DISSERTATION

DYNAMIC ASSESSMENT OF THE LONG-SPAN CABLE-STAYED BRIDGE AND TRAFFIC SYSTEM SUBJECTED TO MULTIPLE HAZARDS

Submitted by

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ABSTRACT

DYNAMIC ASSESSMENT OF THE LONG-SPAN CABLE-STAYED BRIDGE AND TRAFFIC SYSTEM SUBJECTED TO MULTIPLE HAZARDS

Critical infrastructure systems, such as long-span bridges, offer the underlying foundation for many aspects of modern society, such as national security, quality of life and economy. Although the total number of long-span bridges is relatively small compared to short-span and medium-span bridges, long-span bridges often serve as backbones for critical interstate transportation corridors and also evacuation routes. Any traffic disruption due to bridge damage, failure, retrofitting or even major traffic accidents following some hazards can become disastrous to local community and emergency response efforts, underscoring the importance of the continued integrity, functionality and resilience following hazardous conditions.

Wind and traffic are the major service loads for long-span bridges. The extreme loads may include those caused by various natural or man-made hazards, such as earthquake, hazardous winds (hurricane, tornado), fire, blast, vehicle and barge collision etc. Compared to other hazards, hazardous wind and earthquake are particularly critical for long-span bridges, primarily due to their significant threats to the global structure performance and challenges of appropriately modeling the dynamic coupling effects between the bridge, traffic and hazards. In addition, there is another disastrous event: cable loss, which is very unique and critical for cable-supported bridges and could be caused by various natural and manmade hazards. There exist major challenges in the current state of the art on rationally predicting the longspan bridge performance subjected to multiple service and extreme loads. These challenges include realistic load characterization, methodological limitations and considerations of uncertainties.

A suite of holistic analytical frameworks of long-span cable-stayed bridges subjected to various service and hazardous loads are developed, with which insightful numerical analyses of the bridge performance subjected to these loads are carried out in this dissertation. Firstly, two general dynamic assessment frameworks are developed based on the mode superposition and finite element methods respectively for a long-span cable-stayed bridge and traffic system subjected to multiple threats, such as stochastic traffic, wind and some hazardous loads. Although developed based on a long-span cable-stayed bridge, the frameworks can be readily applied to long-span suspension bridges as well as bridges with shorter spans. In both simulation platforms, the bridge model and all individual moving vehicles in the stochastic traffic flow are directly coupled under multiple excitations from bridge deck roughness and other external dynamic loads. Through the established simulation platforms, the global dynamic responses of the bridge and each individual vehicle subjected to various service and extreme loads can be rationally predicted in the time domain. Secondly, built on the proposed general simulation platforms, a novel dynamic safety assessment model and a vehicle ride comfort evaluation model for the bridge-traffic system are further developed. Thirdly, also extended from the proposed simulation platforms, both deterministic and reliability-based assessment frameworks for long-span cable-stayed bridges subjected to breakage of stay cables are established by considering more rational service load conditions as well as cable-breakage characterizations. Lastly, in addition to the in-house programs focusing on research purposes, a hybrid simulation strategy for the bridge under traffic and seismic excitations and a timeprogressive simulation methodology for cable breakage events are also developed by taking advantage of the strength offered by commercial finite element software, e.g., SAP2000. These SAP2000-based strategies are expected to facilitate design engineers to more easily understand and conduct the related analyses in future engineering practices.

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DEDICATION

To my family

for their unconditional love and endless support

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CHAPTER 1 Introduction

1.1 Motivation of the dissertation research

As vital components of transportation network, the functionality and sustainability of long-span bridges may significantly impact the social and economic development of the encompassing society. Critical infrastructure systems, such as long-span bridges, offer the underlying foundation for many aspects of modern society, such as national security, quality of life and economy. Their lifetime performance, under normal, extreme or even hazardous conditions, often goes beyond pure structural damage or loss with cascading consequences in social, economic and environmental domains (ASCE 2003).

Due to the high complexity of structure, loads and uncertainties, reliable lifetime performance of long-span bridges are far from being well assessed. On the one hand, the current AASHTO LRFD design specification (AASHTO 2012) was primarily developed from short- and medium-span bridges with a maximum span less than 500 feet. The existing design guidelines in terms of service and extreme loads cannot be applied to long-span bridges due to some unique characteristics such as more complex dynamic coupling and load characterizations as compared to bridges with shorter spans. As a result, most existing long-span bridges received little support from the design guidelines, particularly on hazard analysis. This is also evidenced in a special report about existing bridges by ASCE (2003) that "... most of these [long-span] bridges were not designed and constructed with in-depth evaluations of the performance under combination loadings, under fatigue and dynamic loadings and for the prediction of their response in extreme events such as wind and ice storms, floods, accidental collision or blasts and earthquakes". On the other hand, during the past decades, there is very limited research on the dynamic performance of long-span bridges under different hazards, which is underscored in the same special report that "Although some performance aspects of our long-span bridges have been already tested through long-term usage, extreme environmental conditions, extreme natural events, or by unusual man-made accidents or incidents

(collisions, fire, explosions, blasts etc.), these represent only a small percentage of the total credible events that must be investigated and assessed for the continued lifetime performance of these bridges".

The **overall goal** of this dissertation is to close the existing gap between long-span bridges and other bridges in terms of dynamic performance assessment under various extreme or hazardous events by focusing on the development of analytical methodology and simulation-based analyses on long-span bridges. It is known that there exist major challenges in the current state of the art on rationally predicting the long-span bridge performance subjected to multiple service and extreme loads. These challenges include realistic load characterization, methodological limitations and considerations of uncertainties. To overcome these challenges, a suite of holistic analytical frameworks of long-span cable-stayed bridges subjected to various service and hazardous loads are developed, with which insightful numerical analysis of the bridge performance subjected to these loads can be carried out. In the following sections, the hazard events critical for long-span bridges and existing studies on related topics will be discussed.

1.2 Scientific context

1.2.1 Long-span cable-stayed bridges (LSCSBs)

Cable-stayed bridges are composed of three main components: bridge girder, pylon and stay cables. Cable-stayed bridges can be categorized into different types according to different characteristics, such as arrangement of stay cables, number of pylons, length of the longest span, etc. In terms of cable arrangement, cable-stayed bridges can have three types including harp, fan and semi-fan arrangement, as shown in Fig. 1.1.



(a) Harp cable arrangement



(c) Semi-fan cable arrangement

Figure 1.1 Three main types of cable-stayed bridges in terms of arrangement of stay cables

In terms of pylon number, cable-stayed bridges can be categorized as single-pylon, double-pylon and three pylon cable-stayed bridges, among which the former two types are more common than the last one. Along with the rapid social and economic development, more and more LSCSBs are erected or under construction. The maximum span length of LSCSBs has increased from several hundred meters to over one thousand meters in the last decade. The ten longest LSCSBs in the world are given in Table 1.2 including the information on the name, length of bridge main span, country and completed year. All the ten bridges have the semi-fan arrangement of stay cables and two pylons.

Rank	Name	Country	Completed year	Main span length (m)
1	Russky Bridge	Russia	2012	1104
2	Sutong Bridge	China	2008	1088
3	Stonecutters Bridge	China	2009	1018
4	Edong Bridge	China	2010	926

Table 1.1 The world's top 10 longest long-span cable-stayed bridges

5	Tatara Bridge	Japan	1999	890
6	Normandie Bridge	France	1995	856
7	Jiujiang Fuyin Bridge	China	2013	818
8	Jingyue Bridge	China	2010	816
9	Incheon Bridge	Korea	2009	800
10	Xiamen Zhangzhou Bridge	China	2013	780

1.2.2 The long-span bridge and traffic system

Long-span bridges usually support a large amount of traffic every day to perform the major functionality of daily service. Although the total number of long-span bridges is relatively small compared to short-span and medium-span bridges, long-span bridges often serve as backbones for critical interstate transportation corridors and also evacuation routes during emergencies. Any traffic disruption due to bridge damage, failure, retrofitting or even major traffic accidents following some hazards can become disastrous to local community, economy and emergency response efforts, underscoring the importance of the continued integrity, functionality and resilience following hazards. Due to the critical roles of supporting local transportations, long-span bridges need to maintain not only structural soundness, but also the capability of safely and smoothly transporting vehicles, particularly under extreme and/or hazardous conditions.

Studies have shown that the presence of multiple vehicles may considerably influence the dynamic behavior of the bridge owing to the coupling effects between bridge and traveling vehicles (Xu and Guo 2003; Cai and Chen 2004). When extreme loads exist, the dynamic coupling phenomena become even more complex, causing some challenges on rationally predicting the bridge dynamic response and further, the safety and serviceability of these bridges. In the meantime, the dynamic performance of moving vehicles on the bridge can also be significantly affected by the dynamic coupling and extreme loads (e.g. winds or earthquake), which may further affect the driving safety of vehicles (Baker 1991; Chen and Cai 2004). Just taking crosswind as an example, long-span bridges are usually built across the strait or river

and therefore vehicles on bridges are usually more open to wind than on the roads with higher elevations and less blocking effects from surrounding environment. Besides, vehicles driven on long-span bridges may be temporarily shielded from the wind by the bridge tower or nearby vehicles, and therefore they may enter the sharp crosswind gust and are more likely to experience rollover accidents. Apparently, it becomes important to evaluate the vehicle driving safety and comfort issues on long-span bridges under multiple dynamic threats in order to provide guidance on emergency evacuation, post-hazard traffic management and accident prevention for the transportation system.

Therefore, under multiple service and extreme loads, excessive dynamic responses of the bridge not only may cause local member damage, serviceability issues, or even global failure of the bridge structure itself, but also may raise traffic safety concern on moving vehicles on the bridge. To rationally predict the performance of a long-span bridge and traffic system under realistic combinations of service and extreme loads becomes vital for appropriately assessing the associated safety and serviceability risks. To analyze the bridge and realistic traffic on the bridge as a whole system becomes necessary to provide a holistic view of the functionality and serviceability of this type of critical infrastructure under various hazards.

1.2.3 Hazardous events critical for long-span bridge and traffic system

For long-span bridges, wind and traffic are usually major service loads. Hazardous winds such as hurricane and tornado are considered as extreme loads on long-span bridges. Other extreme loads include those caused by various natural or man-made hazards, such as earthquake, fire, blast, vehicle and barge collision, etc. Compared to other hazards, hazardous winds and earthquake are particularly critical for long-span bridges, primarily due to their significant impacts on the global structure and possible failure and challenges on appropriately modeling the dynamic coupling effects between the bridge, traffic and hazards. In addition to these two hazards, for cable-supported bridges, there is one unique disastrous event: cable loss, which is very critical to the bridge-traffic system and could be caused by various natural and man-made hazards. A brief review of these critical hazardous events for long-span bridges is made in the following.

1.2.3.1 <u>Hazardous and service winds</u>

For many long-span bridges, slender and streamlined girders are very common, which make these bridges sensitive to wind excitations (Ge and Tanaka 2000). Since the infamous Tacoma Narrows Bridge failure in 1940, bridge aeroelastic and aerodynamic assessments often dominate the hazard designs of slender long-span bridges (Simiu and Scanlan 1996). With enhanced understanding of bridge aerodynamics, careful selection of bridge cross-sections, advanced numerical simulations and sophisticated wind tunnel experiments are commonly conducted to design most long-span bridges against extreme wind hazards. As a result, the risk of aeroelastic or aerodynamic failure (e.g. flutter) for modern long-span bridges becomes practically low. Besides, for the rare "extreme (hazardous) winds" (e.g. hurricane or tornado winds) with very high wind speeds, the chance of experiencing simultaneous occurrence of other hazards or the presence of considerable traffic on the bridge is very low. As a result, hazardous wind effect on long-span bridges can be essentially investigated as a type of traditional single-hazard event.

In addition to the rare hazardous winds with extremely high wind speeds, mild to strong winds exist almost all the time at the height of the bridge decks, which is typically more than 50 meters above the water/ground for long-span bridges. These winds are often called "service winds" due to their high frequency of occurrence during a bridge's service life when normal traffic is often also present (Chen and Wu 2010). In recent years, more attention has been shifted from "hazardous winds" to "service winds" and the compounded effects along with other loads on long-span bridges. It is mainly due to the fact that considerable service winds often exist on long-span bridges including the moments when some hazards, such as an earthquake, may occur at the same time.

1.2.3.2 Earthquake hazard

As compared to bridges with shorter spans, long-span bridges were often deemed safer under earthquakes as evidenced by much fewer past failure records. However, as pointed out by some researchers (e.g., Chen and Duan 2015), the low number of historical earthquake-induced failures in the United States does not mean long-span bridges are immune to earthquake, rather is largely because of the low number of long-span bridges and the fact that existing long-span bridges have not been often tested with "big" earthquakes (Chen and Duan 2015). Table 1.2 lists several recent long-span bridges experiencing significant damage caused by earthquakes despite the fact that these bridges all followed the modern earthquake design guidelines (Chen and Duan 2015). Furthermore, sometimes even being more critical than direct structural damage of long-span bridges, the consequence from the disruption of local traffic and economy following an earthquake can be disastrous due to the substantial roles of these critical infrastructures in the modern society. Two examples of extended retrofitting/rebuilding process are on the San Francisco–Oakland Bay Bridge following the Loma Prieta earthquake and the Higashi-Kobe Bridge due to Kobe earthquake. Both bridges were closed for about a month following earthquakes, which alert people about the excessive time on repairing and significant impact on local traffic as compared to what was originally expected.

Table 1.2 Several historical long-span bridge damages due to earthquake

Bridge Name	Bridge Type	Year	Damage/consequence
Arakawa Bridge	Suspension	1923	Tower cracks
Gosho Bridge	Suspension	1948	Girder buckle
Higashi-Kobe Bridge	Cable-stayed	1995	Comprehensive damage: girder connections; buckling of cross-breams; permanent deformation
Chi-Lu Bridge	Cable-stayed	1999	Plastic hinges in the pylon, box deck; cracking in the bent columns; anchorage failure of cable

1.2.3.3 Cable loss incident caused by various hazards

Cable structures have been used widely on cable-stayed bridges, suspension bridges, and some arch bridges. Stay cables for cable-stayed bridges and vertical suspender cables (hangers) for suspension bridges may experience damages due to various reasons, such as natural hazards (e.g., wind, lighting), man-made hazards (e.g., vehicle/barge collision, explosion, fire, cutting) or structural deterioration (e.g., fatigue cracks). Different from what many people feel that cable-stayed bridges have high intrinsic redundancy of stayed cables, studies on the behavior of the cables under terrorist attack and high velocity impact suggest that abrupt cable loss is "readily achievable", particularly under intentional attack scenarios (Zoli and Steinhouse 2007). For suspension bridges, Zoli and Steinhouse (2007) found that the potential for unzipping and progressive failure of the whole suspension bridge becomes a concern when several consecutive suspenders (hangers) are lost, after which the redistributed force may result in overload of the adjacent suspenders (hangers) that remain. Cable-loss events are actually more realistic than people think those would be. Without being exhaustive, five recent cable-loss incidents by various causes during the past ten years are summarized below in Table 1.3. One bridge was totally collapsed and others experienced mild to moderate damages.

Bridge Name	Bridge Type	Year	Reason for cable failure
Martin Olav Sabo Bridge	Cable-stayed Suspension	2012	Wind
Kutai Kartanegara Bridge	Suspension	2011	Cable replacement
Mezcala Bridge	Cable-stayed	2007	Truck fire
Rion Antirion Bridge	Cable-stayed	2005	Fire caused by lighting
Sgt. Aubrey Cosens VC Memorial Bridge	Suspension	2003	Fatigue cracks

Table 1.3 Recent cable-loss incidents

1.3 State of the art of the research on LSCSBs under dynamic loadings

1.3.1 Research on LSCSBs under single dynamic loading

1.3.1.1 <u>Bridge-vehicle interaction analysis</u>

The major serviceability of bridges is to support daily traffic, which serves as the main live load significantly influencing the bridge performances. The research on bridge-vehicle interactions dates back to the seventies of the 20th century (Blejwas et al. 1979; Olsson 1985). At that time, the inertia effects of the vehicle bodies are ignored and the impact of vehicles on a bridge was assumed to be moving loads. The vehicle model was improved later as a moving mass (Sadiku and Leipholz 1987), through which the inertia effects of vehicles on bridges can be considered. Not until the last decade, the vehicles were modeled as dynamic systems consisting of mass, spring and damping components that not only vibrate on

their own but also interact with the supporting bridges. Studies have shown that traffic loads can be significant to the dynamic performance of LSCSBs owing to the complex dynamic interactions with bridge structure and wind (Xu and Guo 2003; Cai and Chen 2004).

