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Fatigue life estimation of existing bridges under vehicle and non-stationary hurricane wind

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ABSTRACT

During hurricane events, additional damages to existing bridge details might be accumulated from the large stress cycles generated by the non-stationary wind and dynamic vehicle loads. The environmental corrosion could also reduce the structural fatigue strength. Based on the non-stationary wind field modeling techniques and the vehicle–bridge–wind dynamic analysis framework, this paper initiates a fatigue damage assessment of existing bridges with corrosion under vehicle and non-stationary extreme hurricane wind loads. The non-stationary hurricane wind is simulated as a summation of time-varying mean and fluctuating non-stationary components with a time-varying spectrum. Dynamic stress histories are obtained by solving the vehicle–bridge–wind dynamic system. The rain flow counting method is used to obtain the stress ranges and numbers of stress cycles. The corrosion induced fatigue strength degradation is included in the analysis. Numerical examples of an existing long span bridge under simulated non-stationary hurricane winds and vehicles are presented to demonstrate the effects of the dynamic loads and corrosions on the fatigue life estimation of existing bridges.

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1. Introduction

Hurricanes could bring strong winds and large precipitation and are the most financially devastating natural hazards for coastal regions along the Gulf and Atlantic seaboards. According to a report from National Oceanic and Atmospheric Administration (NOAA) (Blake et al., 2005), the average hurricane number per decade is 17.7 for all categories and 6 for major hurricanes with categories from 3 to 5 in the US. For a bridge with a life expectation of 100 years, several hurricanes including some major ones could possibly affect the structure depending on the bridge location. The hurricane records also show an upswing of the maximum wind speed and the duration of hurricanes (Emanuel, 2011). Modeling of the wind during the extreme hurricane events could be complicated and has been extended to multi-variant and multi-dimensional wind fields involving Gaussian and non-Gaussian, stationary and non-stationary, conditional and unconditional random processes (Kareem, 2008). The high sustained hurricane wind could interact with the vehicle–bridge dynamic system in a short period of time and lead to possible large stress cycles with potential damage accumulations.

In the vehicle–bridge system, the vehicles were firstly modeled as moving masses or moving forces in 1970s (Blejwas et al., 1979; Timoshenko et al., 1974). Later, the vehicles were modeled as a combination of several rigid bodies connected by several axle mass blocks, springs, and damping devices and a vehicle–bridge dynamic system was established (Guo and Xu, 2001; Shi et al, 2008; Zhang and Cai 2012). For the long-span bridges, Cai and Chen (2004) built a framework of vehicle–bridge–wind dynamic analysis. In that framework, the wind loads were modeled with time-dependent and time-independent components and the equations of motions of the vehicle–bridge–wind dynamic system were updated at each time step to simulate their interactions. Recently, the dynamic analysis has been extended from analyzing the dynamic displacements and accelerations to dynamic stress ranges and fatigue damages (Xu et al., 2009; Chen and Wu, 2010; Chen et al., 2011a, 2011b; Zhang et al., 2013).

However, in the previous vehicle–bridge dynamic system, corrosions of the structural components were not considered. For those bridges that have served for several decades, the corrosions could possibly have large effects on the structural damage and reliability assessment. According to Federal Highway Administration, 35% of the nation's bridges were classified either as structurally deficient or functionally obsolete and the current average age of the nation's bridges has reached over 42 years. Therefore, the effects of corrosions on structural safety should be

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carefully investigated. In order to consider the effects of corrosions on metal materials, data collection on the rate of material loss of metal specimens in various environments started from the 1940s and possible type of bridge corrosions were studied (Albrecht and Naeemi, 1984). Later, a deterioration model was developed and major parameters for corrosion of structural members were identified, including the deterioration rate (annual loss) and pattern (roughening and pitting) (Czarnecki and Nowak, 2008; Nowak and Szerszen, 2001). In addition, fatigue strength reduction curves were also defined in the model. Since the long span bridges often serve as the backbones of the main transportation lines for hurricane evacuations and daily operations, the safety and reliability of these existing bridges, especially for those with serious corrosions, should be carefully assessed for high sustained winds from hurricanes or non-synoptic winds in a bridge's life-cycle. A full understanding of the bridge fatigue damage accumulation in their life cycles under the multiple dynamic loads including the extreme hurricane winds is needed to prevent potential catastrophic failures.

In the present study, the fatigue damages of existing bridges with corrosion under vehicle and non-stationary extreme hurricane wind loads are evaluated. This paper is organized as the following four main sections. In the first section, the vehicle-bridge-wind dynamic system is introduced, including the modeling of road surface profiles and the wind loads on the dynamic system. In the second section, the non-stationary wind time histories during hurricane events are simulated as a summation of time-varying mean and fluctuating non-stationary components. The third section introduces the corrosion model used in the present study. Numerical examples of an existing long span bridge with corrosions under simulated non-stationary hurricane winds and vehicles are presented in the fourth section to demonstrate the effects of the dynamic loads on the life estimations under extreme hurricane winds. Conclusions are drawn from the results of the case study at the end of the paper.

