70</td>
<td></td>
</tr>
<tr>
<td>E3.83/CS(-2.5)</td>
<td>3.825</td>
<td>-3.375</td>
<td>-2.5</td>
<td>95.225</td>
<td>5.025</td>
<td></td>
</tr>
<tr>
<td>E3.15/CS(-3.0)</td>
<td>3.15</td>
<td>-4.05</td>
<td>-3</td>
<td>95.225</td>
<td>4.35</td>
<td></td>
</tr>
</tbody>
</table>
5.3.1 Structural Vibration

One example of the bridge deck vibrations is shown in Fig. 5.12 for the case of E7.20/ CS(0) with different restraining stiffnesses. The bridge model is located at its original position (95.225m) when the simulation begins. It is noteworthy that the vibration amplitude of Case k15 (the most flexible one) is relatively larger than those of other cases with higher stiffnesses and the time for the bridge deck to reach to its peak displacement is different for all the seven cases. For example, while Case k15 reaches its peak value at 15.16 s; the time is 14.08 s for Case k1534 (the least flexible one). The bridge deck displacement of Case k15 would be completely damped out if longer simulation time was considered.

Comparisons of the structure displacement for all the seven structure elevation cases of k1534 and k43 are shown in Fig. 5.13. The peak values of the structure displacement of k1534 and k43 vary with different submersion coefficient. This relates to the characters of the horizontal forces with different submersion coefficients and will be discussed later.
The maximum (positive, landward) and minimum (negative, seaward) bridge deck displacements for all the studied cases are shown in Fig. 5.14, where the displacements show the same trends, namely, the less restraining stiffness, the larger the deck displacement. For example, the structure displacement is about 8 cm for the cases of k383, where the period of the structure vibration is 1.0 s. However, it is much smaller for cases of k1534 (less than 2 cm), which also can be observed in Fig. 5.13(a). With such a small vibration of 2 cm, the movement of the bridge superstructure probably has negligible effects on the flow fields in the bridge deck-wave interaction, resulting in a very close prediction of the wave forces to those for the rigid setup.

![Fig. 5.14 Maximum and minimum displacement of the structural vibrations](image)

5.3.2 Horizontal Forces without Considering Inertia Forces

Generally, wave forces on coastal structures refer to the net hydraulic forces without including the inertia forces of the bridge deck, i.e., the horizontal forces $F(t)$ in equation (5.6), and these forces are primarily used to design and to evaluate the bridge superstructure. Currently, only the horizontal wave forces that exclude the inertia forces are discussed in this section and those considering the inertia forces will be discussed subsequently. One example of the time-history horizontal forces of Case E6.53/CS(-0.5) is shown in Fig. 5.15. In this case, the time-history force curves differ from each other with different structure stiffnesses.

The positive and negative peak horizontal forces corresponding to the restraining stiffness of 1534 kN/m, the highest restraining stiffness considered in this study, are listed in Table 5.4. The positive and negative peak horizontal forces with considering inertia forces, and the positive peak vertical forces for this specific restraining stiffness are also listed in Table 5.4 for convenience purpose and will be further discussed later.

The trends of the positive and negative peak horizontal forces $F_h$ can be plotted using a normalized expression, $F_h/|F_{h_{k1534}}|$, as shown in Fig. 16. $|F_{h_{k1534}}|$ is the absolute value of the positive or negative horizontal force corresponding to the restraining stiffness of 1534 kN/m as listed in Table 4. It can be seen in Fig. 16(a) that as the stiffness increases, the positive horizontal forces increase and then decrease. However,
as for the negative horizontal forces as shown in Fig. 16(b), the trend is not as consistent for different deck elevations.

![Graph showing predicted horizontal forces for Case E6.53/CS(-0.5)](image)

Fig. 5.15 Predicted horizontal forces for Case E6.53/CS(-0.5)

<table>
<thead>
<tr>
<th>Force Type</th>
<th>PH without I</th>
<th>NH without I</th>
<th>PH with I</th>
<th>NH with I</th>
<th>Vertical</th>
</tr>
</thead>
<tbody>
<tr>
<td>E8.55/CS(1.0)</td>
<td>15.12</td>
<td>-2.4</td>
<td>16.31</td>
<td>-2.56</td>
<td>85.47</td>
</tr>
<tr>
<td>E7.88/CS(0.5)</td>
<td>18.95</td>
<td>-5.68</td>
<td>20.03</td>
<td>-6.27</td>
<td>100.8</td>
</tr>
<tr>
<td>E7.20/CS(0)</td>
<td>20.7</td>
<td>-11.45</td>
<td>21.31</td>
<td>-13.66</td>
<td>156.4</td>
</tr>
<tr>
<td>E6.53/CS(-0.5)</td>
<td>25.56</td>
<td>-17.6</td>
<td>25.75</td>
<td>-18.9</td>
<td>155.3</td>
</tr>
<tr>
<td>E5.85/CS(-1.0)</td>
<td>28.42</td>
<td>-20.2</td>
<td>28.8</td>
<td>-20.88</td>
<td>152.3</td>
</tr>
<tr>
<td>E5.18/CS(-1.5)</td>
<td>27.4</td>
<td>-14.8</td>
<td>27.85</td>
<td>-15.18</td>
<td>114.34</td>
</tr>
<tr>
<td>E4.50/CS(-2.0)</td>
<td>24.2</td>
<td>-12.46</td>
<td>24.5</td>
<td>-12.74</td>
<td>97.26</td>
</tr>
<tr>
<td>E3.83/CS(-2.5)</td>
<td>22.35</td>
<td>-11.75</td>
<td>22.65</td>
<td>-11.94</td>
<td>87.2</td>
</tr>
<tr>
<td>E3.15/CS(-3.0)</td>
<td>20.1</td>
<td>-10.18</td>
<td>20.3</td>
<td>-10.38</td>
<td>80.4</td>
</tr>
</tbody>
</table>

Note: *PH: Positive peak horizontal force; NH: Negative peak horizontal force; I: Inertia force.

![Graphs showing ratios of Fh/k1534](image)

(a) Positive horizontal forces (landward)  (b) Negative horizontal forces (seaward)

Fig. 5.16 The ratios of $\frac{F_h}{|F_{h,k1534}|}$ versus the restraining stiffness
5.3.3 Comparisons of the Horizontal Forces with and without Considering Inertia Forces

Apparently, when considering the flexible supports in the horizontal direction instead of the rigidly support conditions for the bridge deck, the inertia forces of the bridge deck will be prominent in the horizontal direction. The horizontal forces with considering the inertia forces are the forces transferred from the superstructure to the interface and they are used to design the bearing supporting and the substructures. The difference between the horizontal forces with and without considering inertia forces can be found in equation (5.6), i.e., \( F(t) \) and \( F(t) - m\ddot{x} \), where \( m\ddot{x} \) is the inertia forces of the bridge deck. If the bridge deck is fixed or with infinite lateral restraining stiffness, there should be no difference between \( F(t) \) and \( F(t) - m\ddot{x} \) since the inertia force is zero. In this section, this difference is investigated for finite lateral restraining stiffnesses, and the \( F_{h} + \)inertia force shown in the coming figures represents \( F(t) - m\ddot{x} \), i.e., the horizontal hydraulic forces plus the inertia forces of the bridge deck. Fig. 5.17 shows the time histories of the horizontal forces with and without considering the inertia forces.

![Fig. 5.17 Comparisons of the time histories of the horizontal forces with and without considering inertia forces for Case E5.18/CS(-1.5)](image)

It is worth noting that: (a) the difference between these two kinds of horizontal forces is significant when the restraining stiffness is relatively small (i.e., when the structure has a larger flexibility), for example, cases k15 and k43. With the increase of the restraining stiffness, the difference becomes small, for example, in case k383; (b) a phase difference between the horizontal forces with and without considering the inertia forces is observed, as can be more clearly found for cases of k15 and k43. The horizontal
forces with considering the inertia forces always fall behind for the first phase difference, while this trend does not hold on for the later phase differences; and (c) the structural vibration period may dominate the trailer part of the horizontal force curves and this phenomenon is much clear when the inertia force is considered. Taking the Case of k383 for example, the structural vibration period for this specific case is 1.0 s, and the trailer part of the horizontal force curve seems also having a vibration period of around 1.0 s.

Typical comparisons of the positive and negative peak horizontal forces with and without considering inertia forces for Case E4.50/CS(-2.0) are plotted in Fig. 5.18 (a) and Fig. 5.18(b), respectively. With the increase of the restraining stiffness, these two plotted lines tend to be closer with smaller difference. Generally speaking, the absolute peak horizontal forces with considering inertia forces are larger than those without considering inertia forces. All the other six different structure elevations have the similar characteristics.

Similar to the horizontal forces without considering inertia forces, a normalized expression, \( \frac{(F_h + \text{Inertia force})}{|F_{hk1534} + \text{Inertia force}|} \), is also employed to describe the trends of the positive and negative peak horizontal forces with considering the inertia forces, as shown in Fig. 5.19. The values of the forces, \( F_{hk1534} + \text{Inertia force} \), can be found in Table 5.4. Compared with Fig. 5.16, much difference can be found between the ratios of \( F_h \)\( F_{hk1534} \) and the ratios of \( (F_h + \text{Inertia force}) \)/\( F_{hk1534} + \text{Inertia force} \). For the positive forces shown in Fig. 5.19(a), the ratios of \( (F_h + \text{Inertia force}) \)/\( F_{hk1534} + \text{Inertia force} \) of all the seven structure elevations generally increase and then decrease with the increase of the restraining stiffness. As expected, \( (F_h + \text{Inertia force}) \)/\( F_{hk1534} + \text{Inertia force} \) approaches to 1 when the lateral restraining stiffness approaches to \( k_{1534} \). For the negative forces shown in Fig. 5.19(b), for all the seven structure elevations, the ratios of \( (F_h + \text{Inertia force}) \)/\( F_{hk1534} + \text{Inertia force} \) tend to decrease, increase and then level off with the increase of the restraining stiffness.

From above discussion one can conclude that the time histories of the horizontal forces with and without considering inertia forces differ from each other, while the
difference becomes larger with the increase of structure flexibility. For example, if the bridge structure is almost rigid and the corresponding structural vibration period is small, say less than 1.0 s in this study (cases of k383 and k1534), the difference is very small and can be neglected. From the viewpoint of the horizontal forces with and without considering the inertia forces, increasing the structural flexibilities in the horizontal direction will result in larger wave forces on the interfaces between the super and substructures. This, in turn, makes the interfaces more vulnerable and hence transfers larger forces to the substructure.

![Graph](image)

(a) Positive forces (landward)  
(b) Negative ones (seaward)

Fig. 5.19 The ratios of $\frac{F_{h+\text{Inertia force}}}{F_{hk1534+\text{Inertia force}}}$ versus the restraining stiffness

### 5.3.4 Vertical Forces

One example of the time histories of the vertical forces for Case E5.85/CS(-1.0) is shown in Fig. 5.20. It can be observe that both the ascending portion of the curves and the positive peak vertical forces almost coincide with each other for different restraining stiffnesses. In the following discussions, only the positive peak vertical forces are concerned because the negative peak vertical forces can be neglected when compared with the positive peak vertical forces.

![Graph](image)

Fig. 5.20 Vertical forces with different restraining stiffnesses for Case E5.85/CS(-1.0)
Similar to the horizontal forces, the results of the vertical forces after the normalization are plotted in Fig. 5.21. It is noted that when the restraining stiffness is 13 kN/m, the ratio of \( \frac{F_v}{|F_v|_{k1534}} \) falls in the range from 0.96 to 1.09 and it get close to 1.0 with the increase of the restraining stiffness, indicating that increasing the structural flexibilities do not have much effects on the vertical forces during the bridge deck-wave interaction.

![Fig. 5.21 The ratios of \( \frac{F_v}{|F_v|_{k1534}} \) versus the restraining stiffness](image)

### 5.3.5 Coupling Behavior between Horizontal Forces and Structural Vibrations

As discussed in Fig. 5.13, the peak values of the structure position (i.e., displacement) of cases k1534 and k43 vary with different submersion coefficients, which are related to the characters of the horizontal forces. Based on the plotted results of the time histories for the horizontal force and the corresponding structural vibration for Case E5.18/CS(-1.5), as shown in Fig. 5.22, their relations are demonstrated in detail. For k15 (Fig. 5.22 (a) and (b)), the curves of the time-history horizontal force and the structural vibration almost share the same developing pattern that there is only one crest and one trough (neglecting the amplitude of the curves), indicating a coupling behavior between them. This behavior is also observed for the curves of k61 (Fig. 5.22 (c) and (d)). For other cases with higher restraining stiffnesses when the bridge deck is submerged (not shown here), similar trends are observed. The characters (a, b, c, d, e, and f) shown in Fig. 5.22 (c) and (d) represent six snapshots during the process of the bridge deck-wave interaction for k61, and the snapshots are illustrated in Fig. 5.23 and Fig. 5.24.

For this specific case of k61 (Fig. 5.22 (c) and (d), and Figs. 5.23 and 5.24), the deck displacement based on the original moment center (95.225 m) increases as the horizontal force increases (snapshot (b)). For the snapshot (c), the horizontal force almost reaches the positive peak horizontal force when the wave crest reaches the front of the bridge deck. Then the wave forces decreases from the positive peak horizontal force as the structure displacement decreases from the maximum displacement (snapshot (d)).
Fig. 5.22 Examples of the bridge deck-wave interaction for Case E5.18/CS(-1.5)

Fig. 5.23 Snapshots in the bridge deck-wave interaction for k61 of the Case E5.18/CS(-1.5). (a) 0s; (b) 12.5 s; (c) 14 s; (d) 15.5 s; (e) 17 s; (f) 18.5 s.
It should be noted that the structural vibration period is 2.5 s for k61; however, the response of the horizontal force and the structural vibration do not strictly show a period of 2.5 s as shown in Fig. 5.22 (c) and (d) (one period can be measured from snapshot (e) to the next trough). The reason may be that the structural vibration in the bridge deck-wave interaction, especially when the structure is fully submerged, is influenced by the resistance force due to the water, including the trapped water between the girders. The water induced resistance force in the structural vibration may play an important role under this condition.

As for k43 of the Case E7.20/CS(0) (Figs. 5.25 and 5.26), the bottom of the superstructure is just at the SWL. While the structural vibration period is 3 s, only the structure movement shows a vibration period of 3 s (it can be measured from snapshot (e) to the next trough).
The reason may be that the horizontal force comes to zero soon when the wave crest leaves far away, followed by the free vibration of the bridge deck where the structural vibration period can dominate the following vibrations. This is different from the observations when the bridge deck is fully submerged into the water. In addition, turbulence effects are much fierce as observed in the bridge deck-wave interaction, especially for snapshots (e) and (f).

![Fig. 5.26 Snapshots in the bridge deck-wave interaction for k43 of the Case E7.20/CS(0). (a) 0 s; (b) 12.5 s; (c) 14 s; (d) 15.5 s; (e) 17 s; (f) 18.5 s.](image)

### 5.3.6 Dynamic Amplification Factor

In the static structure analysis, the structure displacement is calculated as \( \delta = \frac{F}{k'} \), where \( F \) is the force applied in the direction of the structure displacement. For the bridge deck-wave interaction problems, the dynamic amplification factor is calculated as follows:

\[
c_{\text{amp}} = \frac{k'\delta}{F_h}
\]  

The obtained amplification factors are plotted in Fig. 5.27, where all the studied deck elevations follow the general trend that the values of the amplification factor, \( c_{\text{amp}} \), decrease and approach to 1.0 as the restraining stiffness increases. However, the negative forces, as shown in Fig. 5.27 (b), have more scattered data. For submerged cases (the submersion coefficient ranges from -0.5 to -3.0) with the restraining stiffness 15 kN/m, the value of \( c_{\text{amp}} \) ranges from 1.33 to 1.38 and from 1.47 to 1.65 for positive and negative horizontal forces, respectively. Much larger values of \( c_{\text{amp}} \) are noted for other deck elevation cases (the submersion coefficient ranges from 0 to 1.0), and this may be due to the less contact time between the water and the bridge deck in the bridge deck-wave interaction process.
Based on the above analysis, it is clearly identified that the dynamic amplification factor approaches to 1.0 when the lateral restraining stiffness approaches to infinite. In other words, increasing the structural flexibilities with smaller lateral restraining stiffnesses leads to larger dynamic amplification factors, exhibiting an adverse effect on the horizontal forces on the bridge deck. This observation would be very useful for other near shore and offshore structures and this methodology can be further adopted to study the dynamic characteristics of these structures by taking account in the horizontal, vertical, and rotational restraining stiffnesses (French 1969; Lai 1986; Lai and Lee 1989; Kaplan et al. 1995; McConnell et al. 2004; Yeh 2007). Further studies for the dynamic characteristics of the bridge deck-wave interaction under periodical waves are needed in order to propose potential suggestions to the AASHTO code (2008) since the experimental setup used to build up this code is actually a rigid setup.

5.4 Conclusions and Remarks

In the present study, a numerical methodology using a dynamic mesh updating technique for the mass-spring-damper system is developed. General observations of the
restraining stiffness effect on the bridge deck-wave interaction are obtained through a parametric study, where seven different deck elevations and seven different lateral restraining stiffnesses are involved, representing a large variety cases for the bridge decks. It is proven that this methodology is a valid tool to successfully predict and discuss the bridge deck-wave interaction topics and it may be adopted for other near shore and offshore structures when the dynamic effect is expected to be significant.

Several remarkable observations are noticed in the parametric study: (a) the positive displacement (landward) and the negative displacement (seaward) are numerically shown to have the same trends, i.e., the more flexible the structure, the larger the structure displacement, as expected; (b) significant difference for the horizontal forces with and without considering inertia forces is found when the structure has a large flexibility. However, when the bridge structure is more rigid and the corresponding structural vibration period is less than 1.0 s, the difference is very small; (c) while increasing the structural flexibilities does not necessarily result in larger vertical forces, its adverse effect on increasing the dynamic amplification factor is identified; and (d) the time histories of the horizontal force and the structural vibration have the same trend, indicating that the horizontal force is coupled and interacted with the structural vibration.

5.5 References


Lessons Learned from the 2011 Great East Japan Earthquake, March 1-4, 2012, Tokyo, Japan.