Due to the moving nature of the vehicle model, existing studies of bridge-vehicle dynamic interactions have been primarily carried out in the time domain. Existing bridge-vehicle interaction analyses to consider the contacting conditions can be attributed to two general approaches: iterative approach and non-iterative approach. The former approach treats the differential equations of motion for a bridge and vehicles separately and considers the contacting conditions through iterations at the contacting points (e.g., Guo and Xu 2006). The iteration will terminate until the displacements at all the contacting points meet the prescribed convergence criteria at the same time. The non-iterative approach assumes the tires of the vehicles and the bridge deck to be point-contact without separation and adopts direct integration method to obtain the dynamic response solution at each time step without any iteration (Guo and Xu 2001; Cai and Chen 2004). The coupled stiffness and damping matrices are time-dependent and need to be updated at each time step in order to consider the coupled interaction effects between the bridge and each vehicle. To incorporate the dynamic excitations from wind or earthquake, the iterative approach can only be used in the full-order finite element analysis (e.g., Guo and Xu 2006), while the non-iterative approach can be used in both the modal superposition method (e.g., Cai and Chen 2004) and the full-order finite element method (e.g., Guo and Xu 2001; Xu and Guo 2003). In most of the studies considering bridge-vehicle coupling effects, it usually considered a single vehicle or a series of vehicles at certain intervals driven through the bridge at constant speeds. Nevertheless, considering that traffic on the bridge is actually stochastic following certain rules, e.g., accelerating, decelerating and braking, the interactions between the bridge and vehicles may be largely different from those with a series of vehicles driven at equal intervals with constant speeds. Chen and Wu (2010) proposed a simulation approach to evaluate the bridge dynamic performance considering the combined effects of wind and stochastic traffic. The approach approximately replaced each individual vehicle dynamic model with the equivalent

dynamic wheel loading (EDWL) obtained from the bridge-wind-single-vehicle interaction analysis (Chen and Cai 2007).

1.3.1.2 Bridge aerodynamic analysis

The bridge aerodynamic analysis can be conducted either based on the structural vibration theory or using the method of computational fluid dynamics (CFD). Despite the advances of directly modeling the structure and wind field, there is still a long way for CFD method to be adopted for multi-hazard analysis of LSCSBs. Based on the structural vibration theory, the dynamic simulation methodologies of long-span cable-stayed or suspension bridges under only wind excitations have been well developed and improved gradually during the past several decades.

Bridge aerodynamic analyses can be typically conducted in frequency domain or time domain. For aerodynamic stability study, frequency-domain analyses were mostly carried out in either multi-mode formulation (e.g., Jain et al 1996; Ding et al. 2002) or full-order formulation (e.g., Ge and Tanaka 2000; Ding et al. 2002; Zhang et al. 2011) of three-dimensional bridge model. Aerodynamic analyses considering both self-excited forces and buffeting forces were found to be more popular to be carried out using the time-domain approach than frequency-domain approach during the past decade. Time-domain aerodynamic analysis of long-span bridges under turbulent wind can also be conducted using multi-mode formulation (e.g., Chen and Cai 2003) or the full-order formulation (e.g., Chen and Kareem 2001). In the classical aerodynamic theory, the self-excited forces are expressed by motion vectors and 18 experimentally determined flutter derivatives and the buffeting forces are expressed by static wind coefficients and aerodynamic admittance functions (Jain et al. 1996). The frequency-dependent flutter derivatives and aerodynamic admittance functions can be directly formulated as aerodynamic stiffness and damping matrices in the aerodynamic analysis via the multi-mode approach. The aerodynamic forces in the flutter and buffeting time-domain analyses using full-order approach are usually expressed as convolution integrals via a rational function technique (Chen et al. 2000) or indicial function technique (Scanlan 1984).

1.3.1.3 Bridge seismic analysis

Similar to the classification for bridge aerodynamic analysis, the methodologies for bridge seismic analysis can also be classified as frequency-domain approach and time-domain approach. For bridge seismic analysis, two general frequency-domain approaches can be classified as random vibration approach (e.g., Zerva 1990; Zhang et al. 2009) and response spectrum approach (e.g., Berrah and Kausel 1992; Der kiureghian and Neuenhofer 1992). Frequency-domain analyses are usually carried out because of its computational convenience without losing significant accuracy especially when no strong material or geometric nonlinearity is involved. Since frequency-domain analyses usually depend on the concept of modal superposition, it can only be limited to linear analysis. The time-domain seismic analysis of bridge can be carried out through the multi-mode approach or full-order approach. The multi-mode approach based on mode superposition principle can be carried out in the time domain for multi-point bridge seismic analysis through relative motion method (RMM) (e.g., Léger 1990), large mass method (LMM) (e.g., Léger 1990) and large spring method (LSM) (Edward 2002). The multi-mode approach in the time domain is also limited to linear dynamic analysis due to the inability to take into account various nonlinearities. When considering severe earthquake loading scenarios, LSCSBs are expected to experience inelastic response, which cannot be fulfilled using a frequency-domain approach or multimode approach in the time domain. The nonlinear time history analysis (NTHA) of long-span bridges subjected to spatially varying earthquake excitations can be adopted to consider the nonlinear behavior, as found in some existing studies (e.g., Karmakar et al. 2012).

1.3.1.4 Simulation of cable breakage events

As important load-bearing members of cable-stayed bridges, stay cables may also experience corrosion, fatigue, accidental or intentional actions, which may cause a reduction of sectional resistance capacity and even lead to possible breakage failure. As the primary guideline for the design of bridge cables in the US, the Post Tensioning Institute (PTI 2007) issued the recommendations for stay cable design, testing and installation. Designing a cable-stayed bridge against the breakage of a single stay cable is required in the PTI design recommendations (PTI 2007). The recommendations by PTI (PTI 2007) provide two load

application methods in order to quantify the dynamic effects due to the loss of cable: one is the pseudodynamic method, in which the equivalent static analysis is performed with a pair of impact pseudodynamic forces, resulting from 2.0 times the static forces, applied at the top and the bottom anchorage locations of the ruptured cable; the other one is the nonlinear dynamic analysis, in which the dynamic cable forces due to the cable breakage are applied. As compared to the pseudo-dynamic analysis focusing on the maximum responses during the whole process of the abrupt loss of a cable, nonlinear dynamic analysis can provide more detailed and accurate information throughout the cable breakage process. Different from bridge aerodynamic and seismic analyses, the research on the breakage of stay cables on cable-stayed bridges has just emerged several years ago. As a relatively new area, there have been a couple of studies on nonlinear dynamic analysis of cable breakage in recent years (Mozos and Aparicio 2010a and b; Ruiz-Teran and Aparicio 2009; Cai et al. 2012). In the aforementioned studies, a cable breakage event is typically simulated by changing the cable forces at the two anchorage nodes of the breaking cable. This approach, however, has some limitations, which will be discussed in the later sections.

1.3.2 Research on LSCSBs under multiple dynamic loadings

Most of the existing studies dealt with a single type of dynamic load for LSCSBs, such as bridge dynamic analysis with moving traffic, buffeting analysis under turbulent wind loads, time-history seismic analysis or cable breakage simulation. During the past decade, there have been several studies that conducted dynamic analyses of LSCSBs under a combination of two dynamic loadings, such as vehicle-wind, vehicle-earthquake. Few, if any, study on LSCSBs under other combinations of the concerning dynamic loads has been reported in the literature so far. A review of some relevant existing studies is made in the following.

1.3.2.1 <u>Bridge-vehicle-wind interaction analysis</u>

The study of coupling interaction effects among bridge, wind and vehicles started a decade ago using the time-domain analysis of the bridge-vehicle system (Xu and Guo 2003; Cai and Chen 2004). In those

studies, only a single vehicle or a series of vehicles distributed at equal distances and moving at constant driving speeds are involved in the bridge-traffic system. The aerodynamic forces acting on the bridge girder are composed of the self-excited forces and buffeting forces. The aerodynamic wind forces acting on the vehicles are determined by means of a quasi-static approach. Both the aerodynamic forces and the bridge-vehicle interaction forces are applied on the bridge and vehicles to obtain the dynamic responses under wind excitations. It has been found that the lateral responses of the bridge and vehicles largely depend on wind excitations. The bridge-vehicle-wind interaction analysis plays an important role in the serviceability and safety performance of the bridge and vehicle on LSCSBs.

In order to simulate the realistic traffic flow moving on a long-span bridge, advanced traffic flow simulation in a microscopic scale was conducted on a long-span bridge and connecting roadways (Chen and Wu 2011). A new simulation approach was further proposed to consider the bridge performance under the combined effect of wind and stochastic traffic flow at the same time (Chen and Wu 2010). Such an approach, for the first time, can provide reasonable estimation of bridge response considering the combined effects of wind and stochastic traffic.

1.3.2.2 Seismic analysis considering the presence of moving vehicles

In recent years, some efforts have been put forth on studying more realistic load combination scenarios for long-span bridge seismic analyses. Liu et al. (2011) studied the vibration behavior of a suspension bridge due to moving vehicle loads considering vertical support motions caused by earthquake. It was found in the study that strong coupling effects existed between moving vehicles and vertical seismic excitations, which greatly amplified the response of the bridge near the end supports. In the study, the vehicles were not treated as dynamic systems rather a row of equidistant moving forces. Therefore, the dynamic interactions between the bridge and vehicle systems were not considered in the analysis. Several studies (Xia and Han 2006; Yau 2009) have conducted the seismic analysis of railway bridge and train system with the consideration of bridge-train dynamic interactions. It was concluded from those studies that the presence of vertical ground motions significantly affect the stability and safety of the

moving train. There are currently very limited studies that focus on the dynamic interaction simulation of bridge-vehicle system subjected to earthquake ground motions.

1.3.3 Review of the reliability methods which are potentially applicable to long-span bridge performance study under hazards

Most existing long-span bridges were designed primarily through deterministic methods (Catbas et al. 2008). However, it is known that the structural properties, such as material and sectional properties, and the applied loadings, such as service loads from moving traffic, are innately random owing to different sources of uncertainties (Torres and Ruiz 2007). Accordingly, the reliability method is usually adopted to evaluate the risks of structural failure corresponding to different limit states considering those uncertainties. As a relatively new research area, the existing reliability study of LSCSBs subjected to breakage of stay cables is very rare. No publication has been found in which the structural reliability of LSCSBs is assessed corresponding to different limit states of cable loss. The reliability methods that can be adopted in various structural reliability assessments are reviewed and compared in the following.

In reliability analysis, the performance of the structure is governed by a limit state function (performance function) of a vector of random variables, X_1 , X_2 , ..., X_n . The probability of failure is calculated by integrating the joint probability density function $f_X(x_1,x_2, ..., x_n)$ over the random variable domain. For most engineering problems, the joint probability density function is not explicitly available. The common reliability methods can be attributed to three types: simulation-based reliability method originated from Monte Carlo simulation (MCS), approximate reliability method such as FORM or SORM, and the combination of simulation-based and approximate methods.

For the simulation cases with small probability of failure or the cases that is time-consuming for a single case, e.g., nonlinear dynamic analysis, simple MCS requires long simulation time and limits its usage. To reduce the computational cost and the statistical error inherent in MCS, different variance reduction techniques have been developed and improved during the past several decades, such as simple

importance sampling (Engelund and Rackwitz 1993), adaptive importance sampling (Bucher 1988), Latin hypercube sampling (Florian 1992), or the combination of two sampling techniques (Olsson et al. 2003).

In the cases with explicit limit state functions, the first order reliability method (FORM) and the second order reliability method (SORM) are found to be more efficient than Monte Carlo simulation (MCS) with sufficient accuracy. However for structural reliability analysis, especially for the cases involved with complex vibration analysis or finite element analysis, the limit state functions are inherently implicit rather than explicit, and therefore FORM or SORM are usually not directly applicable. Several recourses have been explored to tackle the difficulties owing to implicit limit state functions.

The first one is to establish an explicit limit state function by introducing random factors with certain distributions and calculate the failure probability directly from the limit state function, such as that adopted in the flutter reliability analysis by Pourzeynali and Datta (2002). Secondly, the response surface method (RSM) can be adopted to approximate the actual response surfaces and actual limit state functions. In RSM, the original limit state function is approximated by the response surface function (RSF) in the form of polynomials of random variables. Thirdly, the combination of stochastic finite element method (SFEM) and FORM (SORM) can be adopted on a basis of sensitivity-based approach. In this method, the search for the design point or checking point can be carried out by evaluating the value and gradient of the performance function at each iteration. The value and gradient of the performance function are obtained from deterministic structural analyses and sensitivity analyses. Lastly, Latin hypercube sampling (LHS) can be adopted to determine the sets of samples for random variables and reliability analysis can be further conducted (Olsson et al. 2003). In LHS, the possible value of each random variable is partitioned into several strata, from each of which a value is randomly selected as a representative value. It is a stratified sampling scheme and may reduce the number of complex finite element analysis significantly with reasonable accuracy.

1.3.4 Research on driving safety of moving vehicles on LSCSBs

In automobile engineering, significant efforts have been made regarding the simulation of vehicle dynamics and accidents. Most existing studies focus on the vehicle safety analysis owing to excessive driving speeds and adverse topographic conditions on rigid road. The studies that evaluate the vehicle driving safety considering the interactions with the supporting structure and wind excitations are still very limited. Baker (1986, 1987, 1991, 1994) have conducted explorative studies on the safety of single road vehicle and investigated the accident wind speeds for different types of accidents under sharp crosswind gust. However, the excitations from road surface roughness and the vibration from the supporting structure were not considered in these studies. Guo and Xu (2006) conducted the vehicle safety analysis considering the bridge-vehicle interaction effects in high cross-winds. In the studies (Chen and Cai 2004; Chen et al. 2009), the authors developed a local vehicle accident assessment model and combined it with the global bridge-vehicle interaction model to consider the vibrating effects from the bridge structure in the windy environment. Chen and Chen (2010) further improved the local vehicle accident model with new transient dynamic equations and accident criteria to consider more realistic weather environment, road surface and topographical conditions. In these studies, the dynamic bridge-vehicle interaction analysis was firstly conducted to obtain the acceleration of the vehicle wheels. The accelerations on vehicle wheels were then applied to the vehicle accident assessment model as external base excitations to continue the vehicle accident analysis.

1.3.5 Research on driving comfort of moving vehicles on LSCSBs

Existing vehicle ride comfort issues are usually evaluated by considering normal driving conditions, such as on roadways under normal weather. For adverse driving conditions, for example on long-span bridges under moderate to strong winds, related studies on ride comfort are still very few. Most existing studies on vehicle ride comfort evaluation focused on the vehicles that are driven on rigid roads without considering the interaction with supporting structures as well as wind excitations (e.g., Navhi et al. 2009). Xu and Guo (2004) evaluated the ride comfort of a group of heavy vehicles on a long-span cable-stayed

bridge under crosswind. Yin et al. (2011) evaluated the ride comfort of a single truck when it moves through a multi-span continuous bridge at constant driving speeds. No other related studies are found for the vehicle ride comfort analysis in which the coupling effects from the supporting structure, moving multiple vehicles and crosswind excitations are taken into account.

1.4 Limitations on the dynamic analysis of LSCSBs subjected to multiple dynamic loadings

It is found from the previous discussions that the time-domain analysis is the dominant way that can be utilized to consider aerodynamic forces and earthquake excitations as well as other loading conditions from stochastic traffic and cable breakage event. Due to the lack of a general simulation platform, longspan bridges were traditionally analyzed under a single type of dynamic loads at a time without considering any other ones, such as flutter and buffeting analysis under turbulent wind loads, time-history seismic analysis, or bridge dynamic analysis with moving vehicle(s). Combined load scenarios were usually considered through superposition of the results of the bridge under individual loads. Despite the convenience of the superposition method, single-load-based approaches have some limitations, such as inability to consider the coupling effects of multiple loads and possibility of overestimating or underestimating the performance of different members (e.g. Chen and Cai 2007). Some challenges related to the state-of-the-art studies on the dynamic analysis of LSCSBs subjected to several combined dynamic loadings are described in the following sections.

1.4.1 Limitations on bridge-wind-traffic interaction studies

As discussed earlier, a new simulation approach of the bridge-traffic-wind system was proposed to consider the bridge performance under the combined effect of wind and stochastic traffic flow at the same time (Chen and Wu 2010, 2011). This approach can provide reasonable estimation of bridge response considering the combined effects of wind and stochastic traffic for the first time. This was achieved by approximately replacing each individual vehicle dynamic model on the bridge with the corresponding time-history excitations named "equivalent dynamic wheel loading" (EDWL) obtained from the bridge/wind/single-vehicle interaction analysis (Chen and Cai 2007). Such an approach (Chen and Wu

2010), however, did not couple all the vehicles and the bridge simultaneously. As a result, the dynamic performance of each individual vehicle in the stochastic traffic flow cannot be rationally obtained.

1.4.2 Limitations on the dynamic analysis of LSCSBs with multiple service and extreme loads from stochastic traffic, wind and earthquake excitations

Long-span bridges usually support a considerable amount of traveling vehicles and experience wind loads on the bridge decks nearly every day. In addition to the vibrating effects of the bridge due to the interactions from vehicles and wind, existing studies have indicated that crosswind may induce some single-vehicle accidents of vehicles traveling at the speeds within a certain range (Chen et al. 2010; Guo and Xu 2006). Along with wind loads, some extreme loads such as earthquake may also occur on the bridges, which may pose some safety risks to the bridge as well as travelling vehicles. In the existing studies of bridge seismic analysis considering bridge-vehicle interactions (Liu et al. 2011; Li et al. 2012), the traffic flow is usually deterministic with a series of vehicles at equal spaces travelling on the bridge with a constant speed. It is known that the movements of vehicles in the traffic flow are actually stochastic and may experience acceleration, deceleration and braking, which comply with certain traffic rules. Considering also the highly nonstationary properties of earthquake excitations, the dynamic coupling effects between the bridge and a deterministic series of vehicles at a constant travelling speed may differ largely from those between the bridge and stochastic traffic flow. The dynamic assessment of the bridge and travelling vehicles with the presence of wind and earthquake ground motions requires the fully-coupled dynamic analysis of the bridge-traffic system which incorporates properly the wind and earthquake excitations. No previous study was found to conduct the detailed dynamic assessment of the bridge and vehicles subjected to wind and earthquake excitations, for which the lack of a general simulation platform is the primary reason.