2. Vehicle–bridge–wind dynamic system

2.1. Equations of motion for vehicle–bridge–wind system

As discussed earlier, after including the wind loads on the vehicle–bridge dynamic system, Cai and Chen (2004) built a framework of vehicle–bridge–wind dynamic analysis. In the present study, the vehicle–bridge–wind dynamic system is built following the same procedure as in the previous study (Cai and Chen, 2004; Zhang et al., 2013). In the dynamic system, the tires and suspension systems are idealized as linear elastic spring elements and dashpots. The contact between the bridge deck and the vehicle is assumed as a point contact, and the forces between them are modeled as coupling forces between the tires and the randomly generated road surfaces. Wind loads on bridges, including static, buffeting, and self-excited components, and the quasi-static wind loads on the vehicles enter the coupled equations of motions for vehicle–bridge dynamic system by updating the stiffness matrices, damping matrices, or force vectors. Vehicles with various axle numbers are modeled in the dynamic system. Different types of elements could be used to model the bridge decks. The conventional finite element method is used to obtain the time independent terms in mass matrix and stiffness matrix of the bridge. Finally, the motions of the bridge and the vehicle can be expressed as the following equations:

$$[M_b]\{\ddot{d}_b\}+[C_b]\{\dot{d}_b\}+[K_b]\{d_b\}=\{F_b^c\}+\{F_b^w\} \tag{1}$$

$$[M_v]\{\ddot{d}_v\}+[C_v]\{\dot{d}_v\}+[K_v]\{d_v\}=\{F_v^G\}+\{F_v^c\}+\{F_v^w\} \tag{2}$$

where, $[\boldsymbol{M}]$ are the mass matrices, $[\boldsymbol{C}]$ are the damping matrices and $[\boldsymbol{K}]$ are the stiffness matrices with subscript b for the bridge and v for vehicles; $\{F_b^c\}$ is the vector of wheel–bridge contact forces on the bridge, $\{F_b^w\}$ is the vector of wind effects on the bridge, $\{F_v{}^G\}$ is the vector of the self-weight of vehicles, $\{F_v{}^c\}$ is the vector of wheel–road contact forces acting on vehicles, and $\{F_v{}^w\}$ is the vector of wind effects on vehicles. The two equations are coupled through the contact condition, i.e., the interaction forces $\{F_v^c\}$ and $\{F_b^c\}$, which are action and reaction forces existing at the contact points of the two systems and can be stated as a function of deformation of the vehicle's lower spring:

$$\{F_b^c\}=-\{F_v^c\}=[\boldsymbol{K_l}]\{Z_a-Z_b-r(x)\}+[\boldsymbol{C_l}]\{\dot{Z}_a-\dot{Z}_b-\dot{r}(x)\} \tag{3}$$

where $[\boldsymbol{K_l}]$ and $[\boldsymbol{C_l}]$ are the coefficients of the vehicle's lower spring and damper; Z_a is the vehicle-axle-suspension displacement; Z_b is the displacement of the bridge at wheel–road contact points; Δ_l is the deformation of the lower springs of the vehicle, $r(x)$ is the road surface profile, $\dot{r}(x)=(dr(x)/dx)(dx/dt)=(dr(x)/dx)V(t)$ and $V(t)$ is the vehicle velocity.

At each time step, the new position of each vehicle is identified and updated and the time dependent terms in the equations of motions will be updated. The equations of motions are solved in time domain using Rouge-Kutta method. At the each time step, when the contact force turns into tension, the vehicle tires are assumed to leave the riding surface and the force is set to be zero. The corresponding time dependent terms in the equations of motions are updated simultaneously, including the contact forces, wind induced changes in stiffness and damping matrices, and wind loads on vehicles. After obtaining the bridge dynamic response $\{\boldsymbol{d_b}\}$, the stress vector can be obtained by (Zienkiewicz et al., 2005)

$$[\boldsymbol{S}]=[\boldsymbol{E}][\boldsymbol{B}]\{\boldsymbol{d_b}\} \tag{4}$$

where $[\boldsymbol{E}]$ is the stress–strain relationship matrix and is assumed to be constant over the element, and $[\boldsymbol{B}]$ is the strain–displacement relationship matrix assembled with x, y and z derivatives of the element shape functions.

2.2. Generating random road surface profiles

Dynamic effects from moving vehicles are attributed to two sources in AASHTO LRFD specifications (AASHTO, 2010). One is the hammering effect due to the discontinuities of the vehicle riding surface, such as deck joints, cracks, potholes, and delaminations. The discontinuities can be modeled with a step up or down for the faulting between the approach slab and the pavement between the bridge deck and the approach slab (Green, 1990; Shi et al., 2008). The other is the long undulations in the roadway pavement. The long undulations in the roadway pavement could be modeled as a zero-mean stationary Gaussian random process based on the studies carried out by Dodds and Robson (1973) and Honda et al. (1982). The road surface profiles could be generated through an inverse Fourier transformation (Wang and Huang, 1992):

$$r(x)=\sum_{k=1}^{N}\sqrt{2\phi(n_k)\Delta n}\cos(2\pi n_k x+\theta_k) \tag{5}$$

where θ_k is the random phase angle uniformly distributed from 0 to 2π; $\phi(\,)$ is the power spectral density (PSD) function (m^3/cycle) for the road surface elevation; and n_k is the wave number (cycle/m). The PSD functions for road surface roughness were developed by Dodds and Robson (1973), and three groups of road classes were defined with the values of roughness exponents ranging from 1.36 to 2.28 for motorways, principal roads, and minor roads. In order to simplify the description of road surface roughness, both of the two roughness exponents were assumed to have a value of two and the PSD function was simplified by