CHAPTER 6. INVESTIGATING WAVE FORCES ON COASTAL TWIN BRIDGE DECKS UNDER SOLITARY WAVES

6.1 Introduction

Tsunamis and hurricane induced high storm surges and waves have caused devastating impact on coastal communities. One of the most disastrous tsunami, the 2004 Indian Ocean Tsunami, caused more than 225,000 lives and destroyed entire cities and communities (FHWA 2008). The 2004 Hurricane Ivan, the 2005 Hurricane Katrina, and the 2011 Great East Japan Tsunami also caused thousands of casualties and catastrophes to coastal communities and infrastructures, including many coastal bridges (Graumann et al. 2005; USAID 2005; Shoji and Moriyama 2007; Yeh et al. 2007; Ghobarah et al. 2006; Bricker et al. 2012).

In these natural disasters, evacuations are essentially necessary to minimize the loss of lives and properties in these coastal communities, while they are often constrained by the transportation capacity of the nearby coastal low-lying bridges that are serving these communities. These bridges are witnessed to be very vulnerable to tsunamis or hurricanes (Douglass et al. 2006; Robertson et al. 2007; Okeil and Cai 2008; Chen et al. 2009; Akiyama et al. 2012; Kosa 2012). Bridges with twin decks (seaward deck and landward deck), normally seen in coastal areas, suffered different damages for each deck. For example, the I-10 Bridge over the Escambia Bay, Florida, experienced a great loss in the 2004 Hurricane Ivan. For its eastbound (seaward) bridge, 51 spans were completely removed from the substructure, 33 spans were displaced from their initial positions, and 25 bents were affected due to the damage of the superstructures. In comparison, the corresponding three numbers are 12, 19 and 7, respectively, in the westbound (landward) bridge (Sheppard and Marin 2009). As such, it is generally acknowledged that the seaward bridge bears more damage than the landward bridge and the bridge decks in the seaward bound are more vulnerable than those in the landward bound. Based on this observation, many previous experimental and numerical studies are conducted mainly focusing on the wave forces on one specific single bridge deck (Denson 1980; Sugimoto and Unjoh 2006; Schumacher et al. 2008; AASHTO 2008; McPherson 2008; Cuomo et al. 2009; Shoji et al. 2011; Xiao et al. 2010; Bozorgnia et al. 2010; Bozorgnia and Lee 2012; Jin and Meng 2011; Hayatdavoodi et al. 2014). However, since twin bridges are very common practices in coastal areas, it is highly necessary to understand the interaction characteristics of the wave forces on the coastal twin bridge decks in the tsunamis or hurricanes.

On the one hand, a landward deck can be considered as a nearby structure of the seaward deck. As observed by Bricker and Nakayama (2014), the presence of a seawall (the nearby structure) downstream near the bridge deck increases the likelihood of the bridge deck failure. Much larger flow forces on the bridge deck were recorded as the gap between the bridge deck and the seawall becomes closer. While the landward deck may not be exactly the same as the nearby seawall, it may have an influence in the wave forces on the seaward deck and this interference effect needs to be quantified. On the other hand, though it is well known that the wave forces on the landward deck are smaller than those on the seaward deck, it is unknown that how much wave forces will be exerted on the landward deck quantitatively when compared with the case of one single bridge
deck studied by many previous studies (AASHTO 2008; McPherson 2008; Xiao et al. 2010; Bozorgnia et al. 2010; Bozorgnia and Lee 2012; Jin and Meng 2011; Hayatdavoodi et al. 2014). However, until now, a comprehensive analysis of the characteristics of the wave forces on the twin bridge decks is very rare, if any, which motivates the current study.

In the present study, a solitary wave model based on the 2nd order wave theory is selected to investigate the wave forces on the twin bridge decks, mainly representing the incident waves in tsunamis that occurred frequently along the Western Pacific Ocean, the Eastern Pacific Ocean and the Indian Ocean (such as the 2004 Indian Ocean Tsunami and the 2011 Great East Japan Tsunami). The laminar flow model, the Shear Stress Transport (SST) $k-\omega$ model and the computational domain are illustrated firstly, accompanied with the wave generation and verification process. One typical bridge model which is normally seen connecting islands in coastal areas is chosen in the following parametric study. Then, the characteristics of the wave forces on the twin bridge decks with a fixed deck gap and variable deck gaps considering different still water levels (SWLs) and various submersion coefficients are studied. In addition, normalized factors based on the wave forces on the seaward deck are given accordingly. Meanwhile, the hydrodynamic inference effects between the twin bridge decks are examined based on the wave forces on the single bridge deck. Moreover, the effects of the deck vibration on the wave forces are also investigated to represent more realistic scenarios in field.

6.2 Methodology

Due to the high cost of laboratory experiments, numerical approaches are becoming more widely adopted in the engineering communities. As for the numerical simulations of wave induced forces on coastal low-lying bridges, two dimensional (2D) models are usually adopted in the literature and the obtained results are believed to be with good accuracy. Therefore, in the present study, 2D models are applied for all the numerical simulations through a commercial software Fluent (V14.5, Academic Version).

6.2.1 Governing Equations

For the laminar flow simulations, water is assumed as an incompressible, viscous fluid. The fluid motion is described based on the Navier-Stokes equations as:

$$\rho \left( \frac{\partial u}{\partial x} + \frac{\partial u}{\partial y} \right) = S_m$$  \hspace{1cm} (6.1a)

$$\rho \left( \frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} \right) = -\frac{\partial p}{\partial x} + \mu \left( \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) + S_x$$  \hspace{1cm} (6.1b)

$$\rho \left( \frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} \right) = -\frac{\partial p}{\partial y} - \rho g + \mu \left( \frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) + S_y$$  \hspace{1cm} (6.1c)

where $\rho$ is the mass density, $u$ and $v$ are the velocity components, $p$ is the pressure, $\mu$ is the viscosity, $g$ is the gravitational acceleration, $S_m$ is the mass source, and $S_x$ and $S_y$ are the momentum sources in the $x$ direction and $y$ direction, respectively.

To account for the turbulent fluctuations in the bridge deck-wave interaction problem, the SST $k-\omega$ model is used as the turbulence closure for the RANS equations and the equations are as follows:
\[
\begin{align*}
\frac{\partial}{\partial t} (\rho k) + \frac{\partial}{\partial x_j} (\rho k u_j) &= \frac{\partial}{\partial x_j} \left( I_k \frac{\partial k}{\partial x_j} \right) + \tilde{G}_k - Y_k + S_k \\
\frac{\partial}{\partial t} (\rho \omega) + \frac{\partial}{\partial x_j} (\rho \omega u_j) &= \frac{\partial}{\partial x_j} \left( I_\omega \frac{\partial \omega}{\partial x_j} \right) + G_\omega - Y_\omega + D_\omega + S_\omega
\end{align*}
\]

(6.2a)

(6.2b)

where \( I_k \) and \( I_\omega \) are the effective diffusivity of \( k \) and \( \omega \); \( \tilde{G}_k \) represents the generation of turbulence kinetic energy due to the mean velocity gradients, calculated from \( G_k \); \( G_\omega \) is the generation of \( \omega \); \( Y_k \) and \( Y_\omega \) are the dissipation of \( k \) and \( \omega \), respectively; \( D_\omega \) is the cross-diffusion term; and \( S_k \) and \( S_\omega \) are user-defined source terms. All the expressions of the parameters can be referred to the theory guide in Fluent and the constants are set as the default values as deemed appropriate.

### 6.2.2 Solitary Wave Model

The water particle velocities \( u \) and \( v \), water pressure \( p \), and free surface profile \( \eta \) of the solitary wave of the 2nd-order are expressed as (Sarpkaya and Isaacson 1981):

\[
\eta = \epsilon \text{sech}^2 q - \frac{3}{4} \epsilon^2 \text{sech}^2 q \text{tanh}^2 q
\]

(6.3a)

\[
\frac{v}{\rho g d} = \frac{\eta}{d} + 1 - \frac{s}{d} - \frac{3}{4} \epsilon^2 \text{sech}^2 q \left( \left( \frac{s}{d} \right)^2 - 1 \right) (2 - 3 \text{sech}^2 q)
\]

(6.3b)

\[
\frac{u}{\sqrt{gd}} = \epsilon \text{sech}^2 q + \epsilon^2 \text{sech}^2 q \left\{ \frac{1}{4} - \text{sech}^2 q - \frac{3}{4} \left( \frac{s}{d} \right)^2 \right\} (2 - 3 \text{sech}^2 q)
\]

(6.3c)

\[
\frac{v}{\sqrt{gd}} = \epsilon \sqrt{3\epsilon} \left( \frac{s}{d} \right) \text{sech}^2 q \ \text{tanh} q \left\{ 1 - \epsilon \left[ \frac{3}{8} + 2 \text{sech}^2 q + \frac{1}{2} \left( \frac{s}{d} \right)^2 \right] (1 - 3 \text{sech}^2 q) \right\}
\]

(6.3d)

where \( \epsilon = \frac{u}{d} \), \( q = \frac{\sqrt{3\epsilon}}{2d} \left( 1 - \frac{5}{8} \epsilon \right) (x - ct) \), \( s = y + d \), \( d \) is the still water depth, \( H \) is the wave height, and \( y \) is the distance from the SWL and is negative if it is in the same direction with the gravitational acceleration. Hence, the wave velocity \( c \) can be calculated as:

\[
\frac{c}{\sqrt{gd}} = 1 + \frac{1}{2} \epsilon - \frac{3}{20} \epsilon^2
\]

(6.4)

It can be observed from Eq. (6.3a) that the solitary wave crest is located at \( x = 0 \) when \( t = 0 \) s, namely, the wave crest is just located at the inlet boundary, which will result in inaccurate results. Hence, the incident solitary wave should be shifted leftward by replacing \( t \) with \( t - t_0 \), where \( t_0 = L_{\text{min}}/c \) and \( L_{\text{min}} \) is defined as the minimum length to allow the wave crest to reach the inlet boundary after a certain time. Therefore, in this way the water surface could increase gradually at the inlet boundary. \( L_{\text{min}} \) should be greater than the effective wave length \( L_e \), where \( L_e = 2\pi d / \sqrt{\frac{3H}{d}} \) (Dong and Zhan 2009).

### 6.2.3 Computational Domain and Model Setup

Fig. 1 shows the schematic diagram for the computational domain of the 2D cases (250 m in length \( \times \) 13 m in height), where the line EF is the still water level (SWL), which separates the regions of the air and water. For the boundary conditions, lines AB and BD are defined as pressure outlet boundary condition maintained as the atmospheric pressure (101,325 pa), the line AC is the velocity inlet, and CD is the no slip stationary wall condition. The equations of \( u \) and \( v \), i.e. Eqs. (6.3c) and (6.3d), are compiled into
Fluent as the velocity inlet boundary condition by the User Defined Functions (UDFs), and the free surface profile $\eta$ is controlled by Eq. (6.3a).

The geometric parameters of the typical twin bridge decks with some simplifications are also shown in Fig. 6.1. This prototype bridge (landward or seaward), 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 island/coastal communities (Huang and Xiao 2009; Xiao et al. 2010; Hayatdavoodi et al. 2014). The width of each deck is 10.45 m, the girder height is 1.05 m and the deck depth is 0.3 m. All the six girders, each with a width of 0.3 m, are simplified as rectangles and evenly distributed. The deck gap is primarily taken as 20 m, and then variable deck gaps will be considered in the parametric study.

For the setups of the laminar flow in Fluent, the pressure-based solver (seggregated) is chosen for the transient flow, the Pressure-Implicit with Splitting of Operators (PISO) scheme (FHWA 2009; Bricker et al. 2012) is utilized for the pressure-velocity coupling method, and the PRESTO (PREssure STaggering Option) scheme is set for the pressure spatial discretization. As a two-phase flow problem, the VOF (Volume of Fluid) method is employed to prescribe the dynamic free surface. The least squares cell based scheme is used for the gradient discretization, second order upwind is used for momentum advection terms, and Geo-Reconstruct is used for the volume fraction equations. For the setups of the SST $k$-$\omega$ model in Fluent, the turbulence damping is turned on. For the velocity inlet boundary (AC) shown in Fig. 6.1, the turbulence intensity is 2% and the turbulence viscosity ratio is 10%. For the top (AB) and outlet (BD) of the calculation domain, the backflow turbulence intensity and the backflow turbulence viscosity ratio are the same as that set for the velocity inlet boundary. Second order upwind is used for the spatial discretization of the turbulence kinetic energy and the specific dissipation rate. All the other setups are the same as that set in the laminar flow.

6.2.4 Wave Model Verification

Prior to the verification of the accuracy of the generated waves with analytical results and the laboratory experiments, a mesh sensitivity study is conducted. As such, in order to accommodate the following verifications, a computational domain of 13 m in length and 0.9 m in height is considered and a value of 0.36 for $\varepsilon$ (the ratio of the wave
height, 0.14 m, to the still water depth, 0.39 m) is chosen. Different mesh resolutions in the near water zone, \( dx = 0.0025 \) m and 0.001 m in the \( x \) direction and \( dy = 0.0025 \) m and 0.001 m in the \( y \) direction are used, respectively; and the time steps of 0.001 s and 0.002 s are considered based on the requirements of the Courant Number. The obtained results show that there are no significant differences on the achieved wave profiles. Therefore, the final grid meshes in the near water zone using the structured mesh method are \( dx = 0.0025 \) m in the \( x \) direction and \( dy = 0.0025 \) m in the \( y \) direction and the time step \( dt = 0.0025 \) s is adopted.

For the verification of the wave profiles with analytical results, the wave height of 0.14 m with the \( \varepsilon \) values of 0.24, 0.30, and 0.36 is chosen to cover the wave profiles used in the current study. Fig. 6.2 shows the comparisons of the free surface profiles between the analytical results and the numerical results from both the laminar flow model and the SST \( k-\omega \) model. It is observed that there is a slight difference of about 2.3\% between the wave crest for \( \varepsilon = 0.36 \) when \( x/d \) is around 15 (the bridge models will be placed well around or within this index to ensure a close prediction of the wave profiles). However, for smaller \( \varepsilon \) values, the numerical wave profiles agree quite well with the analytical ones. This may be due to that the neglected higher order terms in the equations (Eq. (6.3)) would be prominent for large \( \varepsilon \) values. It is noteworthy that there is almost no difference between the results of the laminar flow model and the SST \( k-\omega \) model, promising close predictions of the wave forces on the bridge decks between these two models.

Fig. 6.2 Comparisons of the free surface profiles between the numerical results and analytical ones. \( x \) denotes the distance from the inlet boundary and \( d \) is the still water depth. (a) \( \varepsilon = 0.24 \); (b) \( \varepsilon = 0.30 \); and (c) \( \varepsilon = 0.36 \).

The accuracy of the wave model is further verified with the wave forces obtained in the experimental measurements by McPherson (2008). McPherson (2008) tested a
section of U.S. 90 Bridge in a 1:20 scale in a wave basin and perforated railings on the
seaward side of the bridge deck were considered. One dummy bridge section was placed
on each side of the bridge model in order to eliminate three dimensional (3D) end effects
and bridge pilings were used to support the dummy bridge sections to simulate in field
scenarios. The bridge model was fixed with the bottom of the girder located 0.41 m
above the bottom. Only one wave height, 0.14 m, was used and four water depths, 0.39 m,
0.41 m, 0.48 m, and 0.54 m, were considered with the ε values in the range from 0.24 to
0.36.

The bridge model geometry is shown in Fig. 6.3 where all the parameters are
exactly the same as used in the experimental study except the railings. To accommodate
the accuracy of the bridge model in the 2D computational domain, a railing height of 3
cm is considered above the bridge deck with a 2 cm clearance. The grid resolutions 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; 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. The meshes near the walls of the bridge deck model satisfy the requirement
that the height of the first grid should be in the logarithmic layer in order to obtain an
acceptable accuracy in bridge engineering (Bredberg 2000). Structured meshes are
employed, the simulation time is 6 s, and the time step is dt=0.0025 s.

![Fig. 6.3 Schematic diagram for the bridge model in the experimental study by McPherson (2008)](image)

The wave forces on the bridge deck model are predicted from the beginning of
simulations. The total force component along the specified force vector \( \vec{a} \) (horizontal
force or vertical force) on the wall zones of the bridge deck model is computed by
summing the dot product of the pressure and viscous forces on each face with the
specified force vector. The terms in this summation represent the pressure and viscous
force components in the direction of the vector \( \vec{a} \) as:

\[
F_a = \vec{a} \cdot \vec{F}_p + \vec{a} \cdot \vec{F}_v
\]

(6.5)

where \( \vec{a} \) is the specified force vector, \( \vec{F}_p \) is the pressure force vector, and \( \vec{F}_v \) is the viscous
force vector. A reference pressure, \( p_{ref} \) (the operating pressure, 101,325 Pa), is used to
normalize the cell pressure for computation of the pressure force to reduce the round-off
error as:
\[ \vec{F}_p = \sum_{i=1}^{n}(p - p_{ref})\hat{n} \]  

(6.6)

where \(n\) is the number of faces, \(A\) is the area of the face, and \(\hat{n}\) is the unit normal to the face. The associated force coefficients (\(c_l\) and \(c_d\), uplift and drag force coefficients, respectively) are computed for the selected wall zones in the “Monitors” setup in Fluent. The force coefficient is defined as the force divided by \(\frac{1}{2}\rho v^2 A\), where \(\rho\), \(v\) and \(A\) are the density, velocity, and area, respectively, and these values are set in the “Reference Values” in Fluent.

The verification of the obtained wave forces with those by McPherson (2008) for the laminar flow model was reported in our previous study (Xu and Cai 2014) and is also incorporated here in Figs. 6.4 and 6.5. Generally speaking, the laminar flow model and the SST \(k-\omega\) model predict quite close results on the bluff body-typed bridge deck model and the obtained wave forces by these two flow models are in good agreement with those documented by McPherson (2008). For the comparisons of the wave forces between the numerical results and the experimental measurements, it is observed in Fig. 6.4 that a small difference between the maximum horizontal forces when \(d = 0.54\) m is found, which may be due to the simplified railing as compared with the 3D perforated railings used in the laboratory experiments and this was expected. In Fig. 6.5, some small differences between the maximum vertical forces when \(d = 0.39\) m and \(0.41\) m are noticed and this is probably due to the effects of the entrapped air since the entrapped air cannot escape in a timely manner in 2D simulations.

![Fig. 6.4 Comparisons of the horizontal forces](image)
6.3 Parametric Study

In this section, the characteristics of the solitary wave induced forces on the twin bridge decks, including the hydrodynamic interference effects and the effects of girder types, different SWLs, various deck gaps, and deck vibrations, are parametrically studied. For the following subsections, as an example, most of the simulations considered a fixed deck gap of 20 m that is the same as that of the old Escambia Bay Bridge in Florida. For the analysis of the effects of the deck gaps and the hydrodynamic interference effects, different deck gaps will be considered and discussed thereafter. A wave height of 2.0 m is selected for all the simulations in the current study with the \( \varepsilon \) values in the verified range from 0.24 to 0.36.