1.4.3 Limitations on the simulation methodologies for cable loss scenarios considering service loads

Studies on the dynamic investigation of cable loss events for cable-stayed bridges have gained some progress during the past several years and can be reflected in some publications (e.g., Ruiz-Teran and

Aparicio 2009; Mozos and Aparicio, 2010 a & b). In all these publications, the simulation of cable breakage scenarios is realized by changing the forces at the anchorage nodes of the breaking cable through commercial finite element software. By utilizing the force changing method, the dynamic analysis of cable breakage event can be conducted either on the intact bridge structure or the modified bridge structure with the breaking cable being removed (Zhou and Chen 2014a). Nevertheless, the two procedures can only represent the exact bridge situation either before or after cable breakage occurs, but not both. This is mainly due to the lack of flexibility of commercial FE software, in which the breaking cable cannot be removed during the process of a time-history dynamic simulation.

In almost all the existing studies, the traffic loads were either not considered or simply adopted as nominal design live loads, in the form a uniformly distributed load plus axle loads (e.g. HL-93), as defined in the design codes (e.g. AASHTO 2012). It is well known that the design live loads as defined in AASHTO specifications were calibrated only for short-span and conventional bridges under strength limit state in an "equivalent" sense. Primarily from the strength design perspective, the design traffic loads do not actually represent the realistic traffic on long-span bridges. It is considered that cable-breakage events on cable-stayed bridges, either accidental or intentional, can happen at any time when considerable traffic and wind may still remain on the bridge. The dynamic effects due to bridge-traffic interactions, bridge-wind interactions and buffeting forces may have significant impact on the bridge extreme response subjected to cable loss. Without considering realistic service loads from stochastic traffic and wind, the breakage of cables was assumed to start at a static initial state in most of the existing studies, through which the bridge dynamic response may be underestimated, as indicated in the study by the author (Zhou and Chen 2014a).

1.4.4 Limitations on the driving safety analysis of vehicles on LSCSBs

Existing studies have made some improvements on evaluating the driving safety of vehicles on LSCSBs in a windy environment by considering dynamic interactions between the vehicle and bridge (e.g., Chen and Chen 2010). However in most existing studies considering the interaction from bridges,

the vehicle safety is assessed when only one vehicle is traveling on the bridge at a constant speed and interaction effects from multiple vehicles in the stochastic traffic flow were not included due to the limitation of the simulation model. However in reality, the vehicles driven on the bridge are stochastic and vehicles may need to frequently speed up and down obeying certain traffic rules. To assume only one vehicle driven on the bridge at a constant speed cannot reflect the realistic situation and may also underestimate the accident risk the vehicle may experience in hazardous environment. In addition, there is nearly no study that evaluates the vehicle driving safety on LSCSBs considering simultaneous presence of multiple dynamic loadings, such as wind along with earthquake or possible structural member failure.

1.4.5 Limitations on the driving comfort analysis of vehicles on LSCSBs

Although vehicle ride comfort issues are usually evaluated before any new vehicle is approved to be on the roads, these studies are typically conducted considering normal driving conditions, such as on roadways under normal weather. For adverse driving conditions, for example on long-span bridges and/or with adverse weather, related studies on ride comfort are still rare. Most existing studies on vehicle ride comfort evaluation focused on the vehicles that are driven on rigid roads without considering the interaction with supporting structures as well as wind excitations. Besides, the traffic considered in the existing ride comfort studies was usually very approximate. For example, in the study by Xu and Guo (2004), a group of vehicles were assumed to be equally distributed and move along the bridge at a constant speed, which are apparently different from the realistic traffic on the roadway or bridges. In the study by Yin et al. (2011), only one vehicle was present on the bridge when vehicle comfort analysis was conducted. In addition to the oversimplified traffic simulation, the ride comfort analysis in the study by Xu and Guo (2004) was based on the single-axis root-mean-square (RMS) value with respect to one-third octave-band frequency in ISO 2631/1: 1985, which has been replaced with a new method in a later version (ISO 2631-1: 1997). In the new method (ISO 2631-1: 1997), the frequency-weighted RMS values are evaluated for ride comfort criterion based on multi-axis whole-body vibrations. Yin et al. (2011) evaluated the vehicle ride comfort performance based on the criteria in the latest version of ISO 2631-1
(1997). However, only the vehicle responses at the translational axes were involved to obtain the final RMS value and the participation of the response at other rotational axes are ignored.

1.5 Significance and contributions of the dissertation research

LSCSBs may be exposed to different combinations of dynamic loadings from turbulent wind, stochastic traffic, earthquake ground motions and single-/multiple-cable failure, whereas most studies only dealt with the dynamic simulation of some simple two-load scenarios: turbulent wind and stochastic traffic; deterministic traffic and single-cable failure, etc. The load scenarios on LSCSBs can often be more complex than simple two-load scenarios as introduced above. There is a need to develop general analytical models to assess the dynamic behavior of the LSCSBs under more realistic load combination scenarios. This dissertation attempts to establish a suite of simulation frameworks in order to investigate the dynamic performance of the bridge-traffic system under multiple dynamic loads. The specific scientific contributions of the dissertation are summarized in the following sections.

1.5.1 Contribution to fully-coupled bridge-traffic-wind interaction analysis

Studies have shown that vehicle loads can be significant to the dynamic performance of LSCSBs owing to complex dynamic interactions with bridge structure and wind (Xu and Guo 2003; Cai and Chen 2004). Currently there are very few studies investigating LSCSBs under the combined service and extreme loads effects from stochastic traffic and wind. The state-of-the-art bridge-traffic-wind interaction model is based on the equivalent dynamic wheel load approach to consider the combined effects of bridge from stochastic traffic and Wu 2010). Despite the improvements in the development of the bridge-traffic-wind interaction model, these studies also have several limitations as previously stated.

To overcome the limitations, **two** fully-coupled bridge-traffic-wind interaction models are established in this dissertation research to simulate the dynamic response in the bridge-traffic system more realistically. These models can be directly used as tools to study long-span bridges under more realistic service load conditions. More importantly, these models will serve as the basis for studies to consider various combinations of extreme and service loads in this dissertation. Different from existing studies, both of the fully-coupled bridge-traffic interaction models are developed by coupling the bridge model, all individual moving vehicles of the stochastic traffic flow, road roughness excitation, wind excitations and other hazardous loads simultaneously. In this way, the dynamic response of the bridge and the vehicles will be more accurately predicted. **The first model** is on the basis of linear vibration theory, in which the bridge structure as well as the dynamic loadings is modeled on modal coordinates and the structural responses are obtained by means of the mode superposition approach from participating modes. The coupling matrices are formed to deal with the motion-dependent forces so that the equations of motion can be solved without iteration. **In the second model**, the bridge structure and the dynamic loading are modeled on physical coordinates using the finite element approach. The motion-dependent forces, such as bridge-traffic interaction forces and self-excited forces from wind, are dealt with through an iterative approach. Unlike the first model based on linear vibration theory, the coupled model using finite element formulation is able to consider various sources of geometric and material nonlinearities associated with the structure and loads.

1.5.2 Contribution to bridge multi-hazard analysis considering excitations from traffic, wind and earthquake ground motions

LSCSBs exhibit unique features that are different from short and medium-span bridges such as higher traffic volume, simultaneous presence of multiple vehicles, and sensitivity to wind loads. The dynamic performances of bridges are seldom investigated considering the concurrent excitations from moving vehicles, wind and earthquake ground motions. To realistically investigate the dynamic performances of LSCSBs under multiple dynamic excitations, this dissertation firstly establishes the linear dynamic simulation framework on LSCSBs, in which the coupling effects between the bridge and vehicles under multiple excitations are well incorporated in the analysis.

On top of the fully-coupled simulation framework, a hybrid time-domain simulation methodology of the bridge and vehicles subjected to seismic excitations is further established by taking advantage of the strength from both the mode-based bridge-traffic interaction model and commercial FE software.

1.5.3 Contribution to cable breakage simulation considering interactions from traffic and wind

Stay cables are important load bearing members for LSCSBs, and therefore the realistic dynamic evaluation of the bridge subject to cable loss becomes essential in order to mitigate the unfavorable consequences. A general simulation methodology incorporating traffic and wind loads is necessary for rationally evaluating the dynamic performance of long-span cable-stayed bridges in a cable breakage event. Two simulation methodologies for cable breakage simulations considering concurrent dynamic excitations from moving traffic and/or wind are developed. As the first approach, a time-progressive nonlinear dynamic analysis methodology is developed through the commercial finite element program SAP2000. This methodology simulates the cable breakage event by changing the tension forces at the two anchorage ends of the breaking cable. The bridge-traffic interaction effects are considered in the simulation process of cable breakage by means of the equivalent wheel load approach. Developed on the basis of commercial finite element program, this methodology provides an efficient way to realistically simulate cable breakage events on cable-stayed bridges and can be easily followed by bridge engineers who are not familiar with advanced structural dynamics.

In addition to the approach based on commercial software, as the second approach, a new nonlinear dynamic analysis framework is established to perform the cable breakage simulation based on finiteelement (FE) formulation, in which the coupling interaction effects due to moving traffic and wind excitations are incorporated. It directly simulates cable-loss incidents through nonlinear iteration instead of the traditional force changing method. This methodology is able to model the exact structural formulation and force conditions before and after cable breakage occurs and therefore can simulate the cable breakage events with more accuracy.

1.5.4 Contribution to structural reliability analysis due to breakage of stay cables

The studies on LSCSBs subjected to breakage of stay cables are limited and there is no publication in which the structural reliability analysis is conducted on LSCSBs due to breakage of stay cables. In the dynamic analysis of LSCSBs subjected to breakage of stay cables, the dynamic responses of bridge are

obtained through nonlinear dynamic analysis and thus the limit state function cannot be explicitly expressed in terms of random variables. Although MCS can be utilized for cases with implicit limit state functions, it cannot be adopted in the current study due to its formidable computational cost. The RSM and sensitivity-based approach combining SFEM and FORM will also be computationally prohibitive considering uncertainties of many variables from the structure, service loads and extreme loads. In current study, Latin hypercube sampling (LHS) is adopted to evaluate the reliability of LSCSBs subjected to cable loss with the presence of multiple service loads.

By means of the developed FE simulation platform, sensitivity-based stochastic nonlinear dynamic analysis will be conducted with several random variables that are found to be susceptible to the dynamic behavior of a prototype LSCSB. A restrained sampling technique is applied to obtain a series of structural models with limited computational efforts for nonlinear time history analysis. Both bridge ultimate and serviceability limit states are defined based on the bridge and vehicle behavior, respectively. Different from those hazards with straightforward hazard intensity functions, the fragility of the bridge subjected to cable breakage incidents are assessed against some key parameters without specifying the particular hazard event leading to the cable breakage. The structural failure probability corresponding to bridge ultimate limit state is evaluated in the cable breakage scenarios with different cable breakage parameters. The bridge fragility with respect to dynamic loads from stochastic traffic and wind are also discussed in terms of bridge ultimate and serviceability limit states.

1.5.5 Contribution to vehicle safety analysis in windy environment

Aiming at directly considering the bridge-traffic interaction effects in the vehicle safety analysis, an integrated dynamic interaction and traffic safety assessment model is developed in which the fully-coupled bridge-traffic interaction analysis and the vehicle safety analysis are conducted within the same simulation framework considering multiple dynamic excitations. In this dissertation, vehicle safety analysis is carried out in the system of LSCSB and vehicles subjected to wind excitations. A total force approach is proposed to evaluate the vehicle safety and accident type of the vehicles driven on the bridge

in the windy environment. Based on the total force acting on the vehicle wheel, criteria for vehicle accidents, including lift-up, side-slip and yawing accidents are identified.

1.5.6 Contribution to vehicle comfort analysis in wind environment

Vehicle ride comfort issues for the drivers are related to not only individual satisfaction of driving experience, but also driving safety and long-term health of the drivers. A new methodology of ride comfort analysis is developed for typical vehicles driven on long-span bridges considering realistic traffic and environmental loads such as wind excitations. Built on the simulation framework developed previously by the writers, complex interactions among the long-span bridge, all the vehicles in the traffic flow and wind excitations are appropriately modeled with more accurate vehicle responses as predicted. The guidelines that are recommended in ISO 2631-1 (1997) for vehicle ride comfort evaluation are interpreted in the context of stochastic traffic flow with multiple vehicles. The vehicle ride comfort is evaluated by adopting the advanced procedures as currently recommended in the ISO standard, including obtaining the whole-body vibration response, frequency weighting the original response and determining the Overall Vibration Total Value (OVTV).

1.6 Organization of the dissertation

The objective of the dissertation is to establish the simulation frameworks and investigate the dynamic performance of LSCSBs under multiple dynamic service and extreme loads. In particular, the dynamic excitations from moving traffic, wind, earthquake and breakage of key structural members have been taken into account. An important feature of the developed frameworks is that they are able to incorporate the coupling effects among the bridge structure, each individual moving vehicle of the traffic flow and multiple threats. Chapters 2-9 are based on 5 published/accepted, 1 under review and 2 to-be-submitted journal papers. The chapters of the dissertation are briefly introduced as follows:

Chapter 1 – Introduction: A brief overview of the research background is presented, along with a literature review of the key components of the dissertation, including bridge-vehicle interaction analysis, aerodynamic analysis, seismic analysis, breakage of stay cables as well as multi-hazard analysis and

structural reliability analysis. The limitations of the existing research and the scientific contributions of the dissertation are discussed.

Chapter 2 – Dynamic simulation of long-span bridge and traffic system subjected to combined service and extreme loads (Zhou and Chen 2015a): This chapter will introduce a general simulation platform to investigate the dynamic performance of the bridge-traffic system under multiple service and extreme loads. The analytical platform directly couples the mode-based bridge model, all individual moving vehicles of the simulated stochastic traffic flow as well as excitations from road surface roughness, wind and earthquake. The proposed strategy is applied to a bridge-traffic system subjected to the excitations of road surface roughness, turbulent wind and seismic ground motions for demonstration.

Chapter 3 – A hybrid simulation strategy for the dynamic assessment of long-span bridges and moving traffic subjected to seismic excitations (Zhou and Chen 2014b): In this chapter, a new time-domain simulation methodology is developed for long-span bridges as well as moving vehicles subjected to seismic excitations by taking advantage of the strength from both the modal-based bridge-traffic interaction model and commercial FE software. Numerical investigation is carried out on the prototype long-span cable-stayed bridge and the moving vehicles subjected to earthquake ground motions.

Chapter 4 – Time-progressive dynamic assessment of abrupt cable breakage events on cable-stayed bridges (Zhou and Chen 2014a): This chapter proposes the time-progressive nonlinear dynamic analysis methodology based on SAP2000 to investigate the performance of a cable-stayed bridge subjected to the abrupt cable breakage in the time domain. Demonstrated through a prototype long-span cable-stayed bridge, the cable loss scenarios are simulated in a more realistic manner through incorporating stochastic moving traffic loads, dynamic bridge-vehicles interactions and associated dynamic initial states of the abrupt cable breakage event.

Chapter 5 – Framework of fully-coupled nonlinear dynamic simulation of long-span cable-stayed bridges subjected to cable-loss incidents (Zhou and Chen 2015c): The fully coupled bridge-traffic-wind interaction model is developed on the finite element basis by directly coupling the bridge structure, multiple vehicle dynamic models, and wind excitation. A nonlinear dynamic simulation framework

incorporating various sources of geometric and material nonlinearities is further developed to simulate the cable-loss incidents of the coupled bridge-traffic-wind system through direct nonlinear iteration.

Chapter 6 – Investigation of cable breakage events on long-span cable-stayed bridges under stochastic traffic and wind (Zhou and Chen 2015d): Through the advanced nonlinear dynamic simulation platform based on the finite-element formulation of the bridge, a comprehensive investigation of cable-loss incidents is conducted on a prototype long-span bridge. Comprehensive parametric studies are conducted to evaluate the impact from various important parameters related to cable-breakage process and service loads from stochastic traffic and wind.

Chapter 7 – Fully coupled driving safety analysis of moving traffic on long-span bridges subjected to crosswind (Zhou and Chen 2015b): An integrated dynamic interaction and safety assessment model of the fully-coupled bridge-traffic system is further developed based on the finite-element based bridge-traffic-wind interaction model. Through a total force approach, the vehicle lift-up, side-slip and yawing accidents can be evaluated within the same integrated simulation framework.

Chapter 8 – Vehicle ride comfort analysis with whole-body vibration on long-span bridges subjected to crosswind (Zhou and Chen 2015e): The vehicle ride comfort is evaluated by adopting the advanced procedures as currently recommended in the ISO standard, including obtaining the whole-body vibration response, frequency weighting the original response and determining the Overall Vibration Total Value (OVTV). Based on the finite-element based bridge-traffic-wind interaction model, ride comfort of the vehicles is evaluated by incorporating the dynamic excitations from supporting bridge structure and wind.

Chapter 9 – Reliability assessment framework of long-span cable-stayed bridge and traffic system subjected to cable breakage events: This chapter presents a structural reliability assessment framework for long-span cable-stayed bridges subjected to breakage of stay cables considering service load conditions from both traffic and wind. The bridge ultimate and serviceability limit states are defined from the bridge and vehicle responses subjected to breakage of stay cables, respectively. The fragility corresponding to bridge ultimate and serviceability limit states is evaluated in the cable breakage scenarios conditioned on cable breakage parameters, traffic density and steady-state wind speed.

Chapter 10 – Summary of the dissertation and future studies: This chapter firstly summarizes the achievements in the dissertations, including the proposed simulation frameworks and numerical studies. Then, the possible improvements of the dissertation and future studies are discussed, such as the detailed bridge/vehicle modeling and other advanced simulation models.

CHAPTER 2 Dynamic simulation of long-span bridge-traffic system subjected to combined service and extreme loads^{*}

2.1 Introduction

This chapter aims at investigating the performances of slender long-span bridges as well as moving vehicles of realistic traffic flow subjected to major extreme and service loads, such as road roughness, wind and earthquake loads. An improved fully-coupled bridge-traffic interaction model is proposed in order to perform the dynamic analysis of the bridge-traffic system. Different from most existing studies, the "fully-coupled" bridge-traffic interaction model is developed by coupling the mode-based bridge model, all individual moving vehicles of the stochastic traffic flow, road roughness excitation and external dynamic excitations simultaneously. Therefore, the dynamic responses of the bridge and each individual vehicle subjected to various service and extreme loads can be predicted in the time domain. By means of the proposed simulation platform, numerical investigation of a prototype long-span cable-stayed bridge subjected to different combinations of service and extreme loads is carried out in this study. Conclusions on the dynamic responses of the bridge and vehicles are summarized based on the analysis results in the cases with different loading conditions.

2.2 Modeling of the long-span bridge and the road vehicles

2.2.1 Stochastic traffic flow simulation

Stochastic traffic flow modeling is an important initial step in the development of the analytical framework of the bridge-traffic interaction system. In this study, the two-lane cellular automaton model is adopted to simulate the instantaneous behavior of vehicles temporally and spatially (Nagel and Schreckenberg 1992; Barlovic et al. 1998). As a mathematical idealization of physical systems with discrete time and space, cellular automaton consists of a finite set of discrete variables to represent specific vehicle information. The discrete variables for an individual vehicle in the two-lane traffic flow

^{*} This chapter is adapted from a published paper by the author (Zhou and Chen 2015a) with permission from ASCE.

simulated in the present study include the vehicle-occupied lane, vehicle longitudinal location, vehicle type, vehicle speed and vehicle driving direction. Each lane is divided into an amount of cells with an equal length, each of which can be empty or occupied by a single vehicle at any time instant. The velocity of a vehicle is determined as the number of cells the vehicle advances in one time step. The variables in each cell are updated based on the vehicle information in the adjacent locations and the probabilistic traffic rules regulating the accelerating, decelerating, lane changing and braking. These processes cannot be realized in the traffic flow from a simple Poisson arrival process. The maximum velocity a vehicle may experience is set based on the stochastic traffic flow simulation, e.g., traffic rules and cell variables can be found in the study (Chen and Wu, 2011) and hereby not elaborated for the brevity purpose. The cellular-automaton-based traffic flow through the bridge in a more realistic way. The randomization of the traffic flow is realized by the stochastic initial variables in the cellular of the whole system. Periodic boundary conditions are adopted in the traffic flow model, in which the total number of each type of vehicles in the roadway-bridge-roadway system remains constant (Chen and Wu, 2011).

2.2.2 Mode-based bridge model

To develop the bridge model based on modal coordinates, the bridge system is firstly established as a three-dimensional finite element model with any commercial finite element program such as SAP2000. Then, the nonlinear static analysis under initial cable strain, gravity forces and steady-state wind forces is conducted in order to determine the equilibrium position as the initial deformed state and the modal displacement matrix for the bridge system in the following dynamic analysis. Thirdly, modal analysis is carried out based on the deformed configuration from the nonlinear static analysis to derive all the frequencies and the mode shapes. Finally, a selected amount of modal displacements in the vertical, lateral and rotational directions are extracted to establish the bridge dynamic equations with reduced degrees of freedom (DOF) in the following multi-mode coupled analysis.

2.2.3 Numerical models for the road vehicles

Based on the site-specific traffic characteristics, the vehicles in the stochastic traffic can be categorized into several representative types from a variety of vehicle configurations. Although more refined categories can be adopted, existing studies suggested that three categories can represent most typical traffic with reasonable complexness (Chen and Wu 2010). For each category, the representative vehicle is modeled as a combination of several rigid bodies, wheel axles, springs and dampers. The vehicle bodies are modeled with the main rigid bodies and the suspension system of each axle is modeled as the upper springs. The elastic tires are modeled as lower springs and viscous dampers are adopted to model the energy dissipation system. The mass of the suspension system is assumed to be concentrated on the secondary rigid body at each wheel axle while the mass of the springs and dampers are assumed to be zero. Four degrees of freedom (DOFs) are assigned to each main rigid body, including two translational and two rotational DOFs. The constraint equations are applied in deriving the vehicle matrices for the heavy truck model in which a pivot is used to connect the truck tractor and the trailer. The numerical dynamic model for the heavy truck contains two main rigid bodies, three wheel axle sets, twenty-four sets of springs and dampers in either vertical or lateral direction, shown in Fig. 1. The displacement vector d_{ν} for the heavy truck model contains 19 DOFs including 8 independent vertical, 8 independent lateral and 3 independent rotational DOFs, which is defined in Eq. (2.1).

$$d_{v} = \{Z_{r1} \ \theta_{r1} \ \beta_{r1} \ Z_{r2} \ \beta_{r2} \ Z_{a1L} \ Z_{a1R} \ Z_{a2L} \ Z_{a2R} \ Z_{a3L} \ Z_{a3R} \ Y_{r1} \ Y_{r2} \ Y_{a1L} \ Y_{a1R} \ Y_{a2L} \ Y_{a2R} \ Y_{a3L} \ Y_{a3R} \}$$
(2.1)

where Z_{ri} represents the vertical displacement of the *i*th rigid body; θ_{ri} represents the rotational displacement of the *i*th rigid body in the x-z plane; β_{ri} represents the rotational displacement of the *i*th rigid body in the y-z plane; $Z_{aiL(R)}$ represents the vertical displacement of the *i*th wheel axle in the left (right) side; Y_{ri} represents the lateral displacement of the *i*th rigid body; $Y_{aiL(R)}$ represents the lateral displacement of the *i*th rigid body; $Y_{aiL(R)}$ represents the lateral displacement of the *i*th rigid body; $Y_{aiL(R)}$ represents the lateral displacement of the *i*th rigid body; $Y_{aiL(R)}$ represents the lateral displacement of the *i*th rigid body; $Y_{aiL(R)}$ represents the lateral displacement of the *i*th rigid body; $Y_{aiL(R)}$ represents the lateral displacement of the *i*th rigid body; $Y_{aiL(R)}$ represents the lateral displacement of the *i*th rigid body; $Y_{aiL(R)}$ represents the lateral displacement of the *i*th rigid body; $Y_{aiL(R)}$ represents the lateral displacement of the *i*th rigid body; $Y_{aiL(R)}$ represents the lateral displacement of the *i*th wheel axle in the left (right) side.

As shown in Fig. 1, the spring stiffness coefficient *K* and damping coefficient *C* for the springs and dampers of the vehicle models are labeled accordingly with appropriate subscripts and superscripts based on the axle number, "u"(upper) or "*l*" (lower) position, "y" (lateral)- or "z" (vertical) direction and "*L*" (left) or "*R*" (right) side of the vehicle. The dynamic models of the light trucks and cars are similar to those of the heavy trucks with fewer rigid bodies, which are not shown here for the sake of the brevity. Specifically, the numerical dynamic model for the light truck and light car consists of one main rigid body, two wheel axle sets, sixteen sets of springs and dampers vertically or laterally. The displacement vector d_v for the light truck is constituted of 12 DOFs including 5 independent vertical, 5 independent lateral and 2 independent rotational ones, as demonstrated in Eq. (2.2).



(b) Side view

Figure 2.1 The numerical dynamic model for the heavy truck with one trailer

2.3 Simulation of major service and extreme loads for the bridge-traffic system

2.3.1 Service loads

The major service loads of long-span bridges and road vehicles considered in the present study include the loads from road roughness and turbulent wind acting on both the bridge and vehicles. The simulation and application of the service dynamic loads for the bridge and road vehicles are introduced in the following.

2.3.1.1 Road roughness

The road surface roughness plays an important role in the dynamic coupling of the bridge/traffic system. The roughness on the approaching road and the bridge deck is modeled as a stationary Gaussian random process with zero mean value. The power spectral density function suggested by Huang and Wang (1992) is adopted and the road surface roughness r(x) can be generated by the spectral representation formulation, which was firstly introduced by Shinozuka and Jan (1972) and expressed in Eq. (2.3).

$$r(x) = \sum_{i=1}^{n} \sqrt{2S(\overline{\phi}_k)\Delta\overline{\phi}} \cos(2\pi\overline{\phi}_k x + \theta_k)$$
(2.3)

in which, *n* is the number of points in the inverse Fourier Transform; *x* is the location on the road surface; θ_k is the random phase angle with a uniform distribution between 0 and 2π . The transverse difference of roughness is ignored because of the relative insignificance. The dynamic excitation forces between the *i*th vehicle and the contacting element on the bridge due to the road surface roughness can be expressed in the following equation.

$$F_r^i(t) = \sum_{j=1}^{na} (K_{lzL}^j r(x_j) + C_{lzL}^j \dot{r}(x_j) V_i + K_{lzR}^j r(x_j) + C_{lzR}^j \dot{r}(x_j) V_i)$$
(2.4)

in which, $r(x_j)$ is the road surface roughness at the j^{th} axle; $\dot{r}(x_j)$ is the first order derivative of $r(x_j)$ with respect to time; V_i is the driving speed of the i^{th} vehicle; K and C are the stiffness and damping coefficients of the springs and dampers in the vehicle model, respectively; the subscripts l, z, L(R) represent lower position, vertical (z) direction and left (right) side for the springs or dampers, respectively.

2.3.1.2 Turbulent wind

The fully correlated turbulent wind along the bridge deck is simulated in this study at several uniformly distributed locations. The wind velocity time history $u_j(t)$ in the along-wind direction and $v_j(t)$ in the vertical direction at the *j*th location can be simulated in Eq. (2.5) (Deodatis 1996a).

$$u_{j}(t) = \frac{2}{\sqrt{2\pi}} \sum_{m=1}^{j} \sum_{l=1}^{N_{f}} H_{ujm}(\omega_{ml}) \sqrt{\Delta\omega} \cos(\omega_{ml}t - \theta_{jm} + \varphi_{ml})$$
(2.5a)

$$v_j(t) = \frac{2}{\sqrt{2\pi}} \sum_{m=1}^j \sum_{l=1}^{N_f} H_{vjm}(\omega_{ml}) \sqrt{\Delta\omega} \cos(\omega_{ml}t - \theta_{jm} + \varphi_{ml})$$
(2.5b)

in which, $j=1,2,...,N_s$; N_s is the total number of the simulation points along the bridge deck; $l=1,2,...,N_f$; N_f is the number of frequency intervals; $\Delta \omega = \omega_u / N_f$; ω_u is a upper cut-off frequency; $\omega_{ml} = l\Delta\omega + (m/N_s)\Delta\omega$; $\theta_{jm} = x_j / U_{wj} - x_m / U_{wm}$; x_j is the coordinate of the j^{th} simulation point; U_{wj} is the mean wind speed at the j^{th} simulation point; φ_{ml} is the random variable uniformly distributed in the interval between 0 and 2π ; $H_u(\omega)$ and $H_v(\omega)$ are the lower triangular matrices of the order $N_s \times N_s$, which are obtained from the Cholesky's decomposition of the real parts of $S_u(\omega)$ and $S_v(\omega)$, respectively; $S_u(\omega)$ and $S_v(\omega)$ are the spectral density functions of the wind turbulence in the along-wind and vertical directions, respectively. Kaimal's spectrum (Simiu and Scanlan 1978) is chosen as the target wind spectrum in both along-wind and vertical directions. Kaimal's spectrum is defined as

$$\frac{nS_u(n)}{U_*^2} = \frac{200f}{\left(1+50f\right)^{5/3}}$$
(2.6)

in which, f = nZ/U; $U_* = 0.4U/\ln(Z/Z_0)$; U is the steady-state wind speed; Z is the height of the deck above ground; Z_0 is the ground roughness.

The aerodynamic forces acting on the bridge girder are composed of the self-excited forces and the buffeting forces. The multi-mode aerodynamic analysis method is applied and implemented in the simulation platform to incorporate the buffeting forces from the turbulence and the self-excited forces from wind-structure interactions. The self-excited forces on the unit length of the bridge girder are expressed by eighteen experimentally-determined aerodynamic derivatives and the bridge motion vectors (Jain et al., 1996) and incorporated in the equations of motion in the form of aerodynamic stiffness and

damping matrices. The buffeting forces in the vertical, lateral and rotational direction on unit span of bridge are expressed in the following equations, respectively.

$$L_{b}(t) = \frac{1}{2} \rho U_{w}^{2} B(2C_{L} \chi_{Lw} \frac{u(t)}{U_{w}} + (C_{L} + C_{D}) \chi_{Lw} \frac{v(t)}{U_{w}})$$
(2.7a)

$$D_{b}(t) = \frac{1}{2} \rho U_{w}^{2} B(2C_{D} \chi_{Du} \frac{u(t)}{U_{w}} + C_{D} \chi_{Dv} \frac{v(t)}{U_{w}})$$
(2.7b)

$$M_{b}(t) = \frac{1}{2} \rho U_{w}^{2} B^{2} (2C_{M} \chi_{Mu} \frac{u(t)}{U_{w}} + C_{M} \chi_{Mv} \frac{v(t)}{U_{w}})$$
(2.7c)

in which, ρ is the air mass density; U_w is the steady-state wind speed; C_L , C_D and C_M are the lift, drag and moment coefficients from steady-state wind, respectively; C'_L , C'_D and C'_M are the first derivatives of the lift, drag and moment coefficients with respect to the wind attack angle, respectively; χ_{Lu} , χ_{Lw} , χ_{Du} , χ_{Dv} , χ_{Mu} and χ_{Mv} are the aerodynamic admittance functions, which are dependent on frequency and the bridge deck configuration.

The aerodynamic wind forces acting on the moving vehicles are determined by means of a quasistatic approach proposed by Baker (1986). The information about the aerodynamic forces and moments on the vehicles can be found in the reference (Cai and Chen, 2004) and hereby not elaborated.

2.3.2 Seismic extreme load

There are several significant extreme loads for long-span bridges, such as hurricane, earthquake, blast, fire and vehicle/ship collision. Due to the uniqueness of each extreme load, it is beyond the scope of the present study to investigate each extreme load in detail. These extreme loads, however, share some similarities in terms of modeling the loads as various time-history inputs, which can be accommodated by the proposed general platform. Without loss of generality, earthquake ground motion excitation is selected as the extreme load for demonstration in the present study because of its relative complexness and significance.

The common practice of seismic analysis for structures requires the proper representation of the seismic excitations and suitable subsequent computation of the structural response due to the input

earthquake ground motions. For the structures that have small dimensions, such as most buildings, it is reasonable to assume that the ground motions at different supports of the structure are the same considering that the dimensions of such structures are relatively small compared to the wavelengths of the seismic waves. Past studies have demonstrated that extended structures, such as long multi-span bridges and long-span bridges, should account for the spatial variability of the earthquake ground motions at multiple supports based on the fact that seismic ground motions can vary significantly over distances comparable to the length between multiple supports of extended structures (e.g., Zerva 1990; Soyluk and Dumanoglu 2004). The spatially varying earthquake ground motions are simulated as n-variate non-stationary Gaussian random process for different supports of the prototype bridge. The earthquake ground motions are implemented in terms of acceleration histories in the fully-coupled bridge-traffic interaction analysis.

2.3.2.1 Generation of the evolutionary power spectrum density function (PSDF)

Earthquake ground motion records exhibit strong nonstationarity therefore modeling the nonstationary properties is an important step in the simulation of earthquake ground motions. To provide a realistic representation of a seismic wave, a modulation function is usually combined with the simulated stationary random process to represent the non-stationary characteristics (Deodatis 1996b). In this study, the nonstationary properties of the simulated earthquake ground motions are similar to a selected scenario earthquake by adopting the evolutionary power spectrum density function (PSDF) of the earthquake records (Liang et al. 2007). The evolutionary PSDF of the scenario earthquake is obtained by means of the Short-Time Fourier Transform (STFT) (Flanagan 1972) or the Wavelet Transform (WT) (Daubechies 1992) from which the time-varying frequency distribution of the nonstationary processes with a chosen window or wavelet is represented. The generation of the evolutionary PSDF using the STFT following the approach proposed by Liang et al. (2007) is briefly introduced below for demonstration. The WT can be conducted following a similar approach with a basic wavelet function to obtain the evolutionary PSDF.