The cases considered for the fixed deck gap of 20 m are listed in Table 6.1, where three different SWLs with various structure elevations are chosen to typically represent substantial scenarios for coastal bridge decks. The case name, for example, SWL 6.0 m, is named according to the SWL of 6.0 m defined in the calculation domain. The submersion coefficient \( C_s \) is the ratio of \( S \) to \( H_B \), where \( S \) is the distance between the bottom of the superstructure to the SWL (negative if the structure is submerged in the water) and \( H_B \) is the height of the bridge superstructure. For a fixed value of the submersion coefficient \( C_s \), the bridge elevation is correspondingly varied according to the SWL considered, as shown in Table 6.1.

The grid resolutions for the laminar flow are: \( \Delta x=0.1 \text{ m} \) and \( \Delta y=0.1 \text{ m} \) for the zone nearby the bridge model; \( \Delta x=0.2 \text{ m} \) and \( \Delta y=0.1 \text{ m} \) for the near water surface zone at the far field from the bridge model; \( \Delta x=0.2 \text{ m} \) and \( \Delta y=0.2 \text{ m} \) for the deep water zone, and \( \Delta x=0.2 \text{ m} \) and \( \Delta y=0.4 \text{ m} \) for the air zone at the far field from the bridge model. The
grid resolutions for the turbulent flow are shown in Fig. 6.6: $\Delta x=0.05$ m and $\Delta y=0.05$ m for the zone nearby the bridge model; $\Delta x=0.2$ m and $\Delta y=0.05$ m for the near water surface zone at the far field from the bridge model; $\Delta x=0.2$ m and $\Delta y=0.1$ m for the deep water zone, and $\Delta x=0.2$ m and $\Delta y=0.2$ m for the air zone at the far field from the bridge model. The total meshed cells for the turbulent flow are around 272 000, about 2 times the meshes used in the laminar flow.

Table 6.1 Cases considered with different submersion coefficient for the fixed deck gap of 20 m

<table>
<thead>
<tr>
<th>Cases</th>
<th>Still water depth (m)</th>
<th>$C_s = S/H_b$</th>
<th>Bridge elevation (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWL 6.0m 6.0</td>
<td>0.444</td>
<td>6.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.222</td>
<td>6.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.222</td>
<td>5.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.444</td>
<td>5.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.667</td>
<td>5.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.889</td>
<td>4.8</td>
<td></td>
</tr>
<tr>
<td>SWL 7.2m 7.2</td>
<td>0.444</td>
<td>7.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.222</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>7.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.222</td>
<td>6.9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.444</td>
<td>6.6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.667</td>
<td>6.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.889</td>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td>SWL 8.4m 8.4</td>
<td>0.444</td>
<td>9.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.222</td>
<td>8.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>8.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.222</td>
<td>8.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.444</td>
<td>7.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.667</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.889</td>
<td>7.2</td>
<td></td>
</tr>
</tbody>
</table>

The calculation time is 27 s which is long enough to satisfy the requirement that the wave completely passes through the twin bridge decks. The time step is 0.005 s for the laminar flow and 0.002 s for the turbulent flow. The difference for the run time of the laminar flow and the turbulent flow is significant, with about 10 cpu hours for the laminar
flow and about 240 cpu hours for the turbulent flow. The time is based on 64-bit processors with a frequency of 2.6 GHz and 2 G Random-access memory (RAM).

### 6.3.1 Time History of the Wave Forces on the Twin Bridge Decks

Fig. 6.7 demonstrates the bridge deck-wave interaction for case SWL 7.2 m with the bridge elevation 7.2 m from the turbulent flow simulations. The coming wave crest of 9.2 m (the water depth 7.2 m plus the wave height 2.0 m) is higher than the top surface of the twin bridge decks (the bottom elevation 7.2 m plus \( H_b \) 1.35 m). From the demonstration of the twin bridge deck-wave interaction, the on-deck water height on the landward deck is smaller than that on the seaward deck because of the energy loss occurred during the bridge deck-wave interaction of the seaward deck.

(a) 14s  
(b) 15s  
(c) 16s  
(d) 17s  
(e) 18s  
(f) 19s  
(g) 20s  
(h) 21s

**Fig. 6.7 Twin bridge deck-wave interaction for case SWL 7.2 m with the bridge elevation 7.2 m**

The time-history horizontal and vertical forces from the turbulent flow simulations for the three different SWLs when the submersion coefficient \( C_s \) is 0 are shown in Figs. 6.8 and 6.9, respectively. The absolute values of both the positive and negative peak forces on the seaward deck are correspondingly larger than those on the landward deck. Generally speaking, the positive peak horizontal force is larger than the negative peak horizontal force (absolute value) on both the seaward deck and the landward deck. Hence, only the positive peak horizontal forces are considered in the following discussions. Because the results of the laminar flow show the same characteristics as that by the turbulent flow, the time-history wave forces from the laminar flow simulations are not shown here.

### 6.3.2 Effects of Girder Types

In the literature, the bridge decks with simplified rectangular type girders are usually adopted for the numerical analysis of the wave forces on the bridge deck (Xiao et al. 2010; Bozorgnia et al. 2010; Jin and Meng 2011; Hayatdavoodi et al. 2014). In this part, the wave forces on the single bridge decks with the rectangular type girders and with the “I” type girders are compared in order to obtain an overview of the differences between the corresponding results.
Fig. 6.8 Time histories of horizontal forces for different SWLs when the submersion coefficient $C_s$ is 0

Fig. 6.9 Time histories of vertical forces for different SWLs when the submersion coefficient $C_s$ is 0

Table 6.2 Properties of the simplified “I” type girder

<table>
<thead>
<tr>
<th>Parameter</th>
<th>AASHTO Type III girder (Unit: in)</th>
<th>Numerical model (Unit: m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top flange width</td>
<td>16</td>
<td>0.5</td>
</tr>
<tr>
<td>Top flange average thickness</td>
<td>9</td>
<td>0.25</td>
</tr>
<tr>
<td>Bottom flange width</td>
<td>22</td>
<td>0.5</td>
</tr>
<tr>
<td>Bottom flange average thickness</td>
<td>11</td>
<td>0.3</td>
</tr>
<tr>
<td>Total depth</td>
<td>45</td>
<td>1.15</td>
</tr>
<tr>
<td>Web width</td>
<td>7</td>
<td>0.2</td>
</tr>
</tbody>
</table>
While the single bridge deck with the simplified rectangular type girders has the same geometry as that shown in Fig. 6.1, the single bridge deck with the simplified “I” type girders is considered with the parameters listed in Table 6.2. Several small adjustments are made for the parameters in order to accommodate the modeling and meshing. In addition, all the other parameters, such as the deck depth and width, and the layout of the girders, are kept the same. The grid mesh nearby the single deck with the simplified “I” type girders is shown in Fig. 6.10.

Fig. 6.11 shows the comparisons between the wave forces on the single bridge decks with the rectangular type girders and the “I” type girders. As shown in Fig. 6.11(a), the horizontal forces on the single deck with rectangular type girders are relatively larger than those on the single deck with “I” type girders for both the laminar flow and the turbulent flow, especially when the submersion coefficient $C_s$ is negative. The maximum difference is less than 10% when the submersion coefficient $C_s$ is -0.444 for the laminar flow. The reason may be that the uneven and staggered faces for the “I” type girders disturb the coming wave current to some extent, especially when the bridge deck is submerged. However, this disturbance does not have much influence on the vertical forces as shown in Fig. 6.11(b). Hence, reasonable results can be obtained by employing the bridge decks with simplified rectangular type girders.

Fig. 6.11 Comparisons between the wave forces on the single decks with different girder types

6.3.3 Effects of Different SWLs

The positive peak horizontal and vertical forces from the time-history wave forces under both the laminar flow and the turbulent flow are shown in Figs. 6.12 and 6.13,
respectively. The difference between the results of the laminar flow and the turbulent flow is within 10% for most cases, indicating reasonable results can be achieved by the laminar flow. Hence, the following discussions are based on the results by the turbulent flow.

Generally speaking, the wave forces on the seaward deck are larger than those on the landward deck, which can also be observed in the time-history wave forces as shown previously in Figs. 6.8 and 6.9. In Fig. 6.12, the positive peak horizontal forces with a lower SWL are larger than those with a higher SWL on both the seaward deck and the landward deck with a given submersion coefficient. Comparatively, the positive peak vertical forces with a lower SWL are smaller than those with a higher SWL on the seaward deck ($C_s$ ranges from -0.444 to 0.444) and on the landward deck ($C_s$ ranges from about -0.8 to 0.444) as shown in Fig. 6.13.

![Fig. 6.12 Horizontal forces on the twin bridge decks for different SWLs](image1)

![Fig. 6.13 Vertical forces on the twin bridge decks for different SWLs](image2)

The possible reasons accounting for the differences between the wave forces under different SWLs include: (a) the wave velocity (calculated from Eq. (6.4)) with a
higher SWL ($c=10.08 \text{ m/s when } d=8.4 \text{ m})$ is larger than that with a smaller SWL ($c=8.82 \text{ m/s when } d=6.0 \text{ m}$). However, the horizontal velocities of the water particles at the wave crest sections (from the bottom to the wave crest) have the opposite features (calculated from Eq. (6.3c)), as shown in Fig. 6.14. Here, for example, the wave-crest section for $d=6.0 \text{ m}$ ranges from 0 m to 8.0 m (the water depth, $d$, plus the wave height, $H$). From Fig. 6.14, it is worth noting that the water particle velocities with a smaller SWL are larger than those with a larger SWL along the whole crest section and this may explain that the positive peak horizontal forces with a lower SWL are larger than those with a higher SWL on both the twin decks with a fixed submersion coefficient; and (b) the flow field around the bridge deck (including submerged and subaqueous conditions) is very complex with many changing variables, such as the time when the maximum force occurs, the amount of the green water on the deck, the turbulence underneath the deck and the interference effects of the landward deck. As such, more studies are needed in order to interpret the different phenomena for the horizontal forces and the vertical forces considering different SWLs, such as the comparisons of the pressure distribution, as well as the velocity magnitude, around the bridge decks under different SWLs.

6.3.4 Effects of Different Deck Gaps

In practical design, the bridge deck gap varies according to the site environment, the construction cost, and the structural requirements. For example, while the old Escambia Bay Bridge (damaged in Hurricane Ivan) has a deck gap of 20 m, the old Biloxi Bay Bridge (damaged in Hurricane Katrina) has a very narrow deck gap of less than 1 m. As such, the effects of the deck gaps on the wave forces need to be studied in order to obtain an overview of the deck interference effect. In this part, seven different deck gaps (1 m, 2 m, 3 m, 5 m, 10 m, 15 m and 20 m) with six SWLs (6.9 m, 7.2 m, 7.5 m, 7.8 m, 8.1 m and 8.4 m) and a fixed bridge elevation (7.2 m) are typically chosen for this purpose.

Typical examples of the time-history wave forces selected from the turbulent flow simulations considering different deck gaps are shown in Figs. 15 and 16 with the
submersion coefficient of -0.222. As the deck gap increases, the time lag between the peak wave forces on the seaward deck and the landward deck increases as expected.

![Graph showing time-history horizontal forces with different deck gaps](image1)

**Fig. 15 Examples of the time-history horizontal forces with different deck gaps**

![Graph showing time-history vertical forces with different deck gaps](image2)

**Fig. 16 Examples of the time-history vertical forces with different deck gaps**

The positive peak wave forces on the twin bridge decks are shown in Figs. 17 and 18. For almost all the cases, the wave forces on the seaward deck are larger than those on the landward deck. It is observed that in Fig. 17 there are no clear trends for the horizontal wave forces on the twin bridge decks considering different SWLs when the deck gap ranges from 1 m to 5 m. This phenomenon may be due to the relative positions between different SWLs and the bottom of the bridge deck, as well as the wave height, which complicates the deck-wave interaction process. However, as shown in Fig. 18, the
vertical forces on the seaward deck are less sensitive to the deck gap and in general decreases with the increase of the deck gap.

(a) SWL 6.9m

(b) SWL 7.2m

(c) SWL 7.5m

(d) SWL 7.8m

(e) SWL 8.1m

(f) SWL 8.4m

Fig. 17 Horizontal forces versus different deck gaps
6.3.5 Characteristics of the Wave Forces on the Landward Bridge Deck

By using the wave forces from the above discussed sections, the characteristics of the wave forces on the landward bridge deck can be expressed as normalized expressions, $F_{h_{\text{landward}}}/F_{h_{\text{seaward}}}$ and $F_{v_{\text{landward}}}/F_{v_{\text{seaward}}}$, as shown in Figs. 6.19 and 6.20,
respectively, where the results for both the fixed deck gap and various deck gaps are demonstrated. Here, $F_{h\text{landward}}$ and $F_{v\text{landward}}$ are the peak horizontal and vertical wave forces on the landward deck, respectively, and $F_{h\text{seaward}}$ and $F_{v\text{seaward}}$ are the corresponding ones on the seaward deck.

![Graph](image1)

**Fig. 6.19 Wave force normalization for the fixed deck gap of 20 m**

![Graph](image2)

**Fig. 6.20 Wave force normalization for different deck gaps**

In Fig. 6.19, generally speaking, the ratios of $F_{h\text{landward}}/F_{h\text{seaward}}$ and $F_{v\text{landward}}/F_{v\text{seaward}}$ with a lower SWL are larger than those with a higher SWL when the submersion coefficient is negative ($C_s$ ranges from -0.889 to 0). However, it is not consistent when the submersion coefficient is positive ($C_s$ ranges from 0 to 0.444). The ratios of $F_{h\text{landward}}/F_{h\text{seaward}}$ are smaller than 0.8 when the submersion coefficient is negative and smaller than 0.9 when the submersion coefficient is positive. For most vertical force cases, the ratios of $F_{v\text{landward}}/F_{v\text{seaward}}$ are smaller than 0.9. As a result, for the cases studied with the fixed deck gap (20 m), both the horizontal forces and the vertical forces on the landward bridge deck can be taken as 90% of those on the seaward bridge deck in designing new bridges or evaluating and retrofitting existing bridges.

For the results considering different deck gaps as shown in Fig. 6.20, there are no general trends for the ratios of the horizontal force $F_{h\text{landward}}/F_{h\text{seaward}}$ when the deck gap
ranges from 1 m to 5 m. However, general trends can be found for the ratios of the vertical force $F_v_{\text{landward}}/F_v_{\text{seaward}}$ when the gap ranges from 1 m to 5 m, i.e., the ratios with a higher SWL are larger than those with a lower SWL. When the deck gap is larger than 5 m, the ratios of $F_h_{\text{landward}}/F_h_{\text{seaward}}$ and $F_v_{\text{landward}}/F_v_{\text{seaward}}$ are smaller than 0.8 and 0.9, respectively. Hence, in the cases studied for a fixed bridge elevation (7.2 m) with different SWLs, factors of 0.8 and 0.9 for the horizontal forces and the vertical forces, respectively, can be chosen when the gap between the twin bridge decks is larger than 5 m.

### 6.3.6 Hydrodynamic Interference Effects between the Twin Bridge Decks

As discussed earlier, the landward deck can be deemed as a nearby structure of the seaward deck and, therefore, the flow field may be disturbed by the presence of the landward deck as compared with a single bridge deck. As such, the wave forces on the seaward bridge deck may be larger than those on the single bridge deck due to the presence of the nearby structure (the landward deck) based on the observations by Bricker and Nakayama (2014). The interference effects of the twin decks are quantified here based on the wave forces on the single bridge deck using $F_h/F_h_{\text{single}}$ or $F_v/F_v_{\text{single}}$. In the cases for the single deck, only one bridge deck with the same geometry as that for the twin bridge decks is considered in the computational domain. Based on the above discussion, only the turbulent flow results are discussed subsequently.

Fig. 6.21 shows the comparisons between the wave forces on the twin bridge decks with the deck gap of 20 m and on the corresponding single deck. As demonstrated in this figure, there are almost no differences between the wave forces on the seaward deck and on the single bridge deck with such a deck gap. It means that the interference effects on the seaward deck (i.e., the flow field around the seaward deck) due to the presence of the landward deck are negligible for this deck gap. However, as discussed earlier, the interference effect of the seaward deck on the landward deck is obvious with the ratios of $F_h_{\text{landward}}/F_h_{\text{single}}$ and $F_v_{\text{landward}}/F_v_{\text{single}}$ being much less than 1.0.

![Fig. 6.21 Comparisons between the wave forces on the twin bridge decks with the deck gap of 20 m and those on the single deck](image-url)
In Fig. 6.22, the comparisons between the wave forces on the twin bridge decks with various deck gaps and on the corresponding single deck are demonstrated. It is observed that the ratios of horizontal forces $F_h/F_{h\text{single}}$ are sensitive to the deck gaps, especially when the deck gap is smaller than 5 m. In most cases when the deck gap falls in this range, the horizontal forces on the seaward deck are smaller than those on the single bridge deck. While for vertical forces with the deck gap being smaller than 5 m, the ratios of $F_{v\text{seaward}}/F_{v\text{single}}$ range from 1.1 to 1.3 and in general decrease with the increase of the deck gap. It is identified that the submerision coefficient (the relative position between the SWL and the bridge elevation) is another important factor when considering the hydrodynamic interference effects. Generally speaking, for this fixed bridge elevation (7.2 m), while the ratios of $F_v/F_{v\text{single}}$ decrease with the increase of the submerision coefficient with small deck gaps (from 1 m to 5 m), the ratios of $F_h/F_{h\text{single}}$ almost demonstrate the opposite trend. For both the vertical and horizontal forces, when the deck gap is larger than 10 m, the ratios of $F_h/F_{h\text{single}}$ and $F_v/F_{v\text{single}}$ tend to level off as the gap increases and hence, the twin bridge decks become more independently. It is noted that while $F_v/F_{v\text{single}}$ or $F_h/F_{h\text{single}}$ eventually approaches 1.0 as the increase of the deck gap, the ratios of $F_v/F_{v\text{single}}$ or $F_h/F_{h\text{single}}$ do not approach 1.0. This is because the wave has lost its energy when it passed the seaward deck and reached the landward deck.