The STFT of a function f(t) can be defined as the convolution integral in the following form:

$$F(\omega,t) = \int_{-\infty}^{\infty} f(\tau)h(t-\tau)e^{-i\omega\tau}d\tau$$
(2.8)

in which, h(t) is the chosen time window for the STFT.

The evolutionary PSDF of a zero mean nonstationary stochastic process can be expressed as:

$$S_{ff}(\omega,t) = \left|F(\omega,t)\right|^2 \tag{2.9}$$

Provided that the total energy remains valid as,

$$\int_{-\infty}^{\infty} \int_{-\infty}^{\infty} \left| F(\omega, t) \right|^2 dt d\omega = \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} f^2(\tau) h^2(t-\tau) d\tau dt = \int_{-\infty}^{\infty} f^2(t) dt$$
(2.10)

This total energy condition can be achieved by choosing a time window to satisfy the following condition:

$$\int_{-\infty}^{\infty} h^2(t) dt = 1$$
 (2.11)

In this study, the Gaussian time window with the standard deviation σ of 0.25 is chosen in the STFT to obtain the evolutionary PSDF, which is expressed in the following form.

$$h^{2}(t) = \frac{1}{\sqrt{2\pi\sigma}} e^{-t^{2}/2\sigma^{2}}$$
(2.12)

2.3.2.2 <u>Simulation of spatially varying earthquake ground motions</u>

The spatially varying ground motions at each support are simulated by the spectral representation method as one-dimensional, n-variate non-stationary Gaussian random process with a mean value of zero. The non-homogeneous acceleration time histories at different supports are obtained using a spectral-representation based simulation algorithm proposed by Deodatis (1996b) with the prescribed evolutionary power spectrum density function. The cross spectral density matrix for the one-dimensional, n-variate non-stationary stochastic process with components $f_1(t)$, $f_2(t)$, ..., $f_n(t)$ can be expressed in the following equation (Deodatis 1996b).

$$S(\omega,t) = \begin{bmatrix} S_{11}(\omega,t) & S_{12}(\omega,t) & \cdots & S_{1n}(\omega,t) \\ S_{21}(\omega,t) & S_{22}(\omega,t) & \cdots & S_{2n}(\omega,t) \\ \vdots & \vdots & \ddots & \vdots \\ S_{n1}(\omega,t) & S_{n2}(\omega,t) & \cdots & S_{nn}(\omega,t) \end{bmatrix}$$
(2.13a)

in which, $S_{ii}(\omega,t) = S_i(\omega,t)$; $S_i(\omega,t)$ is the evolutionary PSDF of the *i*th component $f_i(t)$; $S_{ij}(\omega,t)$ is the cross power spectral density function between the *i*th component and the *j*th component;

$$S_{ij}(\omega,t) = \Gamma_{ij} \sqrt{S_i(\omega)S_j(\omega)}$$
(2.13b)

 Γ_{ij} is the complex coherence function between the *i*th component and the *j*th component, which is usually expressed as the multiplication of the coherence function and the exponential term, expressed in the following equation (Der Kiureghian 1996).

$$\Gamma_{ij} = \gamma_{ij} \exp[i\theta_{ij}(\omega)] \tag{2.13c}$$

in which, γ_{ij} is the real value of the incoherence effect between the i^{th} component $f_i(t)$ and the j^{th} component $f_j(t)$; $\theta_{ij}(\omega)$ is the coherence phase, which is usually expressed as the summation of the coherence phase due to the wave-passage effect $\theta_{ij}^{w}(\omega)$ and site-response effect $\theta_{ij}^{s}(\omega)$.

The coherence phase due to the wave-passage effect results from the difference in the arrival time of the waves at support *i* and *j*, which is defined as (Der Kiureghian 1996)

$$\theta_{ij}^{w}(\omega) = -2\omega d_{ij} / (v_i + v_j)$$
(2.13d)

in which, d_{ij} is the distance between the i^{th} and j^{th} support; v_i and v_j are the apparent shear wave velocity beneath support *i* and *j*, respectively.

The coherence phase due to the site-response effect is attributed to the difference in the phase at two support locations *i* and *j*, which is given in the following equation (Der Kiureghian 1996).

$$\theta_{ij}^{s}(\omega) = \theta_{i}^{s}(\omega) - \theta_{j}^{s}(\omega) = \tan^{-1} \frac{\operatorname{Im}[F_{i}(\omega)F_{j}(-\omega)]}{\operatorname{Re}[F_{i}(\omega)F_{j}(-\omega)]}$$
(2.13e)

in which, $F_i(\omega)$ and $F_j(\omega)$ are the local soil frequency response function representing the filtration through soil layers for the soil site at supports *i* and *j*, respectively.

The cross density matrix is decomposed using the Cholesky's method at every frequency and time instant into the following form:

$$S(\omega, t) = H(\omega, t)H^{T^*}(\omega, t)$$
(2.14a)

in which, $H(\omega,t)$ is the lower triangle matrix from the Cholesky decomposition of $S(\omega,t)$;

$$H(\omega,t) = \begin{bmatrix} H_{11}(\omega,t) & 0 & \cdots & 0 \\ H_{21}(\omega,t) & H_{22}(\omega,t) & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ H_{n1}(\omega,t) & H_{n2}(\omega,t) & \cdots & H_{nn}(\omega,t) \end{bmatrix}$$
(2.14b)

 $H^{T^*}(\omega,t)$ is the transpose form of the complex conjugate matrix for $H(\omega,t)$.

After the cross-spectral matrix is decomposed, the non-stationary stochastic process for components $f_1(t), f_2(t), ..., f_n(t)$ can be simulated in the following equation (Deodatis 1996b).

$$f_i(t) = \frac{2}{\sqrt{2\pi}} \sum_{m}^{i} \sum_{l=1}^{N} \left| H_{im}(\omega_l, t) \right| \sqrt{\Delta\omega} \cos[\omega_l t - \theta_{im}(\omega_l, t) + \Phi_{ml}]$$
(2.15)

in which, *N* is number of division of the frequency range; $\omega_l = l\Delta\omega$, l = 1, 2, ..., N; $\Delta\omega = \omega_u/N$; ω_u represent an upper cut-off frequency; Φ_{ml} is the independent random phase angle which is distributed uniformly in the range of 0 to 2π ; $\theta_{im}(\omega_l, t) = \tan^{-1}(\operatorname{Im}(H_{im}(\omega_l, t))/\operatorname{Re}(H_{im}(\omega_l, t)))$

2.3.2.3 Implementation of spatially varying seismic loads using the modal superposition method

The equations of motion of the bridge as a lumped-mass system can be expressed as (Wilson 2002)

$$\begin{bmatrix} M_{ss} & 0\\ 0 & M_{gg} \end{bmatrix} \begin{bmatrix} \ddot{u}_s \\ \ddot{u}_g \end{bmatrix} + \begin{bmatrix} C_{ss} & C_{sg} \\ C_{gs} & C_{gg} \end{bmatrix} \begin{bmatrix} \dot{u}_s \\ \dot{u}_g \end{bmatrix} + \begin{bmatrix} K_{ss} & K_{sg} \\ K_{gs} & K_{gg} \end{bmatrix} \begin{bmatrix} u_s \\ u_g \end{bmatrix} = \begin{cases} F_s \\ F_g \end{cases}$$
(2.16)

in which, u_s and u_g are the displacement vector of the bridge structure with unconstrained degrees of freedom and the bridge base at the ground with constrained degrees of freedom, respectively; M_{ss} , K_{ss} and C_{ss} are the mass, stiffness and damping matrix of the bridge structure with unconstrained degrees of freedom; M_{gg} , K_{gg} and C_{gg} are the mass, stiffness and damping matrix of the bridge base at the ground with constrained degrees of freedom; M_{gg} , K_{gg} and C_{gg} are the mass, stiffness and damping matrix of the bridge base at the ground with constrained degrees of freedom; K_{sg} (K_{gs}) and C_{sg} (C_{gs}) are the coupled stiffness and damping matrix of the bridge structure between the unconstrained and constrained degrees of freedom, respectively; F_s and F_g are the force vector of the bridge structure and the reaction force of the bridge base at the ground.

Through the relative motion method (Léger 1990), the total displacement vector u_s is divided into the static displacement vector u_s^s and dynamic displacement vector u_s^d . Eq. (2.16) can be rearranged in terms of dynamic displacement as shown in Eq. (2.17).

$$M_{ss}\ddot{u}_{s}^{d} + C_{ss}\dot{u}_{s}^{d} + K_{ss}u_{s}^{d} = -M_{ss}R\ddot{u}_{g} - (C_{ss}R + C_{sg})\dot{u}_{g}$$
(2.17)

Considering that the damping force item from the velocity is much smaller compared with the inertial force from the acceleration and mass matrix, the second term of the equation on the right hand can be neglected (Wilson, 2002). The equations of motion can be further simplified as,

$$M_{ss}\ddot{u}_{s}^{d} + C_{ss}\dot{u}_{s}^{d} + K_{ss}u_{s}^{d} = -M_{ss}R\ddot{u}_{g}$$
(2.18)

in which, R is the influence matrix which is defined as,

$$R = -K_{ss}^{-1}K_{sg} (2.19)$$

The displacement vector u_s^d can be decomposed into the multiplication of mode shape matrix Φ and general coordinate vector q, given as

$$u_s^d = \Phi q^d \tag{2.20}$$

By multiplying both sides of Eq. (2.18) with Φ^T and rearranging, the following form can be obtained,

$$\ddot{q}^{d} + 2\zeta_{i}\omega_{i}\dot{q}^{d} + \omega_{i}^{2}q^{d} = -\frac{\Phi^{T}M_{ss}R}{\Phi^{T}M_{ss}\Phi}\ddot{u}_{g}$$
(2.21)

The seismic forces can be defined as follows,

$$F_{eq} = -\frac{\Phi^T M_{ss} R}{\Phi^T M_{ss} \Phi} \ddot{u}_g \tag{2.22}$$

which can be applied in the fully-coupled bridge-traffic interaction model through the modal superposition approach.

2.4 Fully-coupled bridge-traffic dynamic analytical model

The bridge and vehicles are modeled in two subsystems in the bridge-traffic dynamic interaction analysis system. The bridge subsystem is constructed based on the degrees of freedom (DOFs) in modal coordinates corresponding to the total number of the selected modes for the bridge. The vehicle subsystem is established on all the DOFs in physical coordinates of the vehicle numerical dynamic models. Considering that the vehicles are vibrating on the road before entering the bridge, the bridge-traffic system should include all the vehicles that are travelling on the bridge with appropriate initial conditions while entering the bridge from the roadway. Therefore, the traffic flow is simulated over the bridge and two approaching roadways at both ends of the bridge to allow for the dynamic interaction analysis of the vehicles and the approaching roadways before entering the bridge. The periodic boundary conditions are applied in the traffic flow simulation to facilitate the fully coupled dynamic analysis of the bridge and stochastic traffic system, in which all the vehicles in the roadway-bridge-roadway system are involved in the formulation of the matrices. The vehicles in the coupled equations of motion are numbered in a serial sequence depending on the occupied lane number and driving direction. The point contact theory without separation between the road surface and the tires of each vehicle is adopted in the bridge/traffic interaction model by assuming that the vehicle will not separate from the road surface when travelling through the bridge. The motion equations of the bridge-traffic system are as follows:

$$\begin{bmatrix} M_{b} & 0 & 0 & 0 \\ 0 & M_{v_{1}} & 0 & 0 \\ 0 & 0 & \ddots & 0 \\ 0 & 0 & 0 & M_{v_{n}} \end{bmatrix} \begin{bmatrix} \ddot{U}_{b} \\ \ddot{U}_{v_{1}} \\ \vdots \\ \ddot{U}_{v_{n}} \end{bmatrix} + \begin{bmatrix} C_{b} + C_{aero} + \sum_{i=1}^{n} C_{bci} & C_{b,v_{1}} & \cdots & C_{b,v_{n}} \\ C_{v_{1},b} & C_{v_{1}} & 0 & 0 \\ \vdots & 0 & \ddots & 0 \\ C_{v_{n},b} & 0 & 0 & C_{v_{n}} \end{bmatrix} \begin{bmatrix} \dot{U}_{b} \\ \dot{U}_{v_{1}} \\ \vdots \\ \dot{U}_{v_{n}} \end{bmatrix} + \begin{bmatrix} K_{b} + K_{aero} + \sum_{i=1}^{n} K_{bci} & K_{b,v_{1}} & \cdots & K_{b,v_{n}} \\ K_{v_{1},b} & K_{v_{1}} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ K_{v_{n},b} & 0 & \cdots & K_{v_{n}} \end{bmatrix} \begin{bmatrix} U_{b} \\ U_{v_{1}} \\ \vdots \\ U_{v_{n}} \end{bmatrix} = \begin{bmatrix} \sum_{i=1}^{n} F_{v_{1}}^{\ G} + F_{b}^{\ F} + F_{b}^{\ W} + F_{b}^{\ eq} \\ F_{v_{1}}^{\ F} + F_{v_{1}}^{\ W} \\ \vdots \\ F_{v_{n}}^{\ F} + F_{v_{n}}^{\ W} \end{bmatrix}$$
(2.23)

in which, M_b , K_b and C_b are the generalized mass, stiffness and damping matrices for the bridge structure, respectively; K_{aero} and C_{aero} are the aerodynamic stiffness and damping matrix based on the modal coordinates of the bridge, respectively; n is the number of vehicles travelling on the roadwaybridge-roadway system in the traffic flow; M_{v_i} , K_{v_i} and C_{v_i} are the mass, stiffness and damping matrices of the i^{th} vehicle in the traffic flow, respectively; K_{bci} and C_{bci} refer to the stiffness and damping contribution to the bridge structure due to the coupling effects between the i^{th} vehicle in the traffic flow and the bridge system, respectively; K_{b,v_i} and C_{b,v_i} are the coupled stiffness and damping matrices contributing to bridge vibration from the *i*th vehicle in the traffic flow, respectively; $K_{v_i,b}$ and $C_{v_i,b}$ are the coupled stiffness and damping matrices contributing to the vibration of the *i*th vehicle in the traffic flow from the bridge structure; U_b is a vector of generalized coordinates of the bridge corresponding to each mode involved in the analysis; U_{v_i} is a vector of the physical responses corresponding to each degree of freedom of the *i*th vehicle in the traffic flow; one-dot and two-dot superscripts of the displacement vector denote the velocity and acceleration, respectively; F_b and F_{v_i} denote the externally applied loads for the bridge in modal coordinates and the *i*th vehicle in physical coordinates, respectively. The superscripts *r*, *w*, *eq* and *G* denote the loads due to road roughness, turbulent wind, earthquake and self-weight, respectively.

The matrices for the vehicles and the coupling matrices between the bridge and vehicles are derived by the virtual work theory (Cai and Chen, 2004). The nonlinear coupling stiffness and damping matrices are time-dependent which need to be updated at each time step in the dynamic analysis by assembling the coupling matrices of the vehicles in the traffic flow. The coupling bridge/traffic dynamic equations involve a high number of DOFs due to the inclusion of all the vehicles travelling on the roadway-bridgeroadway system. In addition to the large dimensions of the system, the coupling stiffness and damping matrices vary from time to time, making the bridge-traffic system time-dependent in nature. A stable and efficient algorithm is necessary to solve the equations of motion with a converged solution. In this study, the Newmark average acceleration method is selected as the solution scheme. To facilitate demonstrating the numerical algorithm, the motion equations can be rewritten in the following form:

$$[M]_{s}\{U\} + ([C]_{s} + [C]_{coupling})\{U\} + ([K]_{s} + [K]_{coupling})\{U\} = \{F\}$$
(2.24)

in which, $[M]_s$, $[K]_s$ and $[C]_s$ are the constant uncoupling mass, stiffness and damping matrices of the bridge/traffic system, respectively; $[K]_{coupling}$ and $[C]_{coupling}$ are the time-dependent coupling stiffness and damping matrices for the contacting elements in the bridge/traffic system, respectively, as shown below.

$$[K]_{coupling} = \begin{bmatrix} \sum_{i=1}^{n} K_{bci} & K_{b,v_1} & \cdots & K_{b,v_n} \\ K_{v_1,b} & 0 & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ K_{v_n,b} & 0 & \cdots & 0 \end{bmatrix}$$
(2.25a)

$$[C]_{coupling} = \begin{bmatrix} \sum_{i=1}^{n} C_{bci} & C_{b,v_1} & \cdots & C_{b,v_n} \\ C_{v_1,b} & 0 & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ C_{v_n,b} & 0 & \cdots & 0 \end{bmatrix}$$
(2.25b)

With the time step Δt for the algorithm to move from the *i*th time step t_i to the (i+1)th time step t_{i+1} , the computing process can be expressed as follows:

$$[K]_{eff} = \frac{4}{\Delta t^2} [M]_s + [K]_s + \frac{2}{\Delta t} [C]_s + [K]^i_{coupling} + \frac{2}{\Delta t} [C]^i_{coupling}$$
(2.26a)

$$\{F\}_{eff} = \{F\}^{i} + [M]_{s} (\frac{4}{\Delta t^{2}} \{U\}^{i} + \frac{4}{\Delta t} \{\dot{U}\}^{i} + \{\dot{U}\}^{i}) + ([C]_{s} + [C]_{coupling}^{i}) (\frac{2}{\Delta t} \{U\}^{i} + \{\dot{U}\}^{i}) \quad (2.26b)$$

The response vector in the $(i+1)^{\text{th}}$ time step $\{U\}^{i+1}$ can be obtained by solving the linear equations $[K]_{eff} \{U\}^{i+1} = \{F\}_{eff}$. The velocity $\{\dot{U}\}^{i+1}$ and acceleration $\{\ddot{U}\}^{i+1}$ in the $(i+1)^{\text{th}}$ time step can be computed through the following equations.

$$\{\dot{U}\}^{i+1} = \frac{2}{\Delta t} \{U\}^{i+1} - \frac{2}{\Delta t} \{U\}^{i} - \{\dot{U}\}^{i}$$
(2.27a)

$$\{\ddot{U}\}^{i+1} = \frac{4}{\Delta t^2} \{U\}^{i+1} - \frac{4}{\Delta t^2} \{U\}^i - \frac{4}{\Delta t} \{\dot{U}\}^i - \{\ddot{U}\}^i$$
(2.27b)

By solving the fully-coupled bridge-traffic interaction model (Eq. 2.23) in time domain, the responses of the bridge and each vehicle can be directly assessed. The simulation procedure of the fully coupled bridge/traffic interaction analysis as introduced above is programmed as a computer code using MATLAB.