![Graphs showing horizontal and vertical force comparisons](image)

**Fig. 6.22** Comparisons between the wave forces on the twin bridge decks with various deck gaps and those on the single deck

### 6.3.7 Effects of the Deck Vibrations

To investigate the effects of deck movement on the wave forces, the flexible setup of the deck supports is considered in the present study where the twin bridge decks vibrate along with the wave propagation and can be deemed as two separate mass-spring-damper systems as shown in Figs. 6.23 and 6.24. As a matter of fact, the stiffness of the spring represents two parts, the substructure stiffness and the interface stiffness between the substructure and the superstructure (Fig. 6.23). The configuration of the computational domain is shown in Fig. 6.24(a), where the three fixed zones are separated by two remeshing zones. Each remeshing zone is considered as one rigid body, which vibrates horizontally in both directions as a whole body along with the specified bridge
deck within the remeshing zone. During the moving of the remeshing zones, a dynamic mesh updating technique is employed such that new meshes are generated and old meshes collapse at the boundaries between the fixed zones and the remeshing zones. Owing to that the structured meshes are mainly adopted in the simulations, the layering mesh method is chosen with the split factor 0.4 and the collapse factor 0.2.

Fig. 6.23 Schematic diagram for the twin bridge deck-wave interaction under solitary waves. The interface between the superstructure and the substructure is not shown for clarity. Different pile systems (Pile systems 1 and 2) result in different structure stiffnesses or flexibilities.

For the flexible setup as shown in Fig. 6.24(b), the vibrations of the twin bridge decks (connected with the remeshing zone as a whole rigid body for each deck) are determined by the following equations:

\[
C1 \quad k1 \quad m1 \quad y1 \quad F1(t) \quad x1 \\
C2 \quad k2 \quad m2 \quad y2 \quad F2(t) \quad x2
\]
\begin{align}
\ddot{x}_1 + 2\xi_1\omega_{01}\dot{x}_1 + \omega_{01}^2 x_1 &= \frac{F_1(t)}{m_1} \\
\ddot{x}_2 + 2\xi_2\omega_{02}\dot{x}_2 + \omega_{02}^2 x_2 &= \frac{F_2(t)}{m_2}
\end{align}

(6.7)  (6.8)

where $x$ is the instantaneous displacement of the bridge deck in the $x$ direction, $\dot{x}$ is the damping ratio, $\omega_0$ is the natural frequency of the bridge superstructure, $\omega_0 = \frac{2\pi}{T_s}$, $T_s$ is the structural vibration period, $F(t)$ is the instantaneous horizontal force integrated from the hydraulic pressure along the bridge deck surface, and the subscript numbers of 1 and 2 refer to the seaward deck and landward deck, respectively.

The procedure for the dynamic mesh updating technique during the twin bridge deck-wave interaction is described as follows (Xu et al. 2009; Ou et al. 2009; Xu et al. 2014). Firstly, in each time step, the velocity and the pressure fields can be obtained through the CFD calculation. The total force components (the horizontal force $F(t)$, the vertical force, etc.) on the bridge decks can then be obtained for the current time step and saved. Subsequently, by substituting the obtained horizontal forces, $F(t)$, into Eqs. (6.7) and (6.8), the velocities of the twin bridge deck are predicted by using the Newmark-$\beta$ method for the structural dynamic analysis correspondingly. Finally, these velocities are attributed, respectively, to the rigid bodies that will move to new positions for one time step and hence, the remeshing zone is updated accordingly. Once the mesh is updated, the whole fluid domain is ready for the CFD calculation in the next time step. This loop will continue to the final time step.

The capabilities of the mass-spring-damper system were verified through the general observations for the wave forces compared with those from a laboratory experiment (Bradner et al. 2011), where a 1:5 scaled bridge deck model was tested in a large wave flume. This scaled bridge deck model was placed on two linear guide rails with each one supporting one end of the span. Two types of setups, the rigid setup and flexible setup (including soft springs setup and medium springs setup), were considered in the experimental study. For the rigid setup, the bridge deck is rigidly supported and is not allowed to move or vibrate. For the flexible setup, the capability of the deck vibration was determined by the springs’ stiffness installed between the tested bridge deck model and the end anchors on the guide rails (two springs used and each one was installed for each guide rail). In order to determine the springs’ stiffness, a finite element analysis was firstly conducted by typifying several different elevations of the bridge deck. Then, the structural vibration period was obtained for the corresponding deck elevation and a suitable support stiffness was subsequently chosen to match this period value.

The flexible setup employed in the experimental study can be realized using the mass-spring-damper system. However, several differences should be noted in the verification procedure: (a) 2D numerical simulations were conducted in the current study which may not fully capture all the characteristics observed in the experimental study; (b) since the span length of the bridge deck model is 3.45 m, only slightly smaller than the width of the wave flume, 3.66 m, the 3D end effects may play significant roles in the experimental study; (c) the friction force between the bridge deck model and the supporting guide rails was not desired but cannot be avoided in the experimental study. However, this force was neglected and taken as 0 in the numerical simulations; and (d) all the AASHTO type III girders were simplified as rectangles in the numerical simulations. While these differences were noticed, it was expected that reasonable predictions of the general experimental observations would be obtained. Other than the above discussed
differences, all other parameters considered in the verification are exactly the same as those used in the experimental study, as listed in Table 6.3. Due to the reason that Bradner et al. (2011) only presented one figure (Fig. 12 in Bradner et al. 2011) to illustrate the results of the flexible setup and they normalized the time histories of the wave forces without giving the actual wave force and the corresponding wave height information, a direct comparison of the wave forces between the current method and the experimental measurements is not possible.

Based on the wave steepness $H/gT^2$ and relative depth $d/gT^2$ (Sarpkaya and Isaacson 1981), Stokes 2nd order wave theory is proper for the wave height of 0.50 m. The numerical calculation domain for this verification is 40 m long and 2.5 m high. Using the wave properties listed in Table 6.3, the numerical wave profiles are obtained and compared with analytical results as shown in Fig. 6.25, demonstrating a good agreement with each other.

<table>
<thead>
<tr>
<th>Table 6.3 Parameters considered in the verification of the mass-spring-damper system with Bradner et al. (2011)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameter</td>
</tr>
<tr>
<td>Geometry properties</td>
</tr>
<tr>
<td>Girder height (m)</td>
</tr>
<tr>
<td>Girder spacing (m)</td>
</tr>
<tr>
<td>Deck thickness (m)</td>
</tr>
<tr>
<td>Overall height (m)</td>
</tr>
<tr>
<td>Span mass per unit length (kg)</td>
</tr>
<tr>
<td>Flexible setup properties</td>
</tr>
<tr>
<td>(soft springs setup)</td>
</tr>
<tr>
<td>Structural vibration period, $T_s$, (s)</td>
</tr>
<tr>
<td>Spring stiffness, $k$, (N/m)</td>
</tr>
<tr>
<td>Damping ratio, $\xi$</td>
</tr>
<tr>
<td>Damping coefficient, $c_\xi$, (N·s$^2$/m)</td>
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<tr>
<td>Wave properties</td>
</tr>
<tr>
<td>Still water depth, $d$, (m)</td>
</tr>
<tr>
<td>Wave height, $H$, (m)*</td>
</tr>
<tr>
<td>Wave period, $T$, (s)</td>
</tr>
<tr>
<td>Wave length (m)</td>
</tr>
</tbody>
</table>

Note: * Only one wave height of 0.50 m is considered here for the verification.
The general characteristics of the numerically predicted wave forces between the flexible setup and rigid setup are demonstrated in Fig. 6.26, where three distinguishable characteristics are observed: (a) a phase lag can be observed between the positive peak horizontal forces of the rigid setup and the flexible setup (soft springs setup); (b) while much smaller negative horizontal forces are found for the rigid setup, significant negative horizontal forces are observed for the flexible setup (soft springs setup). This is probably due to the consideration of the inertia forces of the bridge deck; and (c) there is no significant difference on the positive peak vertical forces. These observations follow the same trends as those documented by Bradner et al. (2011), indicating that the discussed mass-spring-damper system has a good capability to capture the general dynamic characteristics of the bridge deck-wave interaction problems.

![Fig. 6.26 Comparisons of the numerical wave forces between the flexible setup (soft springs setup) and rigid setup](image)

In the current study, only one stiffness for the twin bridge decks is considered in order to demonstrate the dynamic characteristics of the twin bridge decks under solitary waves. In the 2D simulations, the mass is taken as 9716 kg per unit length according to the study by Xiao et al. (2010) (154000kg/15.85m = 9716kg per unit length). The parameters considered for this chosen stiffness are listed in Table 6.4, where the spring stiffness $k = m\omega_0^2$ and the damping coefficient $c = 2\xi\omega_0 m$. Since only limited cases are considered in the present study, further studies are needed to warrant a deeper understanding of the twin bridge decks-wave interaction problems under solitary waves with variable wave heights and different spring stiffnesses for both of the decks.

Fig. 6.27 shows the selected examples of the structure displacement for the twin bridge decks. The positive displacement means the structure moves in the positive $x$ direction or landward. Apparently, the structure displacement for the seaward deck (ranges from 10 cm to 15 cm) is larger than that for the landward deck (ranges from 9 cm
to 10 cm) for each case, and this is closely related with the wave forces distribution on both decks. It is also observed that the structure displacement has the maximum value when the submersion coefficient $C_s$ is -0.889, which is directly reflected by the general trend of the horizontal forces as shown earlier in Fig. 6.12.

Table 6.4 Parameters for the considered mass-spring-damper systems

<table>
<thead>
<tr>
<th>Parameter</th>
<th>seaward deck</th>
<th>landward deck</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_s$ (s)</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>$m$ (kg)</td>
<td>9716</td>
<td>9716</td>
</tr>
<tr>
<td>$\xi$</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>$k$ (kN/m)</td>
<td>170</td>
<td>170</td>
</tr>
<tr>
<td>$c$ (N·s²/m)</td>
<td>4070</td>
<td>4070</td>
</tr>
</tbody>
</table>

Fig. 6.27 Selected examples of the structure displacement for the twin bridge decks

Fig. 6.28 shows a typical comparison of the wave forces on the twin bridge decks considering the fixed setup and the flexible setup. Generally speaking, there is not much difference between the wave forces on the fixed setup and those on the flexible setup. As compared with the effective wave length $L_e = 50$ m for the prescribed conditions, a movement of 15 cm is not large enough to cause significant disturbance of the flow field. The horizontal wave forces on both the seaward deck and the landward deck are found only slightly larger for the flexible setup when the submersion coefficient $C_s$ is negative. Thus, the bridge deck vibration does not have much influence on the wave forces under the prescribed conditions. However, a more systematic study is needed to fully understand the effect of the deck movement on the wave forces.

6.4 Concluding Remarks

From this research concerning wave forces on coastal twin bridge decks due to solitary waves under the prescribed conditions, conclusions can be drawn as follows:
(1) Comparisons between the wave forces on the twin bridge decks under the laminar flow and the turbulent flow show that reasonable results can be achieved by using the laminar flow model. The difference between the results by these two methods is within 10% for most cases.

![Diagram](image)

Fig. 6.28 Comparisons between the wave forces on the twin bridge decks considering the fixed setup and the flexible setup

(2) For the wave forces on the twin bridge decks with a 20 m deck gap considering different submersion coefficient and different SWLs, the positive peak horizontal forces with a lower SWL are larger than those with a higher SWL on both decks with the same submersion coefficient. However, the positive peak vertical forces with a lower SWL are smaller than those with a higher SWL on both decks with a specifically smaller range of the submersion coefficient for each deck.

(3) For the wave forces considering different deck gaps with a fixed elevation (7.2 m) of the bridge superstructure, the wave forces on the seaward deck are larger than those on the landward deck for almost all the cases. The wave forces on both decks change more significantly when the gap ranges from 1 m to 5 m.

(4) Regarding the characteristics of the wave forces on the landward bridge deck, factors of 0.8 and 0.9 for $F_{h_{\text{landward}}}/F_{h_{\text{seaward}}}$ and $F_{v_{\text{landward}}}/F_{v_{\text{seaward}}}$, respectively, are suggested for practical engineering activities when the deck gap is larger than 5 m. However, in the cases when the bridge deck is well above the SWL with a deck gap of 20 m, larger factors, say, 0.9, should be taken for $F_{h_{\text{landward}}}/F_{h_{\text{seaward}}}$. When the deck gap is smaller than 5 m, the wave forces on the landward bridge deck should better be taken the same as those on the seaward bridge deck.

(6) The hydrodynamic interference effects between the twin bridge decks with the deck gap being smaller than 5 m are prominent, especially for the ratios of $F_{v_{\text{seaward}}}/F_{v_{\text{single}}}$ which ranges from 1.1 to 1.3 with such small deck gaps. However, when the deck gap is larger than 10 m, the ratios of $F_{h}/F_{h_{\text{single}}}$ and $F_{v}/F_{v_{\text{single}}}$ tend to level off as the gap increases and hence, the twin bridge decks become more independently. It is identified that the submersion coefficient is another important factor regarding the interference effects.
(7) For the typical case studied for the effects of the deck vibrations, it is observed that the bridge deck vibration does not have much influence on the wave forces. A small movement, say, 15 cm, for the bridge decks is not large enough to cause significant disturbance of the flow field.

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

(1) In the present study, 2D numerical simulations have been conducted. However, while 3D models (considering the railing and the diaphragm) may provide more reliable results, they may be much more computationally costly.

(2) Limited scenarios for the effects of the deck vibrations have been considered. Once the high waves reach a certain level, the vibration of the bridge decks along with the waves may cause remarkable effects on the characteristics of the wave forces in the twin bridge deck-wave interaction. Hence, more cases, with different wave heights, bridge geometries, attack angles and structure stiﬀnesses, need to be studied.

(3) Wave forces due to other wave types such as Stokes waves and theoretical expressions of the wave types in hurricanes need to be studied (Riggs 2007).

6.5 References


7.1 Introduction

Wave forces induced by hurricanes, one of the most disastrous natural phenomena, have caused devastating impacts on coastal structures, including many coastal bridges. Hurricane Katrina in 2005 is one of the deadliest natural disasters in the U.S. history and claimed $100 billion of property damage (Graumann et al. 2005). Several post-disaster reports have concluded that a large number of submerged bridge decks during a coastal inundation, as well as many subaerial bridge decks, are subjected to huge hydrodynamic loads including wave uplift forces; however, the existing short- and medium-span coastal bridges are rarely designed for this type of uplift forces (Robertson et al. 2007; Okeil and Cai 2008). More details about the bridge damage and the estimated repair costs due to hurricane Katrina in 2005 are reported by Padgett et al. (2008). In order to ensure that coastal bridges can survive these kinds of natural disasters in their service life, the wave effects, the failure mechanisms, and the retrofitting countermeasures, should be clearly understood when designing and retrofitting bridges in coastal areas. However, very few current design codes appear to sufficiently deal with coastal structures for storm surge and wave forces (McConnell et al. 2004; Douglass and Krolak 2008; AASHTO 2008).

As many coastal communities are witnessed to be very vulnerable to tsunamis or hurricanes especially in the last decade, more challenges are raised for researchers and engineers in coastal related engineering topics. One significant topic is the evaluation of the performance of coastal bridges during the tsunamis or the passage of hurricanes. To have a better understanding of the wave forces on coastal bridges, many efforts have been made with different experimental set-ups developed with periodical waves and solitary waves (Denson 1978; Denson 1980; Sugimoto and Unjoh 2006; McPherson 2008; AASHTO 2008; Cuomo et al. 2009; Henry 2011; Bradner et al. 2011; Hayatdavoodi et al. 2014). As for the studies considering periodical waves, different scaled bridge models are considered, such as 1:30 by Henry (2011), 1:25 by Sugimoto and Unjoh (2006), 1:20 by McPherson (2008), 1:5 by Bradner et al. (2011), 1:8 by AASHTO (2008), and 1:10 by Cuomo et al. (2009). For the experimental study considering solitary waves by Hayatdavoodi et al. (2014), a scale of 1:35 was employed. It is generally believed that the Froude similarity law should be valid for these physical experiments regarding the bridge deck-wave interaction problems. However, large scale experiments are considered to be more accurate. This is one reason that the laboratory study a using large scale by Bradner et al. (2011) is considered more prominent in the contributions to the engineering communities.

While most studies assumed that the bridge deck is rigidly supported, only two studies considered flexible setups of the bridge decks. In Sugimoto and Unjoh (2006), two types of bearing conditions were adopted: a fixed steel bearing and a movable steel bearing. For the movable steel bearing, rubber pads were used at the bearing area to represent the structural movability (i.e., flexibility), and the bridge model can move in both the transverse and vertical directions. The other one is the study by Bradner et al. (2011), where they developed a flexible setup using the spring between the specimen and
the end anchorage block to model different dynamic characteristics of the whole bridge. More about their experimental studies can be referred to Bradner (2008) and Schumacher et al. (2008a, 2008b). Comparisons between these two studies considering flexible setups show that the setups adopted by Sugimoto and Unjoh (2006) may not well represent the field conditions. The rubber pads cannot restrain the transverse displacement very well and the structure under this condition cannot be considered as a resilient structural system. However, in the study by Bradner et al. (2011), the spring used in the flexible setup between the specimen and the end anchorage block can be adjusted to well simulate the field conditions. The flexible setup using the spring tends to be much better than that using the rubber pads to realize the dynamic characteristics in the bridge deck-wave interaction problems. This is another reason that makes the study by Bradner et al. (2011) more valuable.

The 1:5 large scale bridge superstructure modeled in the Oregon State University (simply called the Oregon Experiment hereafter) has provided very unique information on wave-induced wave forces on bridge decks and the as-obtained experimental results are well located within the predicted force regimes by the AASHTO code (2008) using the corresponding wave and geometry parameters (Bradner 2008; Schumacher et al. 2008a, 2008b; Bradner et al. 2011). As such, the objective of the present work is to replicate this laboratory experiment using numerical simulations. By taking advantage of this precious experimental data, this comparison is to shed some lights on the numerical methodology for its capability in predicting bridge performance under wave actions. The numerical procedure, once verified, can confidently be used to study the coastal bridge performance under wave actions, or otherwise, the limitation of the numerical procedure can be demonstrated. The development of wave models based on the Stokes 1st order and 2nd order wave theory are firstly introduced and the wave models are validated with the analytical results. Then, the predicted wave forces are compared with the experimental results through the rigid setup considering six different wave heights with a fixed SWL and a fixed bridge superstructure elevation. The scale effects of Froude similarity are discussed as well. Finally, the numerical simulations of the flexible setup and the corresponding results are described and conclusions are presented.

7.2 Wave Generation and Validation with Analytical Results

Designing an experimental setup for wave-structure interaction problems is time consuming and the cost of laboratory experiments is high. Therefore, numerical approaches are becoming more attractive and are often adopted to investigate the wave-induced forces on bridges. The advantages of numerical simulations are that full scale models can be used; model geometries and positions can be conveniently adjusted; and experimental cost and time can be saved. Commercial CFD codes are under rapid development, which provides a powerful tool to investigate the wave-structure interaction problems (Huang and Xiao 2009; Xiao et al. 2010; Bozorgnia et al. 2010; Jin and Meng 2011; Bozorgnia and Lee 2012; Bricker et al. 2012; Hayatdavoodi et al. 2014). However, very few 3D simulations have been conducted until now. Bozorgnia et al. (2012) conducted 3D cases and 2D cases simulations on the bridge-wave problem and compared the numerical results with the Oregon Experiment (Bradner et al. 2011). The differences of the maximum vertical forces between the Test 1 (2D) and Test 5(3D) are only 11% for
\( H/d = 0.44 \) (\( H \) is the wave height and \( d \) is the still water depth), 6\% for \( H/d = 0.34 \), and even less for other 3 smaller ratios of \( H/d \), indicating that 2D simulations may predict relatively reasonable results (Bozorgnia and Lee 2012), especially for small \( H/d \) values. Moreover, 2D simulations can save huge computational cost. For these reasons, in the present work 2D numerical simulations are conducted for an effective, but reasonable estimation of hydrodynamic wave loadings on coastal bridge superstructures.

### 7.2.1 Wave Generation

For the laminar flow simulations, water is assumed as an incompressible, viscous fluid. The fluid motion is described based on the Navier-Stokes equations, which are shown as follows:

\[
\rho \left( \frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} \right) = -\frac{\partial p}{\partial x} + \mu \left( \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) + S_x
\]

\[
\rho \left( \frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} \right) = -\frac{\partial p}{\partial y} - \rho g + \mu \left( \frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) + S_y
\]

where \( \rho \) is the mass density, \( u \) and \( v \) are the velocity components in the \( x \) direction and \( y \) direction, respectively, \( p \) is the pressure, \( \mu \) is the viscosity, \( g \) is the gravitational acceleration, \( S_m \) is the mass source, and \( S_x \) and \( S_y \) are the momentum sources in the \( x \) direction and \( y \) direction, respectively. To account for the turbulent fluctuations in the bridge deck-wave interaction problem, the Reynolds-averaged Navier–Stokes (RANS) equations are used to describe the turbulence effects and the SST \( k-\omega \) model is used as the turbulence closure for the RANS equations.

To replicate the regular waves adopted in the Oregon Experiment, the wave steepness \( H/L^2 \) (\( T \) is the wave period) and relative depth \( H/d \) need to be considered for selecting an appropriate wave theory. According to Sarpkaya and Isaacson (1981), Stokes 3rd order wave theory is suitable for the larger wave heights (0.63 m and 0.81 m when \( T \) is 2.5 s and \( d \) is 1.89 m) used in the Oregon Experiment study, while Stokes 2nd order wave theory is proper for smaller wave heights (0.20 m, 0.34 m, 0.43 m and 0.50 m when \( T \) is 2.5 s and \( d \) is 1.89 m). Currently, the Stokes 1st order and 2nd order wave theory are developed and used in this study. The water particle velocities \( u \) and \( v \), and the free surface profile \( \eta \) of the Stokes 2nd order wave theory are expressed as follows:

\[
u = \frac{H}{2} \frac{g k \sinh k(h+z)}{\cos khh} \left( \cos(kx - \omega t) + \frac{3H^2}{16} \frac{2k(h+z)}{\sinh^4(kh)} \cos2(kx - \omega t) \right)
\]

where \( k \) is the wave number, \( \omega \) 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 with the gravitational acceleration, \( t \) is the simulation time, and \( x \) is the distance from the inlet boundary. For the current wave generation method, \( x \) should be always equal to 0 (at the inlet boundary). From Eqn. (7.2c), it is observed that the
wave crest is located at \( x = 0 \) when \( t = 0 \) s, namely, the wave crest is just located at the inlet boundary when the simulation begins. Hence, the incident Stokes waves is shifted one quarter of the time period leftward or rightward to make the water surfaces consistent with the still water level (SWL) at the beginning of simulations. The wave generation of the Stokes 1\textsuperscript{st} order wave theory follows the same approach, just simply neglecting the 2\textsuperscript{nd} order terms in Eqns. (7.2a), (7.2b) and (7.2c).

Fig. 7.1 shows the schematic diagram for the computational domain of the 2D cases (48 m in length × 2.7 m in height), where the line EF is the SWL, which separates the regions of the air phase and water phase at the initial condition. The geometry of the 1:5 scale bridge deck model (employed in Bradner et al. 2011) is introduced here firstly for the convenience of discussion, and the numerical simulations employing this bridge model will be discussed later. The width of the superstructure is 1.94 m, the girder height is 0.23 m, and the deck depth is 0.05 m. All the six girders, each one with a width of 0.06 m, were simplified as rectangles and evenly distributed. The railing effect is not considered here, being consistent with the Oregon Experiment.

The boundary conditions are specified as follows:

AB: pressure outlet. This keeps the pressure of air being the static gauge pressure that is the same as the operating pressure (101325 Pa).

AC: velocity inlet. The equations of the horizontal velocity \( u \) (Eqn. (7.2a)), the vertical velocity \( v \) (Eqn. (7.2b)) and the wave surface \( \eta \) (Eqn. (7.2c)) are expressed by the UDF and then compiled into Fluent.

CD: No slip stationary wall condition.

BD: two kinds of outlet boundary conditions are considered here. One is pressure out. The other one is no slip stationary wall condition. Since a wall condition will reflect back the incoming wave, it is necessary to set a wave-damping zone in front of this wall. A source term is needed and applied to the wave-damping zone. The source term is composed of two parts, namely a viscous dissipation term and an inertial term as (Du and Leung 2011):

\[
S_i = - \left( \frac{\mu}{\alpha} v_i + C_2 \frac{1}{2} \rho |v_i| v_i \right)
\]  

(7.3)
where $S_i$ is the source term for the $i$ th momentum equation, $|v| \leq$ the magnitude of the velocity, $\alpha$ is the permeability and $C_2$ is the inertial resistance factor. In order to damp out the wave energy without causing much reflected waves, the value of $\alpha$ should be carefully considered (Du and Leung 2011).

For the setups of the laminar flow in Fluent (academic version, v15.0), the pressure-based solver (segregated) is chosen for the transient flow, the Pressure-Implicit with Splitting of Operators (PISO) scheme is utilized for the pressure-velocity coupling method, and the PRESTO! (PREssure STaggering Option) scheme is set for the pressure spatial discretization. As a two-phase flow problem, the VOF (Volume of Fluid) method is employed to describe the dynamic free surface. The least squares cell based scheme is used for the gradient discretization, second order upwind for momentum advection terms, and Geo-Reconstruct for the volume fraction equations. For the setups of the SST $k-\omega$ model in Fluent, the turbulence damping is turned on. For the velocity inlet boundary, the turbulent intensity is set to be 2% and the turbulent viscosity ratio is 10%. For the top and outlet of the calculation domain (see Fig. 7.1), the backflow turbulent intensity and the backflow turbulent viscosity ratio are the same as that set for the velocity inlet boundary. Second order upwind is used for the spatial discretization of the turbulent kinetic energy and the specific dissipation rate. All the other setups are the same as that set in the laminar flow.

### 7.2.2 Wave Model Validation with Analytical Results

In order to acquire the turbulence effects and outlet boundary effects in the process, five methods are considered as shown in Table 7.1. It should be noted that the abbreviated letter L refers to the laminar flow model, T is for the SST $k-\omega$ turbulence model, D represents the outlet boundary as a wall boundary with a damping zone in front of this wall, and P is for the pressure outlet boundary. These abbreviated letters are shown in the figures (such as in Figs. 7.6 and 7.7 later) to represent the specific wave generation method adopted.

<table>
<thead>
<tr>
<th>Method</th>
<th>wave tank (m)</th>
<th>wave model</th>
<th>turbulence</th>
<th>outlet boundary</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>48x2.7</td>
<td>Stokes 1$^{st}$ order</td>
<td>laminar (L)</td>
<td>wall + damping zone (D)</td>
</tr>
<tr>
<td>II</td>
<td>48x2.7</td>
<td>Stokes 1$^{st}$ order</td>
<td>SST $k-\omega$ (T)</td>
<td>wall + damping zone (D)</td>
</tr>
<tr>
<td>III</td>
<td>48x2.7</td>
<td>Stokes 1$^{st}$ order</td>
<td>laminar (L)</td>
<td>pressure outlet (P)</td>
</tr>
<tr>
<td>IV</td>
<td>48x2.7</td>
<td>Stokes 1$^{st}$ order</td>
<td>SST $k-\omega$ (T)</td>
<td>pressure outlet (P)</td>
</tr>
<tr>
<td>V</td>
<td>48x2.7</td>
<td>Stokes 2$^{nd}$ order</td>
<td>SST $k-\omega$ (T)</td>
<td>pressure outlet (P)</td>
</tr>
</tbody>
</table>

Model mesh sensitivity studies have been conducted employing Method I with a wave height $H = 0.34$ m. Different mesh resolutions, $dx = 0.025$ m and 0.04 m in the $x$ direction and $dy = 0.01$ m and 0.02 m in the $y$ direction in the near water zone are used, respectively, and two time steps $dt = 0.002$ s and 0.005 s are employed here. Results show no significant differences for the wave profiles in the mesh sensitivity studies. Hence, $dx = 0.025$ m, $dy = 0.01$ m, and the time step of 0.005 s are chosen for the validation with analytical results. Namely, $dt = T/500$, $dx = L/344$ and $dy = H/34$ are used in the near water zone, where $L$ is the wave length.
Fig. 7.2 Comparisons of the free surface profiles for method I

Fig. 7.3 Comparisons of the free surface profiles for method IV and V

Fig. 7.2 shows the comparisons of the free surface profiles between the numerical results of the laminar flow (Method I) and the analytical solutions at different time when $H = 0.34$ m. Fig. 7.3 shows the comparisons of the free surface profiles of turbulence flow (Method IV and V) when $H = 0.34$ m. The results show that a good match of wave profiles can be achieved by the current method except for the region far away from the inlet boundary due to energy dissipation of the waves and the type of the outlet boundary considered. However, since the bridge model is located at around 20.97 m from the inlet boundary in the $x$ direction and a good wave profile has been developed in this region,
reasonable results can be expected with the generated waves. Therefore, while five methods are employed in the validation with the rigid setup of the Oregon Experiment, only Method I is utilized later to replicate the flexible setup of the Oregon Experiment.

7.3 Numerical Simulations of the Rigid Setup for the Oregon Experiment

7.3.1 Numerical Simulations of the Rigid Setup

The grid resolutions for the laminar flow are the same as those adopted in the wave generation discussed earlier. However, the grid resolutions for the turbulent flow are: \( dy = 0.04 \text{ m}, 0.01 \text{ m} \) and \( 0.02 \text{ m} \) for the air zone, the near water zone, and the deep water zone, respectively; \( dx = 0.02 \text{ m}, 0.01 \text{ m}, \) and \( 0.04 \text{ m} \) are for the near velocity inlet zone, main computational zone, and far field from the main computational zone, respectively. The meshes near the walls of the bridge model satisfy the requirement that the height of the first grid should be in the logarithmic layer. The total meshed cells for the turbulent flow are around 350,000 and the grid mesh in the computational domain is shown in Fig. 7.4 (a) and (b). Structured meshes are mainly used for both the laminar flow and the turbulent flow simulations.

![Grid mesh in the computational domain](image1)

![Grid mesh nearby the bridge model](image2)

Fig. 7.4 Grid mesh for the turbulent model

In the Oregon Experiment study, the data were acquired with 250 Hz (Bradner et al. 2011). Hence, the time step in the current simulation is set as 0.004 s, which also satisfies the requirements of the Courant Number. In the simulations, the waves will be reflected from the bridge deck-wave interaction and will influence the coming waves towards the bridge model and then the wave generation at the inlet boundary condition. As a result, the simulation time is chosen as 20 s to minimize the effects of the reflected waves. Fig. 7.5 shows the snapshots of the bridge deck-wave interaction for the laminar flow simulation when \( H = 0.43 \text{ m} \).

All the forces on the bridge deck model are recorded since the simulations begin. The total force component along the specified force vector \( \vec{a} \) (horizontal force or vertical force) on the wall zones of the bridge deck model is computed by summing the dot product of the pressure and viscous forces on each face with the specified force vector. The terms in this summation represent the pressure and viscous force components in the direction of the vector \( \vec{a} \) as:

\[
F_a = \vec{a} \cdot \vec{F}_p + \vec{a} \cdot \vec{F}_v
\]  

(7.4)

where \( \vec{a} \) is the specified force vector, \( \vec{F}_p \) is the pressure force vector, and \( \vec{F}_v \) is the viscous force vector. A reference pressure \( p_{ref} \) (the operating pressure, 101,325 pascal) is used to
normalize the cell pressure for computation of the pressure force to reduce the round-off error as:

$$\mathbf{F}_p = \sum_{i=1}^{n} (p - p_{ref}) \mathbf{A} \mathbf{n}$$  \hspace{1cm} (7.5)

where \( n \) is the number of faces, \( A \) is the area of the face, and \( \mathbf{n} \) is the unit vector normal to the face. The associated force coefficients (\( cl \) and \( cd \), uplift and drag force coefficients, respectively) are computed for the selected wall zones and are recorded since the simulation begins in the “Monitor” setups. The force coefficient is defined as the force divided by \( \frac{1}{2} \rho v^2 A \), where \( \rho \), \( v \) and \( A \) are the density, velocity, and area, and these values are set in the “Reference Values” in Fluent.

![Fig. 7.5 Bridge deck-wave interaction for the laminar flow simulation with the wave height 0.43 m](image)

Fig. 7.5 Bridge deck-wave interaction for the laminar flow simulation with the wave height 0.43 m

The time-histories of the horizontal and vertical forces for the five different methods of the wave generation are shown in Figs. 7.6 and 7.7, where the abbreviated letters have the same meanings as discussed in Table 7.1. The dashed horizontal line in the figures refers to the mean of the peak values for Method I in four wave periods with the corresponding wave height. Differences between the results by the five methods adopted here can be observed for both horizontal forces and vertical forces, especially for smaller wave heights (\( H = 0.20 \) m and 0.34 m). The results by the laminar flow (Method I, L+D, 1st order) tend to be larger than those by the turbulent flow at most times. However, for the results of the larger wave heights, there are no significant differences between the results by the five methods. The reason may be that the diffusion effects and higher nonlinearity (due to higher \( H/d \)) overcome the difference between the laminar flow and the turbulent flow and the difference between the 1st order and 2nd order Stokes wave.
Fig. 7.6 Comparisons of the horizontal wave forces

(a) $H=0.20$ m
(b) $H=0.34$ m
(c) $H=0.43$ m
(d) $H=0.50$ m
(e) $H=0.63$ m
(f) $H=0.81$ m
Fig. 7.7 Comparisons of the vertical wave forces
In the following discussion, only the averaged peak values of the wave forces by the typical Method I (L+D, 1st order) are compared with the corresponding ones of the Oregon Experimental results (Bradner et al. 2011) and the numerical results by Bozorgnia and Lee (2012) as shown in Fig. 7.8. It should be noted that only five wave heights, 0.34 m, 0.43 m, 0.54 m, 0.65 m and 0.84 m, are studied by Bozorgnia and Lee (2012), i.e., the wave height of 0.20 m is not considered, which is slightly different with those by Bradner et al. (2011). One interesting observation by Bradner et al. (2011) is that when the clearance between the bottom of the superstructure and the SWL is 0 and $T = 2.5$ s, a second-order polynomial relationship between the wave height and the wave forces was found as $F_h = 2.431H + 6.988H^2$ and $F_v = 18.01H + 18.69H^2$, where $F_h$ is the horizontal force and $F_v$ is the vertical force. Results of these two formulas are also plotted in Fig. 7.8.

![Fig. 7.8 Comparisons of wave forces with different wave heights](image)
Fig. 7.8 shows good agreements between the current numerical method (Method I, L+D, 1st order) and the Oregon Experiment with small ratios of \( H/d \). However, for larger wave heights \( (H=0.63 \text{ m and } 0.81 \text{ m}) \), the current method under-predicts both the horizontal forces and vertical forces. The second-order polynomial relationship between the wave height and the wave forces was not witnessed under the prescribed conditions by the current numerical method, neither by the numerical study of Bozorgnia and Lee (2012). The results by Bozorgnia and Lee (2012) are relatively larger than those predicted by the current method when \( H = 0.81 \text{ m} \), which may be due to that the Stokes 5th order waves (more appropriate to accommodate to the Oregon Experiment) are employed in their numerical studies. While for other smaller wave heights, no significant differences are found for the numerical results by the current method and the study by Bozorgnia and Lee (2012). Several causes are responsible for the differences and they are analyzed next.

Firstly, the wave profiles from the numerical simulations with high ratios of \( H/d \) may not be fully developed to the prescribed wave used in the laboratory. The effects of diffusion and nonlinearity may play important roles for high ratios of \( H/d \), and they tend to make the wave profiles to be flat. In addition, the theoretical equations of Stokes 1st order and 2nd order waves are derived from the potential flow theory based on inviscid fluid assumption. Thereby, there are limitations to the accuracy of the laminar flow and turbulent flow simulations since both flow methods consider the viscous effects. Based on current observations, when the ratio of \( H/d \) is less than 0.33, the predicted results are reasonable as compared with the experimental results. As a result, for this specific problem, limitations should be applied to the shallow/intermediate waves in the numerical simulations in order to acquire reasonable results, such as that the ratio of \( H/d \) is less than 0.33. Meanwhile, the requirements of \( H/L \) and the Ursell number corresponding to the adopted wave theory should be satisfied.

Secondly, the wave forces measured in the Oregon Experiment are directly from the six load cells as shown in Fig. 7.9, i.e., two horizontal load cells, LC1 and LC2, and four vertical load cells, LC3, LC4, LC5 and LC6. Apparently, the wave forces from the load cells include the inertia forces, \( F_{ix} \) and \( F_{iy} \) in the \( x \) and \( y \) direction, respectively. However, in the current numerical simulations, the wave forces are computed from the integration of the pressures along the whole surface of the 2D bridge model, which does not include the inertial forces. Though the inertia forces might be small for the fixed setup in most cases, they may become more significant with high wave heights. In addition, the wave forces on the bent caps, located between the specimen and the linear guide rail system, are included in the measured forces; these forces may also become more significant when the wave height is high.