As illustrated in Fig. 2.2, the integrated coupled bridge-traffic interaction analysis considering the dynamic excitations from wind and earthquake ground motions is conducted in four parts: firstly, the stochastic traffic flow is simulated to generate the vehicle information in both time and space and then the mass, stiffness and damping matrices of the bridge-traffic system are developed based on the generalized coordinates for the bridge and physical coordinates for the vehicles; secondly, the dynamic excitations are simulated using spectral representation method for road surface roughness, turbulent wind speed and earthquake ground motions; thirdly, the dynamic excitations from stochastic traffic, wind and earthquake ground motions will be implemented in the fully-coupled equations of motion; lastly, the equations of motion is solved through direct integration algorithm to obtain the bridge and vehicle response with respect to time.



Figure 2.2 Flowchart of the whole analysis process of the bridge-traffic system under multiple hazards

2.5 Numerical example

2.5.1 Prototype bridge and vehicle models

The prototype cable-stayed bridge in the present study has a total length of 836.7 m, with a main span, two side spans and two approach spans as shown in Fig. 2.3. The cable-stayed bridge has a bridge deck with a constant steel twin-box cross-section, which has a width of 28 m and a height of 4.57 m. The two steel pylons have A-shaped cross-sections with a height of 103.6 m. The bridge superstructure is partially supported by the reinforced concrete bridge piers with sliding bearing at the side span. The two cable planes of the bridge are placed in a fan-shaped arrangement with 12 sparsely located cables in each cable plane. The equivalent mass and mass moments of inertia per unit length of the bridge girder are 17690 kg/m and 796050 kg·m²/m, respectively. The support locations are labeled as support 1 to 6 to represent the six locations for the spatially varying ground motion inputs, which are shown in Fig. 2.3.



Figure 2.3 Elevation view of the cable-stayed bridge system

The mechanical and sectional properties of the bridge girder, piers and the strands used for the stay cables are shown in Table 2.1. The experimentally-determined static wind coefficients and the corresponding first derivatives of the bridge section are listed in Table 2.2. The cable-stayed bridge is modeled as a three-dimensional member-based finite element model in the SAP2000 program. The bridge girder, towers and piers are modeled as beam elements considering bending, torsion and shear deformation effects. The cables are modeled with catenary cable elements, highly nonlinear ones including both the tension-stiffening effects and large-deflection effects (CSI 2008). The equivalent static soil stiffness is considered at each pier support in this study, in which a set of six stiffness coefficients are calculated for a

rigid foundation supported on flexible soil site according to FEMA 356 (FEMA 2000). The frequencies and mode shape features of the first 20 modes (excluding those of the tower and piers) for the bridge are determined by means of the eigenvalue analysis and listed in Table 2.3. In order to maximize the accuracy and consider contributions from higher modes, the first 100 modes of the bridge are adopted in the present study to develop the reduced-DOF bridge dynamic equations and the fully coupled bridge-traffic dynamic interaction model.

Properties	Unit	Girder	Piers	Stay cables
Weight density, γ	kN/m^3	74.868	23.56	74.868
Elasticity modulus, E	kN/m^2	1.999E+8	2.779E+7	1.999E+8
Poisson's ratio, ρ	-	0.27	0.2	0.27
Shear modulus, G	kN/m^2	7.872E+7	1.158E+7	7.872E+7
Yield stress, F_y	kN/m^2	2.482E+5	-	1.493E+6
Ultimate stress, F_u	kN/m^2	3.999E+5	-	1.724E+6
Compressive strength, f_c	kN/m ²	-	34474	-
Cross sectional area, A	m^2	1.113	2.61	0.0130~0.0343
Moment of inertia, vertical, <i>I_y</i>	m^4	3.101	0.542	-
Moment of inertia, lateral, I_z	m^4	41.944	0.542	-
Moment of inertia, torsional, J	m^4	2.635	1.084	-

Table 2.1 Properties of the bridge structure

Table 2.2 Static wind coefficients of the bridge deck (0° wind attack angle)

Parameter	Value
Drag coefficient C_d	0.3
Lift coefficient C_l	-0.14
Rotational coefficient C_m	-1.3
First derivative of C_d	-0.1
First derivative of C_l	4.2
First derivative of C_m	1.13

Mode number	Mode frequency (Hz)	Mode shape
1	0.4193	1 st symmetric vertical
2	0.4567	1 st symmetric lateral
4	0.5947	1 st anti-symmetric vertical
5	0.6325	2 nd symmetric lateral
6	0.7444	2 nd anti-symmetric vertical
7	1.0140	2 nd symmetric vertical
10	1.1093	2 nd anti-symmetric vertical
11	1.2017	1 st symmetric torsional
12	1.2280	3 rd symmetric vertical
13	1.4641	1 st anti-symmetric lateral
16	1.6665	3 rd anti-symmetric vertical
21	1.9917	1 st anti-symmetric torsional

Table 2.3 Dynamic properties of the representative modes of the bridge structure

The travelling vehicles in this study are classified into three types: heavy truck, light truck and light car. The dynamic parameters for each type of vehicles involved in the present study, including mass, mass moment of inertia, stiffness coefficients and damping coefficients, are listed in Table 2.4. The size parameters for each type of vehicles are listed in Table 2.5.

Parameter	Unit	Heavy truck	Light truck	Light car
Mass of rigid body 1 (M_r^1)	kg	3930	4450	1460
Pitching moment of inertia of rigid body 1 (J_{yr}^1)	$kg \cdot m^2$	6273	13274	1500
Rolling moment of inertia of rigid body 1 (J_{xr}^1)	$kg \cdot m^2$	6300	2403	450
Mass of rigid body 2 (M_r^2)	kg	15700	-	-
Pitching moment of inertia of rigid body 2 (J_{yr}^2)	$kg \cdot m^2$	75360	-	-
Rolling moment of inertia of rigid body 2 (J_{xr}^2)	$kg \cdot m^2$	25447	-	-
Mass of axle block 1 (M_{aL}^1 , M_{aR}^1)	kg	220	105	37.75
Mass of axle block 2 (M_{aL}^2 , M_{aR}^2)	kg	1500	105	37.75
Mass of axle block 3 (M_{aL}^3 , M_{aR}^3)	kg	1000	-	-
Upper vertical spring stiffness (K_{uzL}^1, K_{uzR}^1)	N/m	280000	125000	108730
Upper vertical spring stiffness (K_{uzL}^2 , K_{uzR}^2)	N/m	2470000	125000	108730

Table 2.4 Dynamic parameters of the vehicles used in the case study

Parameter	Unit	Heavy truck	Light truck	Light car
Upper vertical spring stiffness (K_{uzL}^3 , K_{uzR}^3)	N/m	4230000	-	-
Lower vertical spring stiffness (K_{lzL}^1, K_{lzR}^1)	N/m	1730000	487500	175500
Lower vertical spring stiffness (K_{lzL}^2, K_{lzR}^2)	N/m	3740000	487500	175500
Lower vertical spring stiffness (K_{lzL}^3, K_{lzR}^3)	N/m	4600000	-	-
Upper lateral spring stiffness (K_{uyL}^1, K_{uyR}^1)	N/m	210000	93750	81550
Upper lateral spring stiffness (K_{uyL}^2, K_{uyR}^2)	N/m	1852500	93750	81550
Upper lateral spring stiffness (K_{uyL}^3, K_{uyR}^3)	N/m	3172500	-	-
Lower lateral spring stiffness (K_{lyL}^1, K_{lyR}^1)	N/m	577000	162500	58500
Lower lateral spring stiffness (K_{lyL}^2, K_{lyR}^2)	N/m	1247000	162500	58500
Lower lateral spring stiffness (K_{lyL}^3, K_{lyR}^3)	N/m	1533000	-	-
Upper vertical/lateral damping coefficient $(C_{uzL}^{1}, C_{uzR}^{1}, C_{uyL}^{l}, C_{uyR}^{l})$	N∙s/m	5000	2500	727.5
Upper vertical/lateral damping coefficient $(C_{uzL}^2, C_{uzR}^2, C_{uyL}^2, C_{uyR}^2)$	N∙s/m	30000	2500	727.5
Upper vertical/lateral damping coefficient $(C_{uzL}^3, C_{uzR}^3, C_{uyL}^3, C_{uyR}^3)$	N∙s/m	40000	-	-
Lower vertical/lateral damping coefficient $(C_{lrl}^{1}, C_{lrR}^{1}, C_{lvL}^{1}, C_{lvR}^{1})$	N∙s/m	1200	1000	727.5
Lower vertical/lateral damping coefficient $(C_{lrl}^2, C_{lrR}^2, C_{lvL}^2, C_{lvR}^2)$	N∙s/m	3900	1000	727.5
Lower vertical/lateral damping coefficient $(C_{lzL}^3, C_{lzR}^3, C_{lyL}^3, C_{lyR}^3)$	N∙s/m	4300	-	

Table 2.5 Dimensions of the vehicles used in the case study

Parameter		Heavy truck	Light truck	Light car	
Distance between axle 1 and rigid body 1 (D_1)	т	1.83	1.82	1.34	
Distance between axle 2 and rigid body 1 (D_2)	т	1.83	1.82	1.34	
Distance between axle 2 and rigid body $2(D_3)$	т	3.60	-	-	
Distance between axle 3 and rigid body 2 (D_4)	т	2.60	-	-	
Distance between pin and rigid body 1 (D_5)	m	1.83	-	-	
Distance between pin and rigid body 2 (D_6)	m	3.60	-	-	
Distance between left axle and rigid body 1 (W_l)	т	1.10	0.9	0.8	

Parameter		Heavy truck	Light truck	Light car
Distance between right axle and rigid body 1 (W_2)	т	1.10	-	-
Frontal area (A)	m^2	10.0	3.81	1.96
Reference height (h_v)		2.0	1.65	1.1
Total length of vehicle (L)	т	11.6	5.84	4.54

2.5.2 Simulation of stochastic traffic flow

The total length of the roadway-bridge-roadway path is 1260 m, including two roadways with a length of 210 m each and the bridge with a length of 840 m. In the CA-based traffic flow simulation, the roadway-bridge-roadway system contains 112 cells for the bridge path and 28 cells for each of the two roadway paths with a cell length of 7.5 m. The busy stochastic traffic flow with a density of 31 vehicles/km/lane is simulated with the CA-based traffic flow simulation as introduced previously (Chen and Wu, 2011). The percentage of the three types of vehicles in the traffic flow is chosen as 20%, 30% and 50% for heavy trucks, light trucks and light cars, respectively. The simulated stochastic traffic flow is comprised of a total of 160 vehicles in the roadway-bridge-roadway path, including 32 heavy trucks, 48 light trucks and 80 light cars. The traffic flow is simulated in the inner and outer lanes in both moving directions of the bridge. The total DOFs of the bridge/traffic system are 2244, in which the first 100 DOFs are in the modal coordinates of the bridge corresponding to the first 100 modes and the later 2144 DOFs are in the physical coordinates for all the vehicles.

2.5.3 Service and extreme loads

2.5.3.1 Simulation of road surface roughness

The road roughness coefficient is taken as $20 \times 10^{-6} \text{m}^3/\text{cycle}$ for "good" roughness condition according to the International Organization for Standardization (ISO) specifications.

2.5.3.2 <u>Dynamic wind force simulation</u>

The correlated time histories of turbulent wind velocity are simulated as stationary random processes at eight locations uniformly distributed along the bridge. In this study, the steady state wind velocity of 20 m/s is used in the simulation to represent moderate wind condition at the bridge deck level. Fig. 2.4 demonstrates, as an example, the simulated turbulent wind velocity in the along-wind direction and the vertical direction at the mid-span joint of the bridge.



(b) Vertical component

Figure 2.4 Simulation of turbulent wind velocity time histories at mid-span of the bridge

The self-excited forces are expressed by 18 frequency-dependent flutter derivatives measured from wind tunnel tests (Chen et al. 2013). In the formulation of buffeting forces, the aerodynamic admittance functions related to buffeting drag forces are defined by the following equation (Davenport, 1962):

$$\chi_D^2 = \frac{2}{\left(\lambda\xi\right)^2} \left(\lambda\xi - 1 + e^{-\lambda\xi}\right)$$
(2.28)

in which, $\xi = fH/U_w$; *H* is the depth of the bridge section; λ is the decay factor which is assumed to be 7 in the study, *f* is oscillation frequency (Hz).

The aerodynamic admittance functions related to buffeting lift force (L) and moment (M) are determined using the Sears function (Sears, 1941):

$$\chi_L^2 = \chi_M^2 = \frac{1}{1 + 2\pi^2 \delta}$$
(2.29)

in which, $\delta = fB/U_w$; B is the width of the bridge section.

2.5.3.3 Earthquake ground motion input

Since the present study is to introduce the methodology of considering extreme loads, rather than to conduct a site-specific seismic analysis, only a set of earthquake ground motions from El Centro earthquake is selected for demonstration purposes. Six sets of synthetic ground motion acceleration time histories with components in three directions, i.e. longitudinal, transverse and vertical, are generated as non-stationary stochastic processes for the six different support locations as shown in Fig. 2.3, respectively. The evolutionary power spectrum density functions are obtained for the N-S component, E-W component and vertical component by means of the Short Time Fourier Transform, respectively. The N-S component, E-W component and vertical component of ground motions are applied in the transverse, longitudinal and vertical directions of the bridge, respectively. The complex incoherence function is expressed as the multiplication of the real-value coherence function and the phase angle to consider the wave-passage and site-response effect (Der Kiureghian 1996).

$$\Gamma_{ij} = \gamma_{ij}(\omega) \exp[i\theta_{ij}(\omega)] = \gamma_{ij} \exp[i\theta_{ij}^{w}(\omega) + \theta_{ij}^{s}(\omega)]$$
(2.30a)

in which, $\gamma_{ij}(\omega)$ is the real value of the incoherence effect between the *i*th component $f_i(t)$ and the *j*th component $f_j(t)$; $\theta_{ij}(\omega)$ is the coherence phase, which is usually expressed as the summation of the coherence phase due to the wave-passage effect $\theta_{ij}^w(\omega)$ and site-response effect $\theta_{ij}^s(\omega)$.

$$\theta_{ii}^{w}(\omega) = -\omega d_{ii} / v \tag{2.30b}$$

$$\theta_{ij}^{s}(\omega) = \theta_{i}^{s}(\omega) - \theta_{j}^{s}(\omega) = \tan^{-1} \frac{\operatorname{Im}[F_{i}(\omega)F_{j}(-\omega)]}{\operatorname{Re}[F_{i}(\omega)F_{j}(-\omega)]}$$
(2.30c)

 $F_i(\omega)$ and $F_j(\omega)$ are the local soil frequency response function representing the filtration through soil layers for the soil site at supports *i* and *j*, respectively.

The real-valued coherence function $\gamma_{ij}(\omega)$ that relates the spatial variation of the ground motions among different supports is chosen as the Harichandran and Vanmarcke model (Harichandran and Vanmarcke 1986).

$$\gamma_{ij}(\omega) = A \exp\left[-\frac{2d_{ij}}{\alpha\theta(\omega)}(1-A+\alpha A)\right] + (1-A) \exp\left[-\frac{2d_{ij}}{\theta(\omega)}(1-A+\alpha A)\right]$$
(2.31a)

in which, A, α are the constant model parameters; $\theta(\omega)$ is the frequency dependent spatial scale fluctuation, which is expressed as follows.

$$\theta(\omega) = \kappa \left[1 + \left(\frac{\omega}{2\pi f_0} \right)^b \right]^{-1/2}$$
(2.31b)

in which, κ , f_0 , *b* are the model parameters.

The soil frequency response function can be idealized as the Kanai-Tajimi (K-T) filter function (Clough and Penzien 2003). The supports 1, 2, 5 and 6 are assumed to be founded on the medium soil site.

$$F_{j}^{K-T}(\omega) = \frac{\omega_{j}^{2} + 2i\zeta_{j}\omega_{j}\omega}{\omega_{j}^{2} - \omega^{2} + 2i\zeta_{j}\omega_{j}\omega}$$
(2.32)

in which, ω_j and ζ_j are the soil characteristic frequency and damping ratio at support *j*, respectively.