Fig. 7.9 Test mechanism in the Oregon Experiment
Finally, the differences of the vertical forces may be partially attributed to the effects of the entrapped air effects. In the present 2D simulations, the entrapped air cannot escape in the direction normal to the wave propagating direction (this gives higher vertical loads with small ratios of $H/d$), while in the experiment the entrapped air may be released in this direction. In addition, the Oregon Experiment does not use end panels to consider the 3-dimensional end effects that are stated in Shih and Anastasiou (1992) and Tirindelli et al. (2003). Tirindelli et al. (2003) found significant differences in the occurrence and magnitude of loading between tests with end panels (lateral wave effects were excluded) and tests without end panels. Moreover, the 3-dimensional end effects may play more important roles when the wave height is larger.

In summary, the numerical results by the current method (Method I) agree well with the experiment study by Bradner et al. (2011) with small ratios of $H/d$. However, the current method cannot well predict both the horizontal forces and vertical forces with high ratios of $H/d$. Hence, more studies are needed for the validations of high ratios of $H/d$.

### 7.3.2 Scale Effects of Froude Similarity

It is reported that the entrapped air effects cannot be well expressed by the Froude law. One example was found in previous experimental studies considering the hydrodynamic effects on a horizontal platform (Shih and Anastasiou 1989, 1992). In their studies, Shih and Anastasiou (1989, 1992) conducted two different sets of tests on wave-induced uplift pressures on a horizontal platform. The larger wave flume is 55 m long, 2.8 m wide with a maximum operating water depth 1.2m. The smaller wave flume is 15m long, 0.30m wide with a maximum operating water depth 0.30m. The test model in the larger wave flume is about four times the model in the smaller wave flume. The test results show that while the mean pressure values from the pressure transducers tend to obey the Froude’s scaling law, discrepancies are found in the case of the maximum values. The results also reveal that the duration of the impact pressure, with the mean duration varying between 8.8 and 16.3 milliseconds, does not obey the Froude’s scaling law. While Froude similarity needs to be considered in scaled experimental studies, it is difficult and too expensive to directly verify it using full scale experiments. However, numerical results of both the prototype models and the scaled models can be more conveniently acquired to examine the Froude similarity.

For the Oregon Experiment, a 1:5 large scale bridge superstructure specimen was tested. According to the Froude law, the wave forces on the bridge superstructure (full scale) should be 125 times the wave forces on the specimen. The parameters of the specimen and the corresponding prototype bridge deck as well as the wave models adopted for these two scales (full scale and 1:5 scale) are shown in Table 7.2. The real scaling factors of the girder height, girder spacing, total weight, and total mass are slightly different from the ideal ones. However, their effects on the similitude ratios of the wave forces should be small and negligible.

In predicting the scale effects of the Froude similarity, the calculation domain is 250 m in the $x$ direction and 15 m in the $y$ direction. Numerical simulations are conducted for the target wave conditions. Because of the significant differences found in previous
comparisons with the Oregon Experiment of wave forces as shown in Fig. 7.8 when \( H = 0.81 \, \text{m} \), the scale effects of this wave height are not studied here. The values of \( F_{\text{prototype}} / F_{\text{model}} \) are plotted in Fig. 7.10, where \( F_{\text{prototype}} \) and \( F_{\text{model}} \) are the wave forces on the prototype bridge deck and the specimen, respectively.

Table 7.2 Parameters for the specimen and the corresponding prototype bridge deck

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specimen</th>
<th>Prototype</th>
<th>Ideal scaling factor</th>
<th>Real scaling factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total length</td>
<td>3.45 m</td>
<td>17.27 m</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Span length (simply supported)</td>
<td>3.32 m</td>
<td>16.64 m</td>
<td>5</td>
<td>5.01</td>
</tr>
<tr>
<td>Width</td>
<td>1.94 m</td>
<td>9.70 m</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Girder height</td>
<td>0.23 m</td>
<td>1.14 m</td>
<td>5</td>
<td>4.96</td>
</tr>
<tr>
<td>Girder spacing</td>
<td>0.37 m</td>
<td>1.83 m</td>
<td>5</td>
<td>4.95</td>
</tr>
<tr>
<td>Deck thickness</td>
<td>0.05 m</td>
<td>0.25 m</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Overall height</td>
<td>0.28 m</td>
<td>1.40 m</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Total weight (one span)</td>
<td>18.0 kN</td>
<td>2430 kN</td>
<td>125</td>
<td>135</td>
</tr>
<tr>
<td>Total mass (one span)</td>
<td>1830 kg</td>
<td>248 t</td>
<td>125</td>
<td>135.5</td>
</tr>
<tr>
<td>Wave period</td>
<td>2.5 s</td>
<td>5.59 s</td>
<td>2.236</td>
<td>2.236</td>
</tr>
<tr>
<td>Water depth</td>
<td>1.89 m</td>
<td>9.45 m</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Wave length</td>
<td>8.599 m</td>
<td>42.99 m</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Wave speed</td>
<td>3.44 m/s</td>
<td>7.69 m</td>
<td>2.236</td>
<td>2.236</td>
</tr>
<tr>
<td>wave group velocity</td>
<td>2.32 m/s</td>
<td>5.19 m</td>
<td>2.236</td>
<td>2.236</td>
</tr>
</tbody>
</table>

Fig. 7.10 Values of \( F_{\text{prototype}} / F_{\text{model}} \) considering the Froude similarity

In Fig. 7.10, the ratios of \( F_{\text{prototype}} / F_{\text{model}} \) for the vertical forces obey the Froude similarity very well with all the wave heights considered. However, small discrepancies are found for the ratios of \( F_{\text{prototype}} / F_{\text{model}} \) for the horizontal forces. As more information from field tests or the prototype model tests becomes available, the
scale effects of the Froude similarity can be further clarified and quantified. In summary, the Froude similarity is well predicted in the present study.

### 7.4 Numerical Simulation of the Flexible Setup for the Oregon Experiment

#### 7.4.1 Building up the Mass-Spring-Damper system

A mass-spring-damper system shown in Fig. 7.11 is used for a flexible setup analysis with a computational domain of 48 m long and 2.7 m high (the same as in Fig. 7.1). The boundary conditions, the wave generating method, and the parameters of the bridge model are the same as those set for Method I (L+D, 1st order) as discussed earlier. In the calculation, the mesh in the Fixed zones remains the same as its original mesh while the Remeshing zone is remeshed to generate a moving mesh using the layering mesh method. For the layering mesh method, the height based approach is chosen with the split factor 0.4 and the collapse factor 0.2. Meshes are generated or vanished along the interfaces in the domain of the Remeshing zone when the split factor or the collapse factor is satisfied.

![Remeshing zone and Fixed zone](image)

![The mass-spring-damper system](image)

**Fig. 7.11** Schematic diagram of the computational domain for the mass-spring-damper system

For the flexible setup, the bridge model shown in Fig. 7.11(b) can vibrate in the $x$ direction, where $m$ is the unit length weight, $k$ is the lateral stiffness of the bridge model, and $c$ is the damping coefficient. The motion of the bridge model can be described as the following equation:

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

(7.6)
where \( x \) is the instantaneous displacement of the bridge model in the \( x \) direction, \( \xi \) is the damping ratio, \( \omega_0 \) is the natural frequency of the bridge superstructure and \( F(t) \) is the instantaneous horizontal force integrated from the pressure along the bridge model surface. More details for the fluid-structure-interaction employing the mass-spring-damper system can be refereed in the studies by Xu et al. (2009) and Ou et al. (2009).

### 7.4.2 Determination of the Lateral Restraining Stiffness

In the Oregon Experiment, 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 (Bradner et al. 2011). Meanwhile, three different elevations of the superstructure, 3.05m, 6.10m and 9.14m (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 9.14m and pinned at the foundation, representing the most flexible configuration, was calculated as 0.88s, corresponding to a model value of 0.40s based on the Froude similarity criterion. A suitable support stiffness was chosen to match this period value in the 1:5 scale model tests. It should be noted that the flexible setup of the Oregon Experiment does not consider the damping effect.

Generally speaking, the lateral restraining stiffness, represented by the elastic springs in the Oregon Experiment, 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, shear keys, 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 the Oregon Experiment. Both cases with and without considering the damping effects are conducted.

In the Oregon Experiment, two sets of springs, medium springs and soft springs, were adopted with the corresponding fundamental periods of vibrations being 0.46s and 0.95s. In the present study, two sets of the lateral stiffness corresponding to these two fundamental periods are chosen, designated as k31 and k134 in Table 7.3. The damping coefficient (\( \xi \)) of 0.05 is used for the cases considering damping effects. In the 2D simulations, the mass is taken as 716kg per unit length according to the study by Bradner et al. (2011) (2470kg/3.45m = 716 kg per unit length).

<table>
<thead>
<tr>
<th>Cases</th>
<th>( T ) (s)</th>
<th>( m ) (kg)</th>
<th>( \xi )</th>
<th>( k ) (N/m)</th>
<th>( c ) (N·s²/m)</th>
</tr>
</thead>
<tbody>
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<td>medium springs</td>
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</tr>
<tr>
<td>k134 undamped</td>
<td>0.46</td>
<td>716</td>
<td>0</td>
<td>133574</td>
<td>0</td>
</tr>
<tr>
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<td>716</td>
<td>0.05</td>
<td>133574</td>
<td>977.9</td>
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<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>0.95</td>
<td>716</td>
<td>0</td>
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<td>0</td>
</tr>
<tr>
<td>k31 damped</td>
<td>0.95</td>
<td>716</td>
<td>0.05</td>
<td>31318</td>
<td>473.5</td>
</tr>
</tbody>
</table>

Table 7.3 Parameters for the mass-spring-damper system used in the flexible setup
7.4.3 Numerical Results

In the following discussions, the results of the dynamic characteristics of the bridge deck and the wave forces related to the flexible setup are presented in detail. Due to the reason that Bradner et al. (2011) only presented one figure (Fig. 7.12 in Bradner et al. 2011) to illustrate the results of the flexible setup and they normalized the time histories of the wave forces without giving the actual wave force and the corresponding wave height information, a direct comparison of the wave forces between the current method and the Oregon Experiment is not possible. Therefore, two wave heights, 0.34m and 0.50m, are considered here to only validate the general observations found in the Oregon Experiment.

The bridge superstructure vibrations are shown in Fig. 7.12 where all the cases start from the original position 20.97 m (bridge section center in the x direction). The structure position is recorded since the simulation begins and the total simulation time is 20 s, the same as that used in the rigid setup simulations.

![Graphs showing structural vibrations](image)

Fig. 7.12 Structural vibration

It can be observed from Fig. 7.12 that, as expected, the structure displacement considering damping effects tends to be smaller than that without considering damping
effects for most times. Another observation is that, with the increase of the wave height from 0.34 m to 0.50 m, the structure displacement becomes larger. It is noticed that when the wave height is 0.50 m, the maximum structure displacement is larger than 3 cm for the setup of soft springs, corresponding to 15 cm for the prototype bridge deck according to the Froude similarity law.

As discussed in Fig. 7.9, \( F_h \) is the horizontal force using the pressure-based method without accounting for the inertia force. However, the recorded forces in the Oregon Experiment are from load cells, LC1 and LC2. As a result, the horizontal forces presented in the Oregon Experiment for the flexible setup should be equal to \( F(t) - m\ddot{x} \), derived from Eqn. (7.6). \( F(t) - m\ddot{x} \) is the force transferred from the superstructure to the supporting bases adjacent to the bottom of the bridge deck girders. This force is denoted as “\( F_h \), soft springs” as shown in Fig. 7.13, where the comparisons of the time histories of the wave forces between the results of the rigid setup and the flexible setup (soft springs) are made, similar to the figure (Fig. 7.12) in the study by Bradner et al. (2011). Observations found in Fig. 7.13 are listed as follows:

(1) Generally speaking, the horizontal forces with considering the inertia forces in the soft springs cases tend to be much larger than those for the rigid setup, as shown in Fig. 7.13 (a) and (b). In addition, a phase lag can be observed between the maximum horizontal forces of the rigid setup and the flexible setup due to the inertia forces. Moreover, while much smaller negative horizontal forces are found for the rigid setup, significant negative horizontal forces are observed for cases with considering inertia forces. These observations were also documented by Bradner et al. (2011).

(2) For the vertical forces, because no vertical movement is allowed in the vertical direction, the recorded forces from the load cells should be equal to the wave induced vertical forces theoretically, i.e., without inertia forces. It can be observed that there is no significant difference on the aspects of quasi-static force, the same as that observed by Bradner et al. (2011). Bradner et al. (2011) also concluded that the time histories of the vertical forces from the flexible setup and the rigid setup bear much similarity with each
other. The sharp spikes shown in Fig. 7.13 may be caused by the dynamic meshing method. The pressure distribution along the walls of the bridge model may change sharply due to the different moving velocities of the bridge model in the horizontal direction attributed by the mass-spring-damper system and the wave in the dynamic meshing process.

### 7.5 Concluding Remarks

In the present study, numerical simulations have been conducted, trying to replicate the Oregon Experiment for a large-scale bridge superstructure model subjected to waves based on CFD software Fluent. The Stokes 1\textsuperscript{st} order and 2\textsuperscript{nd} order waves are employed and both the rigid setup and the flexible setup are numerically simulated. In addition, the scale effects of Froude similarity are examined. The following conclusions can be drawn:

1. The comparisons of wave forces for the rigid setup show good agreement with each other between the current method and the Oregon Experiment when the ratios of the wave height to the water depth are small. However, considerable differences are found with high ratios of the wave height to the water depth, which may be due to the effects of diffusion and nonlinearity as well as the in-viscid fluid assumption during the derivation of the theoretical equations of the Stokes wave theories. In order to achieve acceptable results for this specific problem, the ratio of $H/d$ less than 0.33 is suggested in the numerical simulations for employing the Stokes 2\textsuperscript{nd} order wave theory.

2. The Froude similarity law is examined in the current study and the vertical forces obey the Froude similarity law much better than the horizontal forces. As more information from field tests or the prototype model tests becomes available, the scale effects of the Froude similarity can be further clarified and quantified.

3. Good agreements are found in the observations obtained from the comparisons of the time histories of the wave forces between the results of the rigid setup and the flexible setup (soft springs) in both the current study and the Oregon Experiment study. Some significant observations found in the Oregon Experiment are also observed and realized in the numerical simulations. For the horizontal forces, significant negative horizontal forces are observed for cases with considering inertia forces for the flexible setup. The horizontal forces with considering inertia forces for the flexible setup tend to be much larger than those for the rigid setup. In addition, a phase lag can be found between the maximum horizontal forces of the rigid setup and the flexible setup. For the vertical forces, there is no significant difference between the rigid setup and the flexible setup.

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

1. More studies are needed for the wave generation, especially with large ratios of the wave height to the water depth. Since the predicted wave forces by the Stokes 5\textsuperscript{th} wave theory with large ratios of the wave height to the water depth (Bozorgnia and Lee 2012) are comparably smaller than the as-obtained experimental results, alternative wave theories, such as Cnoidal wave theory, need to be considered.
(2) Further observation shows that the wall condition plus a damping zone in front of it for the setup of the outlet boundary condition may be not good for long time simulations because the SWL tends to increase as the simulation time lasts and the effects of reflection may become significant. Pressure outlet boundary is tested to be a better way.

(3) Only 2D numerical simulations have been conducted. Hence, 3D models may provide more realistic results, such as the entrapped air releasing problems and the Froude similarity studies.

7.6 References


8.1 Introduction

Recently, many hurricanes strike the coastal areas along the Gulf of Mexico and refresh people’s memories with devastating damages to coastal communities, accompanied with many coastal bridges displaced or washed away. In these natural disasters, many low-lying coastal twin bridges, including the superstructure and the substructure, displayed the vulnerability to the hurricane induced high waves combined with the storm surge (Douglass et al. 2006; Robertson et al. 2007; Okeil and Cai 2008; Chen et al. 2009). The 2004 hurricane Ivan caused massive damage to the I-10 Bridge over the Escambia Bay, Florida, with 51 spans completely removed from the substructure, 33 spans displaced, and 25 bents affected by the damage of the superstructures in the eastbound (seaward) bridge. However, in the westbound (landward) bridge, the three numbers are 12, 19 and 7, correspondingly. The 2005 hurricane Katrina also brought severe damage to several coastal bridges along the gulf coasts of Louisiana and Mississippi, including the I-10 Bridge over the Lake Pontchartrain, the US 90 Bridge over the St. Louis Bay, and the US 90 Bridge over the Biloxi Bay. For the I-10 Bridge over the Lake Pontchartrain, while 38 spans were completely removed from the substructure in the eastbound (seaward) bridge, the number is 20 in the westbound (landward) bridge (Sheppard and Marin 2009).

Due to such an importance of the coastal bridges for hurricane evacuation in these extreme events and in the recovery process, many efforts have been made to reveal the failure mechanisms under wave conditions. While some experimental and numerical studies have been conducted to investigate the bridge deck-wave interaction problems for single bridge decks (Denson 1978, 1980; AASHTO 2008; McPherson 2008; Cuomo et al. 2009, Bradner et al. 2011; Henry 2011; Xiao et al. 2010; Bozorgnia and Lee 2012; Jin and Meng 2011), very few previous studies focused on the twin bridge decks under wave conditions. A comprehensive analysis of the wave forces on coastal twin bridge decks is very rare, if any. As concluded in the post disaster survey and in the lessons learned from these natural disasters, the seaward bridge bears more damage than the landward bridge and the bridge decks in the seaward bound may be more vulnerable than those in the landward bound. Therefore, more studies are needed for better understanding of the wave forces on the coastal twin bridge decks under hurricane conditions.

Useful information from the classical problems of the aerodynamic interference on two circular cylinders in the fluid mechanics can be drawn to investigate the hydrodynamic interference effects on the twin bridge decks. Carmo and Meneghini (2006) investigated the incompressible flow around pairs of circular cylinders in tandem arrangement considering different center-to-center distances and various Reynolds numbers. Alam and Zhou (2008) estimated the aerodynamic interference effects on two tandem circular cylinders with different diameters experimentally. The flow passing two tandem square cylinders were also investigated by some other researchers (Sakamoto et al. 1987; Takeuchi and Matsumoto 1992; Liu and Chen 2002). The aerodynamic interference effects on the twin bridge decks in tandem may shed more lights on the study
of wave forces on the twin bridge decks (Irwin et al. 2005; Kimura et al. 2008; Nieto et al. 2010).

The present work aims to investigate the hydrodynamic interference effects on coastal twin bridge decks under hurricane induced waves by employing numerical simulations. As such, the wave generation based on the Stokes 2nd order wave theory is firstly developed through ANSYS Fluent (V15.0, academic version) and the numerical wave profiles are compared with analytical ones. The Shear Stress Transport (SST) $k$-$\omega$ model is adopted as the turbulence closure for the RANS equations. Verifications with a large-scale bridge superstructure model (1:5 scale) by Bradner et al. (2011) and a flat plat model (1:20 scale) by McPherson (2008) are made to verify the generated waves. One typical coastal bridge deck, similar to the Escambia Bay Bridge, which was damaged during Hurricane Ivan in 2004, is chosen as the prototype bridge model to conduct numerical simulations. Then, three still water levels (SWLs) with a series of structure elevations and one fixed gap between the twin bridge decks are studied. The time histories of the wave forces are analyzed. In addition, three different SWLs for one fixed elevation of the twin bridge decks with different gaps between the twin bridge decks are further considered. Conclusions on the hydrodynamic interference effects are finally presented.

8.2 Methodology

In the present study, 2D numerical simulations are adopted. The RANS equations can be written as:

$$\frac{\partial u_i}{\partial x_i} = 0 \quad (8.1a)$$

$$\frac{\partial}{\partial t} (\rho u_i) + \frac{\partial}{\partial x_j} (\rho u_i u_j) = -\frac{\partial p}{\partial x_i} + \frac{\partial}{\partial x_j} (2\mu S_{ij} - \rho \bar{u}_i \bar{u}_j) \quad (8.1b)$$

where $\rho$ is the mass density, $u$, $p$ are the time-average value of velocity and pressure, respectively; $\mu$ is the viscosity, and $S_{ij}$ is the mean stress tensor. The small-scale fluctuations of velocity related to the turbulence, reduced as $\bar{u}_i' \bar{u}_j'$, is considered as the Reynolds stress. The Shear Stress Transport (SST) $k$-$\omega$ model is adopted as the turbulence closure for the RANS equations to account for the turbulent fluctuations in the bridge deck-wave interaction problem.

For the setups of the SST $k$-$\omega$ model in Fluent, the pressure-based solver (segregated) is chosen for the transient flow, the Pressure-Implicit with Splitting of Operators (PISO) scheme (FHWA 2009; Bricker et al. 2012) is utilized for the pressure-velocity coupling method, and the PRESTO! (PREssure STaggering Option) scheme is set for the pressure spatial discretization. As a two-phase flow problem, the VOF (Volume of Fluid) method is employed to prescribe the dynamic free surface. The least squares cell based scheme is used for the gradient discretization, second order upwind for momentum advection terms, and Geo-Reconstruct for the volume fraction equations. The turbulence damping is turned on. For the velocity inlet boundary (shown in Fig. 8.1), the turbulent intensity is 2% and the turbulent viscosity ratio is 10%. For the top and outlet of the calculation domain, the backflow turbulent intensity and the backflow turbulent viscosity ratio are the same as that set for the velocity inlet boundary. Second order
upwind is used for the spatial discretization of the turbulent kinetic energy and the specific dissipation rate.

The water particle velocities $u$ and $v$, and the free surface profile $\eta$ of the Stokes 2\textsuperscript{nd} order wave theory are expressed as follows:

$$u = \frac{Hgk \cosh k(d+z)}{2\omega \cosh kd} \cos(kx - \omega t) + \frac{3H^2\omega k \cosh 2k(d+z)}{16 \sinh^4(kd)} \cos2(kx - \omega t)$$ (8.2a)

$$v = \frac{Hgk \sinh k(d+z)}{2\omega \cosh kd} \sin(kx - \omega t) + \frac{3H^2\omega k \sinh 2k(d+z)}{16 \sinh^4(kd)} \sin2(kx - \omega t)$$ (8.2b)

$$\eta = \frac{H}{2} \cos(kx - \omega t) + \frac{H^2k \cosh(kd)}{16 \sinh^3(kd)} (2 + \cosh 2kd) \cos2(kx - \omega t)$$ (8.2c)

where $k$ is the wave number, $\omega$ is the wave frequency, $d$ 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 with the gravitational acceleration, $t$ is the simulation time, and $x$ is the distance from the inlet boundary.

Fig. 8.1 illustrates the schematic diagram for the computational domain for the 2D simulations (250 m length × 13 m height) with the line EF defined as the SWL, which separates the regions of the air and water. For the boundary conditions, the Top AB and Outlet BD are defined as pressure outlet boundary condition maintained as the atmospheric pressure (101,325 Pa), the Inlet AC is the velocity inlet and the Bottom CD is the no slip stationary wall condition. The equations of $u$ and $v$, i.e. Eqns. (8.2a) and (8.2b), are compiled into Fluent through the velocity inlet boundary by the User Defined Functions (UDF), and the free surface profile $\eta$ is controlled by Eqn. (8.2c).

![Fig. 8.1 Schematic diagram for computational domain](image)

The geometric parameters of the Escambia Bay Bridge decks with some simplifications are also shown in Fig. 8.1 for brevity purpose and the wave forces on the twin bridge decks will be discussed later. The width of the deck is 10.45 m, the girder height is 1.05 m and the deck depth is 0.3 m. All the six girders, each with a width of 0.3 m, are simplified as rectangles and evenly distributed. Bridge models with the same geometry were also studied by Xiao et al. (2010) and Huang and Xiao (2009).
8.3 Wave Model Verification

8.3.1 Wave Profile Verification with Analytical Results

Model mesh sensitivity studies are conducted using different mesh solutions: \( \Delta x = 0.1 \) m and 0.05 m in the \( x \) direction and \( \Delta y = 0.1 \) m and 0.05 m in the \( y \) direction in the near water zone, respectively. For this step, the water depth is chosen as 8.4 m with a wave height of 2.0 m and a wave period of 5.5 s. The results show no significant differences for the wave profiles in the mesh sensitivity studies. The results with a mesh of \( \Delta x = 0.05 \) m, \( \Delta y = 0.05 \) m in the near water zone and a time step of \( \Delta t = 0.005 \) s are chosen for the verification with analytical results, as shown in Fig. 8.2. In this figure, three different SWLs, 6.0 m, 7.2 m and 8.4 m, are considered with the same wave height of 2.0 m and the wave period of 5.5 s. The results show that good agreement of wave profiles can be achieved by the current method, especially near 95 m from the inlet boundary in the \( x \) direction where the bridge model will be located at.

![Fig. 8.2 Comparisons of the free surface profiles at \( t = 40 \) s](image)

8.3.2 Verification of Deck Forces with a Large Scale Bridge Superstructure Model

For the verification with a large-scale bridge superstructure model by Bradner et al. (2011), the chosen computational domain is 48 m in length and 2.7 m in height with the SWL 1.89 m. Structured meshes are mainly adopted and the grid resolutions are: \( \Delta y = 0.04 \) m, 0.01 m and 0.02 m for the air zone, the near water zone, and the deep water zone, respectively; \( \Delta x = 0.02 \) m, 0.01 m, and 0.04 m for the near velocity inlet zone, main computational zone, and far field near the outlet boundary zone, respectively. The near wall meshes of the bridge model satisfy the requirement that the height of the first grid should be in the logarithmic layer. The total meshed cells are around 350,000. The width of the superstructure is 1.94 m, the girder height is 0.23 m, and the deck depth is 0.05 m.
All the six girders, each one with a width of 0.06m, were simplified as rectangles. The grid mesh in the computational domain is shown in Fig. 8.3 (a) and (b).

![Grid mesh in the computational domain](image1)

(a) Grid mesh in the computational domain

![Grid mesh nearby the bridge model](image2)

(b) Grid mesh nearby the bridge model

Fig. 8.3 Grid mesh for the verification with a large scale bridge superstructure model

The time step is set as 0.004 s to accommodate with the experimental study by Bradner et al. (2011). Four wave heights, 0.34 m, 0.43 m, 0.50 m and 0.63 m, are chosen here as examples to conduct the comparisons between the current method and the experimental study. The time histories of the horizontal and vertical forces are recorded by the “Monitor” in Fluent and shown in Figs. 8.4 and 8.5. The mean values of the peak forces by the current method and the experimental study by Bradner et al. (2011) are also shown in these two figures. Due to that the current method is based on 2D numerical simulations, limitations should be noticed in the verification with the experimental study and they are discussed as follows.

For the horizontal forces, good agreements are found between the current method and the experimental study with small ratios of $H/d$. However, with the increase of the wave height, the difference becomes larger. The critical reasons for this phenomenon are believed to include: (a) the 3D effects (discussed by Tirindelli et al. (2003)) were not excluded in the experimental study by Brander et al. (2011); (b) the inertial forces were considered by the load cells in the experimental setup; (c) the simplified rectangle girders may have shortcomings as compared with the “I” type girders; (d) additional wave forces may be counted in, such as the wave forces on the bent caps, located between the specimen and the linear guide rail system, as well as the wave forces on the load gages in the experimental setup. These factors may also become more significant when the wave height is high, especially the 3D effects. In addition, it may not be suitable to consider the case with high wave height in the experimental study as a 2D case in numerical simulations as many features may not be captured, such as counting in the diaphragms, choosing the more suitable wave theories, and the release of the entrapped air.
For the vertical forces, good agreements are observed when the wave height is 0.43 m and 0.50 m. Higher vertical forces are predicted with the wave height of 0.34 m by the current method, which may be due to the entrapped air effect. The entrapped air
cannot escape in the normal direction in a timely manner in the current simulations, and the vertical forces induced by the entrapped air may be of a larger portion of the total vertical forces when the wave height is small. However, smaller vertical forces are recorded when the wave height is 0.63 m. The reason may partially be due to the limitations of simulating the shallow/intermediate water waves with high wave heights because of the diffusion and nonlinearity effects.

8.3.3 Deck Force Verification with a Flat Deck Model

While the study by Bradner et al. (2011) was focused on a girder bridge deck, McPherson (2008) considered periodic waves on a 1:20 scaled flat deck and the result is used here for another verification study. The experiment was conducted in the Haynes Coastal Engineering Laboratory 3D shallow water wave basin at the 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.

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 × 0.9 m in height. Based on the mesh sensitivity studies, the grid resolutions for this verification are: \(\Delta y=0.02\) m, 0.0025 m and 0.005 m for the air zone, the near water zone, and the deep water zone, respectively; \(\Delta x=0.005\) 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. The grid mesh in the computational domain is shown in Fig. 8.6.

![Fig. 8.6 Grid mesh for the verification with a flat deck model](image)

Comparisons between the results by the current method and by McPherson (2008) are shown in Fig. 8.7. As seen in Fig. 8.7, a small difference between the maximum horizontal forces is found when \(d=0.48\) m. The reason may due to that there are difficulties to keep the same wave height as 1.06m in a direction normal to the wave propagation. However, good agreements are witnessed for the verifications with a flat deck, indicating that the current method can be further utilized.
Fig. 8.7 Comparisons between the time histories of the vertical forces for the verification with a flat deck

8.4 Wave Forces with a Fixed Deck Gap of 20m

The clearance between the eastbound and westbound of the Escambia Bay Bridge is 19.8 m (65 feet). Hence, a fixed deck gap of 20 m is chosen in this section to analyze the wave forces on the twin bridge decks. Three different SWLs, 6.0 m, 7.2 m and 8.4 m, with various structure elevations are considered. Comparisons between the wave forces on a single bridge deck and the twin bridge decks are made and analyzed with the parameters listed in Table 8.1. The case name of “SWL 6.0 m” refers to the SWL of 6.0 m at the initial condition defined in the computational domain. The submersion coefficient $C_s$ is defined as the ratio of $S$ to $H_b$, where $S$ is the distance between the bottom of the superstructure to the SWL (negative if the structure is submerged in the water) and $H_b$ is the height of the bridge superstructure. The bridge elevation refers to the elevation of the bottom of the superstructure.

Only one wave height of 2.0 m is considered here for all the simulations to demonstrate the hydrodynamic interference effects. The calculation time is 45 s and the time step is 0.005 s. The grid resolutions are shown in Fig. 8.8: $\Delta x=0.05$ m and $\Delta y=0.05$ m for the zone nearby the bridge model; $\Delta x=0.2$ m and $\Delta y=0.05$ m for the near water surface zone at the far field from the bridge model; $\Delta x=0.2$ m and $\Delta y=0.1$ m for the deep water zone, and $\Delta x=0.2$ m and $\Delta y=0.2$ m for the air zone at the far field from the bridge model. Again, the meshes near the walls of the bridge model satisfy the requirement that the height of the first grid should be in the logarithmic layer.
Table 8.1 Cases considered with different submersion coefficient and SWLs

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<th>Bridge elevation (m)</th>
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<td></td>
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<td>6.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.222</td>
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</tr>
<tr>
<td>SWL 6.0m</td>
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<td>6.0</td>
</tr>
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<td></td>
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<td>5.7</td>
</tr>
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<td></td>
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(a) Grid mesh in the computational domain
(b) Grid mesh nearby the bridge

Fig. 8.8 Grid mesh adopted for the twin bridge deck
8.4.1 Comparison between Single Bridge Deck and Twin Bridge Deck

The comparisons between the time histories of the wave forces on a single bridge deck and the twin bridge decks with the SWL of 8.4 m and the bridge elevation of 8.1 m (gap=20 m) are shown in Fig. 8.9 to demonstrate the hydrodynamic interference effects (gap=20 m for Figs. 11-14 in the following discussion). As shown in Fig. 8.9, there is a phase difference between the peak wave forces on the seaward deck and the landward deck as expected. It is observed that during the first wave period among the shown four wave periods, the wave forces on the seaward deck are almost equal to those on the single bridge deck. However, during the following three wave periods, the horizontal forces on the single bridge deck are larger than those on the seaward deck and it is opposite for the vertical forces. The reason may mainly due to the reflected waves from the landward deck. The letters, such as, (b) and (c), labelled in Fig. 8.9 refer to the snapshots taken from the numerical simulations, and these snapshots for the single bridge deck and the twin bridge deck are shown in Figs. 8.10 and 8.11, respectively. It should be noticed that the letter (a) stands for the starting point, i.e., 0 s, in these two figures.

![Fig. 8.9 Comparisons of the wave forces on a single bridge deck and the twin bridge decks](image)

The averaged peak wave forces in the four wave periods (20s-45s, same as shown in Fig. 8.9) on the single bridge deck and the seaward deck are compared in Fig. 8.12. As shown in Fig. 8.12(a), while the horizontal forces on the single bridge deck are relatively larger than those on the seaward deck when the submersion coefficient is smaller than 0, the horizontal forces on the single bridge deck are smaller for most times when the submersion coefficient is larger than 0. However, the vertical forces on the single bridge deck are smaller than those on the seaward deck, as shown in Fig. 8.12(b).
Fig. 8.10 Snapshots for the single bridge deck-wave interaction

Fig. 8.11 Snapshots for the twin bridge decks-wave interaction

Fig. 8.12 Comparisons of the averaged peak wave forces on the single bridge deck and the seaward deck
The interference effects of the landward deck may hinder the wave group velocity to some extent, and the superposition of coming waves and the reflected waves may cause the horizontal wave forces smaller on the seaward deck when the submersion coefficient is less than 0, i.e., the bottom of the bridge girder is below the SWL. When the bottom of the bridge girder is above the SWL, the reflected waves may cause less negative horizontal forces (reverse to the wave propagation direction). However, the superimposed waves may exert larger vertical wave forces on the seaward deck.

### 8.4.2 Comparison of Wave Forces on the Seaward and Landward Bridge Decks

The time-histories of horizontal and vertical forces for the three different SWLs when the submersion coefficient $C_s$ is 0 are shown in Figs. 8.13 and 8.14, respectively. In Fig. 8.13, the positive horizontal forces on the seaward deck are larger than those on the landward deck. It is similar for the positive peak vertical forces as shown in Fig. 8.14. The negative peak horizontal and vertical forces are not considered here because they are much smaller than the corresponding positive ones on both the seaward deck and the landward deck.

![Fig. 8.13 Time-history of horizontal forces for different SWLs for $C_s = 0$](image)

### 8.4.3 Hydrodynamic Interference Effects of Twin Bridge Decks

The averaged positive peak horizontal and vertical forces from the time-history wave forces are shown in Fig. 8.15. Generally speaking, the wave forces on the seaward deck are larger than those on the landward deck for all the cases studied, which can also be observed in the time-history wave forces as shown earlier in Fig. 8.13 and 8.14. As for different SWLs, small differences are found for both the horizontal and vertical forces on
the landward deck. The wave forces on the seaward deck vary with different SWLs; however, they follow the same general trends.

As shown in Fig. 8.15(a), while the horizontal forces on the landward deck increase with the decrease of the submersion coefficient (more submerged), the horizontal forces on the seaward deck do not follow the same pattern as those on the landward deck. There is a big difference between the horizontal forces on the seaward deck and the landward deck when the submersion coefficient is 0, i.e., the bottom of the girder is just at the SWL. For the vertical forces as shown in Fig. 8.15(b), the maximum vertical force occurs on both the seaward deck and the landward deck when the bottom of
the girder is near the SWL, and this observation is also documented by Bradner et al. (2011).

Normalized expressions, $F_{\text{h}_{\text{landward}}}/F_{\text{h}_{\text{seaward}}}$ and $F_{\text{v}_{\text{landward}}}/F_{\text{v}_{\text{seaward}}}$, are presented here in Fig. 8.16 to show the interference effects. $F_{\text{h}_{\text{landward}}}$ and $F_{\text{v}_{\text{landward}}}$ are the averaged values of the peak wave forces on the landward deck for one specific case, and $F_{\text{h}_{\text{seaward}}}$ and $F_{\text{v}_{\text{seaward}}}$ are the corresponding averaged values of the peak wave forces on the seaward deck for the same case.

The ratios of $F_{\text{h}_{\text{landward}}}/F_{\text{h}_{\text{seaward}}}$ are smaller than 0.8 for all the cases studied, especially that they are smaller than 0.5 when the submersion coefficient is positive. For the vertical force cases, the ratios of $F_{\text{v}_{\text{landward}}}/F_{\text{v}_{\text{seaward}}}$ are smaller than 0.7, and decrease with the increase of the submersion coefficient (more subaerial). Based on the cases studied with one fixed gap (20m) between the twin bridge decks, it can be concluded that much less wave forces are on the landward deck than those on the seaward deck when the bottom of the girder is above the SWL. However, when the bottom of the girder is submerged in the water, for example, when the submersion coefficient is -0.889, factors of 0.8 and 0.7 for the horizontal forces and vertical forces, respectively, can be taken to consider the hydrodynamic interference effects.