The supports 3 and 4 are assumed to be founded on the soft soil site, while the other four supports are assumed to be founded on the medium soil site. The filter parameters, including the soil frequency and damping ratio, for the medium and soft soil types are adopted as suggested in the publication (Der Kiureghian and Neuenhofer 1992). The simulated ground motion acceleration history in the N-S, E-W and vertical directions for the support location 1 and 3 are demonstrated in Fig. 2.5.



(a) N-S component



(b) E-W component



(c) Vertical component

Figure 2.5 Simulated ground motion acceleration at support location 1 and 3

2.5.4 Bridge responses under service traffic and turbulent wind loads

The dynamic performances of the bridge system subjected to the service loads including turbulent wind and stochastic traffic are studied first. The vertical and lateral dynamic responses are investigated and compared for the three types of load scenarios: (1) both stochastic traffic and turbulent wind, (2) stochastic traffic only, and (3) turbulent wind only. Moderate turbulent wind in the along-wind and vertical directions corresponding to a steady-state wind speed of 20 m/s is applied on the bridge and vehicles during the whole simulation process. By carrying out the fully-coupled bridge/traffic interaction analysis in time domain, the bridge response at any node of the bridge finite element model can be obtained following the modal superposition concept. The vertical and lateral response histories at the middle joint of the bridge deck are demonstrated in Fig. 2.6a-b, respectively.



(b) Lateral displacements

Figure 2.6 Dynamic response histories at the middle joint of the bridge deck considering traffic and/or turbulent wind loads

It is shown in Fig. 2.6a that the vertical dynamic displacements of the bridge under both stochastic traffic and turbulent wind are not close to the simple summation of the dynamic displacements under two individual loads. The mean value of the vertical displacements is around zero when only turbulent wind excitations are applied, while below zero when stochastic traffic is considered primarily due to the self-weight of the vehicles. Stochastic traffic contributes most part of the vertical bridge response when wind is moderate (Fig. 2.6a), while the lateral dynamic responses of the bridge are primarily caused by turbulent wind (Fig. 2.6b). Comparatively, dynamic interaction effects between wind, traffic and bridge are observed to be stronger in the vertical direction than in lateral direction. The mean values of the lateral

dynamic displacements are around zero in all the three comparative cases. Figs. 2.6a and b suggest that the presence of both stochastic traffic and turbulent wind will generally cause larger vertical and lateral dynamic displacements than those under any single type of loading.



(b) Lateral displacement

Figure 2.7 Response spectra of the middle bridge joint considering traffic and/or turbulent wind loads

The response spectra of the same middle bridge joint in vertical and lateral directions are presented in Fig. 2.7a-b. It is seen in Fig. 2.7a that notable peak values for the vertical displacements occur around 0.34 Hz and 0.80 Hz, corresponding to the 1st and 2nd symmetric vertical modes, respectively, when only traffic load is applied. When turbulent wind is also applied, the 1st torsional mode is also substantially excited in addition to the 1st and 2nd symmetric vertical modes, along with some higher vertical modes being slightly excited. As shown in Fig. 2.7b, the presence of wind load will greatly excite the lateral response at frequencies of 0.44 Hz and 0.62 Hz, corresponding to the 1st and 2nd lateral modes,
respectively. When only the traffic load is applied, the magnitudes of the lateral response spectrum at the dominating frequencies are much smaller than those in the cases with applied turbulent wind loads, indicating weak resonance excited by the traffic in the lateral direction.

2.5.5 Bridge responses with traffic and earthquake loads

As discussed earlier, normal traffic usually remains on the bridge when an earthquake occurs due to the current limitation on earthquake forecasting techniques. Such a scenario with traffic and earthquake loads therefore stands for a realistic situation when wind can be practically ignored due to mildness. To take into account the initial dynamic states induced by the bridge/traffic interactions, the spatially-varying earthquake ground motions are assumed to take place at the 50 second and last for 50 seconds. The fullycoupled bridge/traffic interaction analysis is conducted to obtain the dynamic responses of the bridge and the vehicles. For comparison purposes, a group of studies are conducted to investigate the dynamic responses of the bridge under three load combinations: (1) both stochastic traffic and earthquake, (2) stochastic traffic only, and (3) earthquake only. The time histories of vertical and lateral dynamic displacements at the middle joint of the bridge deck are demonstrated in Fig. 7a-b. In the figure, a small sketch of the seismic excitation shows the time instant (50 seconds) during which the earthquake sustains.

As expected, the dynamic responses have significant increases both vertically and laterally starting from the 50 second when the earthquake happens (Fig. 2.8a-b). As shown in Fig. 2.8a, the mean values of the vertical responses in the cases with stochastic traffic are below zero due to the vehicle gravity while the mean value in the case with only seismic excitations is close to zero. The extreme values of the vertical displacements after 50 second for the three cases, i.e. traffic and earthquake, traffic only and earthquake only, are -0.204 m, -0.077 m and -0.132 m, respectively. The simultaneous presence of traffic and earthquake seems to cause considerably larger vertical displacement response than any single-load scenario. Fig. 2.8b demonstrates the lateral dynamic displacements at the mid-span of the bridge in the three comparative cases. The lateral displacements under stochastic traffic load only are negligible as compared to the other two cases with the presence of earthquake excitations. Seismic excitations

contribute majority of the bridge lateral response. Different from the vertical displacements as discussed previously, the extreme value of the lateral displacements in the case with both stochastic traffic and earthquake loads are smaller than those in the case with only earthquake excitations by 17.0 %. It is indicated that the presence of stochastic traffic suppresses the lateral responses after the earthquake occurs, denoted as "suppression effects".



(a) Vertical displacements



(b) Lateral displacements

Figure 2.8 Dynamic response histories at the middle joint of the bridge deck considering traffic and/or earthquake loads

Figs. 2.9a-b list the magnitude results of the response spectrum for the vertical and lateral displacements, respectively. The results in the three comparative cases are plotted together for comparison purposes. As shown in Fig. 2.9a, when the earthquake is not considered, there exist two peaks on the vertical response spectrum at the frequencies of 0.34 Hz and 0.78 Hz, which are in the vicinity of the 1^{st} and 2^{nd} symmetric vertical modes, respectively. After the earthquake occurs, significantly larger

peaks are observed at those two frequencies and some higher-order modes are also considerably excited, such as the 2nd anti-symmetric vertical mode, 3rd symmetric vertical mode, 1st and 2nd torsional modes (Fig. 2.9a). The scenario with combined earthquake and traffic loads causes the highest peak at the 1st symmetric vertical mode while the scenario with only earthquake excitations causes the highest spectral magnitude at the 2nd symmetric vertical mode among all the cases. As shown in Fig. 8b, the occurrence of earthquake causes strong lateral response resonance at 0.42 Hz and 0.62 Hz corresponding to the 1st and 2nd symmetric lateral modes, respectively. Such resonant effects are very weak when only traffic load is considered. Same as observed in the time-history results in Fig. 2.8, the inclusion of stochastic traffic along with earthquake will cause the suppression of the response peaks in the 2nd symmetric lateral mode.



(b) Lateral displacement

Figure 2.9 Response spectra of the middle bridge joint considering traffic and/or earthquake loads

2.5.6 Bridge responses with traffic and applied wind and earthquake load

Depending on when an earthquake actually occurs, moderate to relative strong wind may possibly exist on the bridge along with traffic at the same time. It is therefore interesting to study another realistic scenario when moderate wind loads also exist on the bridge when the earthquake occurs. Similar to the previous scenario, the earthquake ground motions are assumed to occur at 50 second to allow for non-zero initial states, which are determined after the bridge/traffic/wind system has started vibrating for a certain period of time. A steady-state wind speed of 20 m/s is considered for moderate turbulent wind load applied on both the bridge and vehicles.

The vertical and lateral displacement histories of the middle joint of the bridge deck are depicted in Figs. 2.10a-b, respectively. The dynamic displacements of the bridge with stochastic traffic and earthquake excitations are also shown in the figure for a comparison. It is found from Fig. 2.10a that the presence of turbulent wind excitations induces small increase of the vertical dynamic displacements. The dynamic coupling effect of the bridge/traffic system dominates the bridge vertical displacements before the earthquake occurs. After the earthquake occurs, the extreme values of the vertical displacements are - 0.1678 m and -0.2037 m for the case with both wind and earthquake excitations and earthquake excitation only, respectively. So the additional presence of turbulent wind on the bridge/traffic system actually suppresses the bridge dynamic vertical responses when the earthquake occurs.

It is shown in Fig. 2.10b that the lateral dynamic displacements of the bridge are mainly controlled by the dynamic wind excitations due to the weak coupling effects with the vehicles in the lateral direction before the occurrence of the earthquake. After the earthquake occurs, the bridge vertical and lateral dynamic displacements are all dominated mainly by the earthquake excitations. The extreme values of lateral responses are -0.2147 m for the case with both earthquake and wind excitations and -0.1620 m for the case with only earthquake excitations. The suppression phenomenon observed in the bridge vertical response (Fig. 2.10a) due to the additional presence of turbulent wind is not observed in the lateral responses.



(b) Lateral displacements

Figure 2.10 Dynamic response histories at the middle joint of the bridge deck considering traffic,

earthquake and/or turbulent wind loads

Spectral results as shown in Figs. 2.11a-b can provide more detailed information. The frequency contents in the two comparative cases are very similar and both vertical and lateral bridge responses are dominated by their respective first two modes. The suppression effect of the existence of wind as observed in the time-history results (Fig. 2.10a) is found to be caused by the weaker resonance of the 1st vertical mode (Fig. 2.11a). For the lateral response, the presence of wind load causes the spectral magnitude to increase at the 1st lateral mode and decrease at the 2nd lateral mode, resulting in increase of the overall lateral response.



(b) Lateral displacement

Figure 2.11 Response spectrums of the middle bridge joint considering traffic, earthquake and/or turbulent wind loads

2.5.7 Statistical analysis of the time-history bridge responses of all cases

As illustrated above, six realistic load combinations among stochastic traffic service load (T), wind turbulence service load (W), and earthquake extreme load (Eq) were studied. The statistical analysis of the time-history results of the bridge vertical and lateral displacements are conducted with the mean value, standard deviations and extreme values summarized in Fig. 2.12. As shown in Fig. 2.12a, the mean values for the vertical displacement are usually around zero except when traffic load is considered. The standard deviations have slight variations among different cases and generally have larger values when earthquake

extreme event occurs. The statistical results of the lateral response (Fig. 2.12b) show overall similar trends in terms of means and standard deviations as those of the vertical displacement, except the lateral displacement exhibits close-to-zero means for all the cases. Extreme values are usually more critical to bridge designs than means and standard deviations and the results show the largest two extreme vertical displacements occur when earthquake occurs (Fig. 2.12a). The presence of normal stochastic traffic increases vertical extreme displacement, but decreases lateral extreme displacement when earthquake occurs. The presence of moderate wind along with earthquake and normal traffic actually "helps" the bridge by slightly suppressing the vertical response, yet slightly "worsens" the situation for lateral response. In terms of the extreme displacement, the worst case scenario in the vertical direction is that neither considerable traffic nor wind exists when earthquake occurs.



(a) Vertical displacement



(b) Lateral displacement

Figure 2.12 Statistical values of the bridge vertical and lateral displacements at the mid-span

2.5.8 Vehicle responses under different combinations of dynamic loading

By means of the developed fully-coupled bridge-traffic interaction analysis model, the dynamic responses of each individual vehicle of the traffic flow can be obtained subjected to different combinations of dynamic loading. Vehicle responses are critical information to assess possible accident risks during various realistic scenarios. Although detailed accident risk assessment of vehicles is beyond the current scope, the dynamic response of the representative vehicles is studied with results presented as below. The 1st vehicle of the east-bound traffic flow is a light car, which is selected as the representative vehicle to demonstrate the results.

The vertical and lateral responses for the representative vehicle traveling on the roadway-bridgeroadway system are investigated under the different combinations among bridge-traffic (road-vehicle) interaction forces, turbulent wind loads and earthquake loads. The vehicles in the traffic flow start from the beginning of one approach roadway, then get on the bridge and exit onto the other approach roadway section. Fig. 2.13 shows the instantaneous position of the representative vehicle in the longitudinal direction versus time, while the slope of the curve at any time instant stands for the instantaneous speed. As shown in Fig. 2.13, the vehicle movement is able to exhibit various features such as constant speed, acceleration, deceleration and braking behavior, following the advanced traffic flow simulation based on realistic traffic rules. For example, a straight line with non-zero slope indicates that the vehicle is moving in a constant speed. A steeper slope in some time steps indicates that the vehicle is accelerating while a gentler slope in some other steps indicates that the vehicle is decelerating. A straight line with zero slope means that the vehicle stops movement after braking. According to the vehicle location simulated in the traffic flow, as demonstrated in Fig. 2.13, the 1st vehicle enters the bridge at the 20.5 second and leaves the bridge at the 78.50 second, corresponding to the vehicle longitudinal location of 210 m and 1050 m, respectively. For the loading cases involving earthquake excitations, the time-history responses of the vehicle can be divided into two parts: the first part from the beginning to 50 seconds when earthquake occurs, and the second part from 50 seconds to 100 seconds, during which earthquake occurs.

For vehicles, acceleration responses are usually critical due to the strong correlation with traffic safety and driver comfort assessment. Fig. 2.14 shows the vertical acceleration responses of the vehicle rigid body of the representative vehicle in three different loading cases. The time durations for the vehicle to stay on the roadways and on the bridge are marked in the figures. When the vehicles travel on the roadway, road roughness is the only excitation except for the cases that moderate wind loads are also applied. When the vehicle travels on the bridge, the absolute dynamic responses of the vehicle as shown in Fig. 2.14 also include the contribution from the response of the contacting points on the bridge deck. In addition, the presence of other vehicles of the stochastic traffic flow on the same bridge can also influence the dynamic response of the representative vehicle through complex dynamic interactions with the flexible bridge structure. The fully-coupled interaction model introduced previously can fully capture such coupling effects between individual vehicles of the stochastic traffic flow and the bridge. It is assumed in the current study that the roadways are fully rigid and unaffected by any dynamic loadings. Therefore, the dynamic coupling effects only exist when the vehicles are traveling on the bridge and no coupling effects are considered when the vehicles are traveling on the roadways.

As shown in Fig. 2.14, the representative vehicle usually experiences a sudden increase of the vertical when entering and exiting the bridge. After the earthquake occurs at the 50 second, the abrupt change of

the trend of the vertical acceleration of the representative vehicle is observed. The comparison between the vertical acceleration of the representative vehicle is made for the three cases as shown in Fig. 2.14. It is found that the peak vertical acceleration of the representative vehicle under the wind can reach to a little over 0.5 g when moderate wind is applied on the bridge. When an earthquake occurs without the presence of considerable wind loads, the peak vertical acceleration of the representative vehicle can exceed 1g. The simultaneous presence of both earthquake and moderate wind does not cause remarkable additional increase of vertical acceleration response.



Figure 2.13 The travelling path in the longitudinal direction for the representative vehicle

The time-history results of the lateral acceleration responses for the representative vehicle are demonstrated in Fig. 2.15. The lateral responses of the vehicles travelling on the roadways are only related to the applied turbulent wind forces. It is seen that different from the vertical displacement of the vehicle, the lateral displacement is dominated by the dynamic wind excitation before the earthquake occurs in the wind-only case and the case with both wind and earthquake. It is found that the representative vehicle experiences the large extreme lateral acceleration (over 1g) as well as the most abrupt change for the earthquake-only case. The addition of wind load on top of the earthquake only causes slight increase of the peak lateral acceleration response. Although detailed traffic safety assessment is not made in the present study, some general findings can be made based on the large lateral acceleration extreme value as well as the characteristics of the time-history responses. In the service load condition (e.g. wind and traffic), vehicle safety is mainly affected by the sudden applied wind force at the

entry of the bridge before the occurrence of the earthquake. After the earthquake occurs, the safety of the vehicle is mainly controlled by the earthquake excitation. As shown in Fig. 2.15, the extreme lateral acceleration can exceed 1g for the representative car, which may pose serious safety threats such as side-slipping and rollover on some vehicles, especially large trucks according to some existing studies (e.g. Chen and Chen 2010). In the present study, depending on the vehicle locations on the bridge and the magnitude of the earthquake inputs at the corresponding time instants, the dynamic responses may vary significantly among different vehicles. The proposed fully-coupled bridge-traffic interaction system offers a powerful tool for advanced traffic safety assessment through predicting detailed time-history responses of any single vehicle of the stochastic traffic flow subjected to various service and extreme loads. Detailed traffic safety assessment of passing vehicles on the bridge when earthquake occurs is beyond the scope of the present topic and deserves a separate study.