![Normalized expressions](image)

**Fig. 8.16 Wave force normalization with a deck gap of 20m**

### 8.5 Effects of Deck Gaps on Wave Forces

The deck gaps may vary according to the construction environment, structural type, the investment and so on. For example, the Biloxi Bridge damaged during Hurricane Katrina bears a very small clearance between the twin decks. Hence, in this section, three SWLs (7.2 m, 7.8 m and 8.4 m) are considered and one fixed bridge elevation (7.2 m) with different deck gaps (1 m, 2 m, 3 m, 5 m, 8 m, 9 m, 10 m, 11 m, 12 m, 15 m and 20 m) between the twin bridge decks are studied. The wave height, the calculation time and the time steps for the studied cases are the same as those discussed above.
Three examples of the time-history wave forces and the corresponding snapshots from the numerical simulations are selected to demonstrate the effects of different gaps on the wave forces, and they are shown in Figs. 8.17-8.22. For these three examples, the SWL is 7.8 m, the bridge elevation is 7.2 m, and the submersion coefficient is -0.444. The three gaps between the twin bridge decks are chosen as 1 m, 10 m, and 20 m, respectively. The letters, (a) to (f), shown in the time histories of the wave forces refer to the corresponding snapshots for each example.

The wave forces on the twin bridge decks with the gap of 1m are shown in Fig. 8.17, accompanied with the corresponding snapshots as shown in Fig. 8.18. In general, the wave forces on the seaward deck are larger than those on the landward deck. Due to the seaward deck-wave interaction, the horizontal forces on the landward deck do not show the general force profiles as those on the seaward deck or on the single bridge deck. Instead, the horizontal forces on the landward deck last for a long duration time. In Fig. 8.18, the water movement within the deck gap is of much turbulence, and the water jumps onto both nearby deck surfaces for snapshots (a) and (f), i.e. the moment the wave front striking the seaward deck. Green water on both decks is observed. While the green water on the seaward deck mainly comes from the impinging waves with a relatively large amount, the green water on the landward is from the water jump within the deck gap with a small amount.

![Graph showing wave forces](image)

**Fig. 8.17 Time-history wave forces on the twin bridge decks with a deck gap of 1m**

The results for the twin bridge decks with the gap of 10m are shown in Figs. 8.19 and 8.20. In this case, the horizontal forces on the seaward deck are larger than those on the landward deck; however, the vertical forces on both bridge decks bear the same magnitude.
Fig. 8.18 Snapshots of the twin bridge decks-wave interaction with a deck gap of 1 m

For snapshot (a) in Fig. 8.20, the wave front impacts the seaward deck with a large quantity of water overrunning on the seaward deck and at this moment, the wave forces almost reach its peak values. For snapshot (c) in Fig. 8.20, the wave front comes to the landward deck with a small amount of water overrunning on the landward deck due to the energy dissipation of the seaward deck-wave interaction. When another wave front comes to the seaward deck, the reflected wave from the landward deck reaches the seaward deck too, as shown in snapshot (f) in Fig. 8.20. As a result, the interference...
effects cause the vertical forces on the seaward deck to show the same magnitude as on the landward deck.

It can be further interpreted that the wave leaves the seaward deck, is reflected back from the landward deck, and then reaches the seaward deck with a travelling distance (20 m) of about half wave length. Under the condition with the known parameters ($d=7.8$ m, $T=5.5$ s and $H=2$ m), the wave length is about 40 m for the Stokes 2nd order wave theory. Hence, the superposed waves may weaken the vertical forces on the seaward deck.

Similarly, the results for the twin bridge decks with the gap of 20 m are shown in Figs. 8.21 and 8.22. With such a deck gap, the wave forces, including both the horizontal and vertical forces, on the seaward deck are larger than those on the landward deck. The bridge deck-wave interaction for the seaward deck is stronger than the landward deck, as can be observed from the snapshots in Fig. 8.22. While the water underneath the seaward deck has much turbulence, there is not much turbulence for the water underneath the landward deck.

![Fig. 8.20 Snapshots of the twin bridge decks-wave interaction with a gap of 10 m](image)

![Fig. 8.21 Time-history wave forces on the twin bridge decks with a gap of 20 m](image)
Fig. 8.22 Snapshots of the twin bridge decks-wave interaction with a deck gap of 20 m

For snapshot (f) in Fig. 8.22, reflected wave is also observed as it reaches the seaward deck with a travelling distance of about one wave length. However, the vertical forces on the seaward deck are much larger than those on the landward deck. As a result, the superimposed waves may strengthen the vertical forces at this moment.

The averaged wave forces for the three SWLs with different deck gaps are shown in Figs. 8.23 and 8.24. Generally speaking, the horizontal forces on the seaward deck are larger than those on the landward deck at most times. The general trends vary with the increase of the deck gap. When the bottom of the girder is just above the SWL (cases of SWL 7.2 m), the horizontal forces on the seaward deck and the landward deck tend to become constant when the gap is larger than 10m. When the gap is smaller than 10m, the interference effects should be taken into consideration. However, for submerged cases (cases of SWLs 7.8 m and 8.4 m), the interference effects play a significant role in the twin bridge decks-wave interaction. The horizontal forces on the seaward deck are much larger than the corresponding forces on the landward deck when the deck gap is around 10 m.

Fig. 8.23 Horizontal forces versus different deck gaps
Fig. 8.23 (continued) Horizontal forces versus different deck gaps

(c) SWL 8.4 m

Fig. 8.24 vertical forces versus different deck gaps

(a) SWL 7.2 m

(b) SWL 7.8 m

(c) SWL 8.4 m
For the vertical forces, generally speaking, the wave forces on the landward deck do not change too much along with the changing of the deck gaps. However, a concave shape for the vertical forces on the seaward deck with the variable gaps is observed for the three SWLs studied. The wave lengths under the three SWLs are around 40 m and vary slightly with different SWLs. As such, the vertical forces drop to relative small values when the gap is around 10m due to the interference effects of the superimposed waves.

In order to estimate the general law of the hydrodynamic interference effects on the twin bridge decks with various deck gaps, the interference factors are defined as:

\[ IF_h = \frac{F_{h\text{ seaward}}}{F_{h\text{ single}}} \quad \text{or} \quad \frac{F_{h\text{ landward}}}{F_{h\text{ single}}} \]  \hspace{1cm} (8.3a)

\[ IF_v = \frac{F_{v\text{ seaward}}}{F_{v\text{ single}}} \quad \text{or} \quad \frac{F_{v\text{ landward}}}{F_{v\text{ single}}} \]  \hspace{1cm} (8.3b)

where \( IF_h \) and \( IF_v \) are the hydrodynamic interference factors for the horizontal and vertical forces, respectively, on the seaward deck or the landward deck; \( F_{h\text{ seaward}} \) and \( F_{h\text{ landward}} \) are the averaged positive peak horizontal forces on the seaward deck and landward deck, respectively; \( F_{v\text{ seaward}} \) and \( F_{v\text{ landward}} \) are the averaged peak vertical forces on the specific decks; \( F_{h\text{ single}} \) and \( F_{v\text{ single}} \) are the corresponding horizontal and vertical forces on the single bridge deck. The obtained factors are plotted in Fig. 25.

For the horizontal forces as shown in Fig. 8.25 (a), the interference factor, \( IF_h \), for the seaward deck can be taken as 1 when the bottom of the superstructure is just at the SWL (case SWL 7.2m). When the superstructure is partially submerged (cases SWL 7.8m and 8.4m), the \( IF_h \) for the seaward deck with the deck gap around 10m can reach up to 1.32 due to the superimposed wave effects. When the deck gap is less than 10m, the horizontal forces on the landward deck can be conservatively taken as the same values as those on the single bridge deck. However, when the deck gap is larger than 10m, a value of 0.7 can be considered as the interference factor.

While for the vertical forces as shown in Fig. 8.25 (b), the \( IF_v \) for the seaward deck tends to be larger when the deck gap is close to 1m than that when the deck gap is 20m, with the factor up to 1.55. When the deck gap is around 10m, the \( IF_v \) for the seaward deck can be conservatively taken as 0.85. As for the landward deck, the interference factor, \( IF_v \), can be taken as 1.0 when the deck gap is less than 10m and 0.80 when larger than 10m.

### 8.6 Concluding Remarks

Based on the prescribed conditions concerning wave forces due to hurricane waves on coastal twin bridge decks, the hydrodynamic interference effects are studied and conclusions can be drawn as follows:

1. Comparisons of the averaged peak wave forces on the single bridge deck and the seaward deck show that it is of significant importance to investigate the wave forces on twin bridge decks instead of one single bridge deck.

2. Based on the cases studied with one fixed gap (20 m) between the twin bridge decks, much less wave forces are exerted on the landward deck than those on the seaward
deck when the bottom of the girder is above the SWL. For submerged cases, factors of 0.8 and 0.7 for the horizontal and vertical forces, respectively, can be taken to consider the hydrodynamic interference effects.

Fig. 8.25 Hydrodynamic interference factors of horizontal and vertical forces

(3) For the cases considering different gaps, the superimposed waves (consisting of the coming wave and the reflected wave) may weaken the vertical forces when the reflected wave travels a distance of about half wave length from leaving the seaward deck to arriving at the seaward deck the second time, and may strengthen the vertical forces when the reflected wave travels about one wave length from leaving the seaward deck to arriving at the seaward deck the second time. However, the superimposed waves may play a more significant role for horizontal forces when the twin bridge decks are partially submerged than that when the twin bridge decks are just above the SWL.
The limitations of the current study and future work are described as follows: (1) The observations found in this study are based on numerical simulations. In order to acquire more accurate and reliable results, more studies, especially experimental studies, are very necessary. (2) In the present study, 2D numerical simulations have been conducted. The bridge models are simplified without considering the railing and the diaphragm. 3D models may provide more reliable results to consider these features. However, 3D models may be much more computationally costly. (3) In this study, limited scenarios have been considered with waves propagating normal to the bridge models. As such, more cases, considering different wave heights, bridge geometries and attack angles, need to be studied. (4) Wave forces on the twin bridge decks under random waves and oblique waves need to be further studied.

8.7 References


CHAPTER 9. CONCLUSIONS AND FUTURE STUDIES

9.1 Summary and Conclusion

In this dissertation, the wave forces due to solitary waves and Stokes waves on typical coastal bridge decks are analyzed with variable parameters. The contribution of this dissertation is mainly suggesting an improved method that can be used in the design stage, giving a better understanding of the solitary wave forces on the bridge decks based on an component level approach and the countermeasure of the air venting holes, providing a deeper insight of the wave forces on the bridge decks with different inclinations and those with different restraining stiffnesses, and demonstrating a general observation for the wave forces on the twin bridge decks.

9.1.1 Suggestion of an Improved Method

An improved method for calculating the solitary wave forces on the bridge decks based on the reviewed studies is suggested and is proven to be a practical and simple way to assess the wave forces, though slightly conservative results are predicted:

(a) The velocity related force (named the hydrodynamic force) is considered by taking into account the effects of wave periods, wave types, and water particle velocities near the structures, though not in a straightforward way. As such, more realistic results can be expected.

(b) The overtopping water on the bridge deck and the water at the trailing edge of the bridge deck are properly considered.

(c) The gauge pressure at the approximate time when the positive peak vertical force occurs shows that taking the same pressure \( \gamma \delta Z \) on the whole projected area \( A_v \) may overestimate the force component of \( F_{\text{hydrostatic}} = \gamma \delta Z A_v - F_w \).

9.1.2 Better understanding of the Wave Forces based on Component Level Analysis and the Countermeasure of Air Venting Holes

The results based on the parametric study shows that for each wave height with viable submersion coefficient, the maximum horizontal forces occur when the bridge superstructure is just fully submerged and the maximum vertical ones occur when the bottom of the bridge superstructure is at around the SWL. The positive moments tend to be larger than the negative moments for one specific case before the bridge superstructure is fully submerged; however, it goes to the opposite when the bridge superstructure is fully submerged.

When considering the maximum wave forces with different still water depths, interesting phenomena were found:

(a) The positive horizontal forces at lower water depth are larger than those at higher water depth. This is probably due to the reason that the horizontal water particle velocities at the crest section at lower water depth are larger than those at higher water depth;
(b) The uplift forces at higher water depth are larger than those at lower water depth before the superstructure is fully submerged, while the uplift forces at higher water depth are smaller than those at lower water depth when fully submerged;

(c) The positive and negative moments at lower water depth are larger than the corresponding ones at higher water depth.

Component level based analysis of the wave forces are conducted with three typical cases. The time histories of the wave forces on each component will enable engineers to acquire characteristics of wave forces at localities.

As for the countermeasure of the air venting holes, two observations are documented as follows:

(a) The venting ratio of 1% with five evenly distributed venting holes based on the whole area is enough to mitigate the vertical forces on the bridge decks;

(b) The hydrostatic force due to the entrapped air contributes a larger portion of the total uplift force when the wave height is smaller, while the portion of the hydrodynamic force in the total uplift force increases with the increase of the wave height.

9.1.3 Deeper Insight of the Wave Forces on the Bridge Decks with Inclinations and Restraining Stiffness

Generally speaking, the wave forces on the inclined bridge decks under solitary waves are affected by the following factors: wave heights, bridge deck inclinations, and the relative position between the wave peak (the height of water depth plus the wave height) and the top of the seaward girder. The wave heights and bridge deck inclinations play more significant roles than the other factors. The normalized ratios of the horizontal and vertical forces increase as the bridge deck inclinations increase, but with different increment rate. The ratios of the moments do not have consistent trends for the studied seven elevations.

For the bridge deck-wave interaction, the lateral restraining stiffness is taken into account by introducing the mass-spring-damper system. Based on the numerical simulations of the solitary waves on a typical coastal bridge deck, interesting phenomena are witnessed:

(a) As expected, the more flexible the structure is, the larger the structure displacement is. Big difference can be found for the horizontal forces with and without considering inertia forces when the structure is flexible.

(b) The horizontal forces with considering inertia forces should be considered and applied in practical designs. While if the period of the structure vibration is less than 1.0s, the difference can be very small and the effects of the bridge deck-wave interaction on the flow field can be neglected.

(c) The time-history of the horizontal force and the structure displacement have the same trend, which indicates that the horizontal force is coupled and interacted with the structure vibration.
9.1.4 General Observation for the Wave Forces on the Twin Bridge Decks

For the results regarding to the solitary wave forces on the twin bridge decks based on a parametric analysis, the observations are listed as follows:

(a) For the wave forces considering different submersion coefficients and different SWLs, the positive peak horizontal forces with a lower SWL are larger than those with a higher SWL on both decks with the same submersion coefficient. However, the positive peak vertical forces with a lower SWL are smaller than those with a higher SWL on both decks with a specific range of the submersion coefficient for each deck.

(b) The wave forces on the seaward deck of the twin bridge deck are of the same magnitude with those on the single bridge deck, indicating that the presence of the nearby structure, the landward deck, has negligible effects on the wave forces on the seaward deck.

(c) For the cases studied with a 20 m deck gap, a factor of 0.9 for both the horizontal forces and the vertical forces on the landward deck is suggested for practical engineering activities. For the wave forces considering different deck gaps with a fixed elevation (7.2 m) of the bridge superstructure, the wave forces on the seaward deck are larger than those on the landward deck for almost all the cases. The wave forces on both decks changes more significantly when the gap ranges from 1m to 5m, and the wave forces on the landward bridge deck should better be taken the same as those on the seaward bridge deck when the deck gap falls in this range. Factors of 0.8 and 0.9 for the horizontal forces and the vertical forces, respectively, on the landward bridge deck are suggested when the deck gap is larger than 5m.

The wave forces on the twin bridge decks due to Stokes waves demonstrate different characteristics from those due to solitary waves:

(a) It is of significant importance to investigate the wave forces on twin bridge decks instead of one single bridge deck since there is much difference between the averaged peak wave forces under these two conditions.

(b) For the cases with one fixed gap (20 m) between the twin bridge decks, much less wave forces are exerted on the landward deck than those on the seaward deck when the bottom of the girder is above the SWL. For submerged cases, factors of 0.8 and 0.7 for the horizontal and vertical forces, respectively, can be taken to consider the hydrodynamic interference effects.

(c) For the cases considering different gaps, the superimposed waves (consisting of the coming wave and the reflected wave) may weaken the vertical forces when the reflected wave travels a distance of about half wave length from leaving the seaward deck to arriving at the seaward deck the second time, and may strengthen the vertical forces when the reflected wave travels about one wave length from leaving the seaward deck to arriving at the seaward deck the second time. However, the superimposed waves may play a more significant role for the horizontal forces when the twin bridge decks are partially submerged than the case when the twin bridge decks are just above the SWL.
9.2 Future Studies

The bridge deck-wave interaction under tsunamis and hurricane waves is at its early stage of research and further studies are needed to gain a more comprehensive understanding of the wave loadings on the bridge decks and the corresponding mitigation methods:

(a) For the 2D numerical simulations, the results are more reasonable for cases that the ratio of the hydrostatic uplift force due to the entrapped air to the total uplift forces is small and the cases when the bridge models are fully submerged. When the ratios are larger, say, in some cases that the bridge deck is located well above the SWL, 3D models are expected to provide more reliable results since the entrapped air can flow in the longitudinal direction.

(b) Larger ratios of the wave height to the water depth need to be further studied. Waves that are close to the breaking and in the breaking need to be paid attention in cases that the bridges located near the shore may experience such kind of waves in their service life.

(c) Waves with different attacking angles need to be considered, though the induced wave forces are smaller than those induced by the waves propagating normal to the length of bridge decks.

(d) The countermeasure of the air venting holes produces much more favorable results when the bridge decks are located at around the SWL when compared with the other two countermeasures, elevating structures and rigidifying structures. However, the impact pressure due to the rapid rising water inside the chamber may induce noticeable local damage and result in a subsequent total failure of the bridge superstructure. The degree of the local damage is of stochastic characters and needs further investigations.

(e) General observations of the characteristics of the wave forces on the twin bridge decks are documented. More studies, especially experimental studies, are needed to provide more concrete and accurate results.
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Mr. Guoji Xu was born in Henan Province, People’s Republic of China. He received his bachelor and Master of Science degrees from the Department of Civil Engineering, Hunan University, P.R. China, in 2007 and 2010, respectively. From June 2010 to December 2010, Mr. Xu worked as a structural engineer in WISDRI Engineering & Research Inc., Wuhan, China. He has worked as a Graduate Research Assistant at Louisiana State University since August 2011.

The degree of Doctor of Philosophy will be awarded to Guoji Xu at the August 2015 commencement.

As a Graduate Research Assistant, Mr. Xu has done research in the area of wave induced forces on coastal bridge decks. As a culmination of his research, he has 1 journal paper published and 2 journal papers under review, which are listed as follows:

**Journal Papers**