Figure 2.14 Vertical dynamic acceleration responses for the representative vehicle



Figure 2.15 Lateral dynamic acceleration responses for the representative vehicle

2.6 Conclusions

This chapter presents the general simulation methodology of fully-coupled bridge-traffic interaction analysis subjected to various service and extreme loads, such as road roughness, turbulent wind and earthquake ground motions. Compared to the existing studies, the originality of the proposed simulation strategy can be reflected in the following aspects:

(1) The proposed fully-coupled bridge-traffic interaction model directly couples the bridge system and all the individual vehicles from the stochastic traffic flow simultaneously for the first time. The complex dynamic coupling effects due to the simultaneous presence of multiple vehicles of stochastic traffic flow on the bridge can be realistically considered, hence the bridge responses are predicted with more accuracy. (2) The proposed strategy provides a general simulation platform which incorporates the dynamic analysis of the bridge-traffic system subjected to various types of service and extreme loads. Specifically, road roughness, turbulent wind and earthquake ground motions were considered in the demonstrative example. However, such a platform can also be applied to consider other extreme loads defined appropriately as the time-history excitations.

(3) The dynamic responses of each individual vehicle of the traffic flow can be directly obtained by fully considering the dynamic coupling effects among the bridge/traffic system as well as various loads. Considering the fact that the vehicle responses differ largely from each other especially when non-stationary dynamic loading is applied, the availability of all the vehicle responses from the fully-coupled analysis provides crucial information to assess driver comfort and vehicle crash risk in various loading scenarios. The simulation strategy proposed in this study possesses notable potential on evaluating the overall driving safety of traffic on long-span bridges subjected stationary and non-stationary dynamic loadings.

By means of the fully-coupled dynamic simulation methodology, the bridge-traffic system consisting of a prototype cable-stayed bridge and stochastic traffic are investigated when multiple loadings of road roughness, turbulent wind and/or earthquake ground motions are applied. The main findings regarding the bridge and vehicle dynamic responses include: (1) Stronger coupling effects are found between the bridge and passing vehicles in the vertical direction than in the lateral direction. The bridge dynamic responses under stochastic traffic and turbulent wind excitations are mainly dominated by the bridge-traffic coupling effects in the vertical direction, while by the wind excitations in the lateral direction; (2) When earthquake ground motions are involved, the dynamic displacements of the bridge and vehicles are dominated by the seismic excitations in both the vertical and lateral directions. The largest extreme vertical displacement occurs in the case when both stochastic traffic and earthquake excitations are present, while the largest extreme lateral displacement occurs in the case when only the earthquake excitations are applied; (3) The presence of traffic along with the earthquake will suppress the lateral bridge displacement while considerably increase the vertical bridge displacement. In contrast, the additional presence of wind along with the traffic and earthquake will slightly suppress the vertical bridge responses, but increase the lateral responses; (4) The proposed fully-coupled bridge-traffic interaction model can predict the dynamic response of any individual vehicle of the traffic flow. The numerical study shows the representative vehicle experiences over 0.5g and 1g extreme accelerations (both vertical and lateral) when moderate wind and earthquake are applied, respectively. It is found that considerable safety risks may exist on moving vehicles under wind and/or earthquake scenarios, which deserve further studies. As summarized above, the proposed simulation strategy can predict the global bridge performances under various service and extreme loads in most cases. Because the bridge model was developed based on selected modes using modal superposition method, it may not be able to accurately characterize the nonlinear local responses of the bridge in some extreme events with strong nonlinearity, e.g. strong earthquake. However, it is believed that the established simulation strategy has laid an important foundation to evaluate the dynamic performances of the bridge-traffic interaction system subjected to multiple dynamic loadings which are simultaneously present.

CHAPTER 3 A hybrid simulation strategy for the dynamic assessment of long-span bridges and moving traffic subjected to seismic excitations^{*}

3.1 Introduction

Compared to their counterparts with shorter spans, long-span bridges usually support major traffic corridors and become the "backbones" of modern society. As one of the most devastating hazards threatening modern transportation systems, earthquake significantly influences the integrity of the bridge structures as well as the safety of moving vehicles on the bridges. Traditionally, bridge seismic analysis is conducted by considering seismic loads only, without considering the dynamic effects from moving traffic or with very simplified model of traffic service loads. For instance, in the AASHRO LRFD code (AASHTO 2012), traffic design load is considered in the seismic extreme limit state for design purposes for short- and medium-span bridges. It is not yet clear whether the same load combination consideration can also be applied to the design of long-span bridges. Moreover, nonlinear time-history analysis is often required for detailed seismic analysis of bridge structures, yet little information is available on how to consider realistic traffic loads in the nonlinear seismic analysis.

Very limited studies have been found on long-span bridge seismic analysis considering traffic loads. Liu et al. (2011) studied the dynamic effect of a simple model of a suspension bridge considering both vertical seismic excitations and moving vehicle loads. This study showed that the response of the longspan suspension bridge can be substantially amplified by the interaction of moving loads and seismic excitations. In the study, the vehicles were simplified as moving loads, and therefore the bridge and vehicles were not physically coupled in a system and the vibrating effects from vehicles cannot be incorporated. Considering that the vehicles in the traffic flow are stochastic and may experience acceleration, deceleration and braking, the adoption of a deterministic series of vehicles may not realistically represent the traffic on the bridge during seismic analyses. Furthermore, earthquake ground

^{*} A significant portion of this chapter has been published in a conference paper (Zhou and Chen 2014b).

motions exhibit highly nonstationary characteristics, which may increase the uncertainty of the extreme response of bridge-traffic system subjected to earthquake excitations. In Chapter 2, a fully-coupled bridge-traffic interaction model was proposed by directly coupling the bridge model in the modal coordinates and all the vehicles in the physical coordinates. For the first time, the proposed simulation model considers the coupling interactions between the bridge and vehicles in a stochastic traffic flow. It can predict more realistically the dynamic effects of the bridge-traffic system subjected to multiple dynamic loadings such as wind and earthquake, especially when the bridge response is within the linear elastic range. However under the excitations of moderate to strong earthquake excitations, the bridge structure may behave nonlinearly with the occurrence of plastic deformations. Since the fully-coupled bridge-traffic interaction model is developed based on the modal coordinates and the bridge response is obtained through mode superposition approach, the inelastic behavior incurred in the analysis may not be well taken into account. In order to study seismic performance of long-span bridges with nonlinear effects, nonlinear finite element (FE) analysis is usually needed and the mode-based reduced-DOF bridge model may not be sufficient. Compared to in-house nonlinear FE software developed by some researchers, some popular commercial FE software, such as SAP2000 and ANSYS etc., has some unique strength for seismic analysis, such as well-validated nonlinear analysis module, efficient computational engine and excellent pre- and post-processing functionality. Other advantages include the easiness to follow, validate and transfer the model to other commercial software. Nevertheless for most commercial FE software, it is difficult to directly conduct the complex bridge-vehicle dynamic interaction analysis when the dynamic excitations from wind and earthquake are also incorporated.

The present study aims at developing a new hybrid time-domain simulation methodology of longspan bridges as well as moving vehicles subjected to seismic excitations by taking advantage of the strength from both the modal-based bridge-traffic interaction model and commercial FE software. Considering that the probability of simultaneous occurrence of strong wind and earthquake is low, this study considers only the no wind or very mild wind conditions. Firstly, the fully-coupled bridge-traffic interaction analysis is conducted using modal superposition method to obtain the dynamic response of the bridge and moving vehicles subjected to earthquake excitations. Secondly, the equivalent moving traffic loads (EMTL) with respect to time are further obtained based on the stiffness and damping parameters of vehicles and relative displacements of vehicles at contact points. Thirdly, the time histories of EMTL at bridge deck joints are applied on the bridge finite element model (FEM) in commercial finite element software for nonlinear seismic analysis. After the methodology is introduced, numerical investigation is carried out on the performance of the prototype long-span cable-stayed bridge and the moving vehicles subjected to the scenario earthquake ground motion.

3.2 EMTL-FE approach for nonlinear seismic analysis

Through the mode superposition of all the involved modes, the response at any location of the bridge can be obtained from the fully-coupled bridge-traffic interaction analysis. Since the fully-coupled bridge-traffic dynamic interaction model was developed from the reduced-DOF bridge model based on modal analysis results, it cannot consider complex local nonlinearity which may become significant under some extreme loads, such as earthquake or collision. Therefore, for some extreme loads, the derived bridge response from the fully coupled bridge-traffic dynamic interaction model can only provide reasonable information of the bridge global response. If detailed local response of the bridge is to be assessed under some extreme loads, nonlinear analysis of the bridge model with refined scales is often needed. This is especially true for the situations that significant nonlinearity and local damage are expected on the bridge structure. In order to accommodate some special needs of considering nonlinearity associated with extreme loads, a hybrid EMTL-FE hybrid strategy is proposed, which is introduced in the following.

3.2.1 Fully-coupled bridge-traffic interaction analysis under spatially varying earthquake ground motions

The methodology of fully-coupled bridge-traffic analysis under spatially varying earthquake ground motions has been described in Chapter 2. Some key information is briefly repeated here to facilitate the following presentation. The bridge subsystem is constructed based on the degrees of freedom (DOFs) in modal coordinates corresponding to the total number of the selected modes for the bridge. The vehicles are modeled as a combination of several rigid bodies, wheel axles, springs and dampers. The vehicle subsystem is established on all the DOFs in physical coordinates of the vehicle numerical dynamic models. It is assumed that the tires of each vehicle and road surface have point contact without separation. The roughness on the approaching road and the bridge deck is modeled as a stationary Gaussian random process with zero mean value. The motion equations in a matrix form of a bridge and traffic system can be expressed as follows:

$$\begin{bmatrix} M_{b} & 0 & 0 & 0 \\ 0 & M_{v_{1}} & 0 & 0 \\ 0 & 0 & \ddots & 0 \\ 0 & 0 & 0 & M_{v_{n}} \end{bmatrix} \begin{bmatrix} \ddot{q}_{b} \\ \ddot{U}_{v_{1}} \\ \vdots \\ \ddot{U}_{v_{n}} \end{bmatrix} + \begin{bmatrix} C_{b} + \sum_{i=1}^{n} C_{bci} & C_{b,v_{1}} & \cdots & C_{b,v_{n}} \\ C_{v_{1},b} & C_{v_{1}} & 0 & 0 \\ \vdots & 0 & \ddots & 0 \\ C_{v_{n},b} & 0 & 0 & C_{v_{n}} \end{bmatrix} \begin{bmatrix} \dot{q}_{b} \\ \dot{U}_{v_{1}} \\ \vdots \\ \dot{U}_{v_{n}} \end{bmatrix} + \begin{bmatrix} K_{b} + \sum_{i=1}^{n} K_{bci} & K_{b,v_{1}} & \cdots & K_{b,v_{n}} \\ K_{v_{1},b} & K_{v_{1}} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ K_{v_{n},b} & 0 & \cdots & K_{v_{n}} \end{bmatrix} \begin{bmatrix} q_{b} \\ U_{v_{1}} \\ \vdots \\ U_{v_{n}} \end{bmatrix} = \begin{bmatrix} F_{b}^{\ r} + F_{b}^{\ eq} \\ F_{v_{1}}^{\ r} + F_{v_{1}}^{\ r} \\ \vdots \\ F_{v_{n}}^{\ r} + F_{v_{n}}^{\ r} \end{bmatrix}$$
(3.1)

in which, M_b , K_b and C_b are the generalized mass, stiffness and damping matrices for the bridge structure, respectively; *n* is the number of vehicles travelling on the roadway-bridge-roadway system in the traffic flow; M_{v_i} , K_{v_i} and C_{v_i} are the mass, stiffness and damping matrices of the *i*th vehicle in the traffic flow, respectively; K_{bci} and C_{bci} refer to the stiffness and damping contributions to the bridge structure due to the coupling effects between the *i*th vehicle in the traffic flow and the bridge system, respectively; K_{b,v_i} and C_{b,v_i} are the coupled stiffness and damping matrices for the bridge structure corresponding to the *i*th vehicle in the traffic flow, respectively; $K_{v_i,b}$ and $C_{v_i,b}$ are the coupled stiffness and damping matrices for the *i*th vehicle in the traffic flow corresponding to the bridge structure, which are equal to the transposed matrices of K_{b,v_i} and C_{b,v_i} , respectively; q_b is a vector of generalized coordinates of the bridge corresponding to each mode involved in the analysis; U_{v_i} is a vector of the physical responses corresponding to each degree of freedom of the *i*th vehicle in the traffic flow; one-dot and two-dot superscripts of the displacement vector denote the velocity and acceleration, respectively; F_b and F_{v_i} denote the external applied loads for the bridge in modal coordinates and the *i*th vehicle in physical coordinates, respectively. The superscripts *r*, *eq* and *G* denote the loads due to road roughness, earthquake and self-weight, respectively.

The spatially-varying ground motions at different supports are simulated as one-dimensional, nvariate non-stationary Gaussian random process with a mean value of zero using the spectral representation method. The simulation of spatially varying earthquake ground motions and the application of spatially varying seismic inputs on the bridge can be referred to Sec. 2.3.2. Through the mode superposition of all the involved modes, the response at any location of the bridge can be obtained from the fully-coupled bridge-traffic interaction analysis. As discussed above, in order to accommodate some special needs of considering nonlinearity associated with extreme loads, a hybrid EMTL-FE hybrid strategy is proposed, which is introduced in the following.

3.2.2 Equivalent moving traffic loads (EMTL)

Considering that the elasticity and energy dissipation of the tires are modeled as springs and dampers in the lower location, the dynamic wheel loads acting on the bridge structure are equal to the dynamic forces of the lower springs and dampers at the contacting points. The equivalent wheel loads are obtained directly for each vehicle in the stochastic traffic flow from the time-history simulation results of the fullycoupled bridge-traffic interaction system. The vertical and lateral equivalent moving traffic loads (EMTL) for the bridge girder joints are further accumulated by distributing the equivalent wheel loads (EWL) for each vehicle linearly to the bridge girder joints both longitudinally and laterally. The EMTL for each bridge girder joint can be applied on the bridge structure in the finite element analysis under multiple loading scenarios in which both the material and geometric nonlinearities can be considered in a dynamic analysis.

The vertical equivalent wheel load (EWL) for the i^{th} vehicle is determined as the summation of the vertical equivalent dynamic wheel loads and the gravity loads as expressed in Eq. (3.2).

$$F_{ewl}^{iz}(t) = F_{edwl}^{iz}(t) + G^i$$
(3.2)

in which, G^i is gravity load of the *i*th vehicle; $F_{edwl}^{iz}(t)$ is the vertical dynamic wheel loads for the *i*th vehicle in the traffic flow at time instant *t*, which are defined as (Chen and Cai 2007):

$$F_{edwl}^{iz}(t) = \sum_{j=1}^{na} (K_{lzL}^{j} \hat{Z}_{ajL}(t) + C_{lzL}^{j} \dot{Z}_{ajL}(t) + K_{lzR}^{j} \hat{Z}_{ajR}(t) + C_{lzR}^{j} \dot{Z}_{ajR}(t))$$
(3.3)

in which, $\hat{Z}_{ajL(R)}(t)$ and $\hat{Z}_{ajL(R)}(t)$ are the relative vertical displacements and the corresponding first derivatives between the lower mass block on the vehicle at the left (right) side and the contacting point on the bridge, respectively; *na* is the total number of wheel axles for the *i*th vehicle; *K* and *C* are the stiffness and damping coefficients of the springs and dampers in the vehicle model, respectively; the subscripts *l*, *z*, L(R) represent lower position, vertical (*z*) direction and left (right) side for the springs or dampers, respectively.

The corresponding vertical equivalent wheel loads for the i^{th} vehicle in the p^{th} modal coordinate of the bridge subsystem can be expressed in the following equation.

$$F_{ewl}^{piz}(t) = \sum_{j=1}^{na} (K_{lzL}^{j} \hat{Z}_{ajL}(t) + C_{lzL}^{j} \dot{\hat{Z}}_{ajL}(t) + G_{j}^{i})(h_{j}^{pi}(t) + d_{jL}^{i}(t)\alpha_{j}^{pi}(t)) + \sum_{j=1}^{na} (K_{lzR}^{j} \hat{Z}_{ajR}(t) + C_{lzR}^{j} \dot{\hat{Z}}_{ajR}(t) + G_{j}^{i})(h_{j}^{pi}(t) + d_{jR}^{i}(t)\alpha_{j}^{pi}(t))$$
(3.4)

in which, G_j^i is the gravity load at the j^{th} axle of the i^{th} vehicle; $h_j^{pi}(t)$ and $\alpha_j^{pi}(t)$ are p^{th} modal coordinates in the vertical and rotational directions for the j^{th} axle of the i^{th} vehicle at time instant t, respectively; $d_{jL}^i(t)$ and $d_{jR}^i(t)$ are the transverse distances to the centerline for the j^{th} axle of the i^{th} vehicle at the left and right sides, respectively.

The lateral equivalent wheel loads for the i^{th} vehicle are equal to the lateral dynamic wheel loads, which can be expressed as Eq. (3.5).

$$F_{ewl}^{iy}(t) = F_{dwl}^{iy}(t) = \sum_{j=1}^{na} (K_{lyL}^j \hat{Y}_{aj}(t) + C_{lyL}^j \dot{\hat{Y}}_{aj}(t) + K_{lyR}^j \hat{Y}_{aj}(t) + C_{lyR}^j \dot{\hat{Y}}_{aj}(t))$$
(3.5)

in which, $\hat{Y}_{aj}(t)$ is the relative lateral displacement between the lower mass block on the vehicle and the contacting point on the bridge; $\dot{\hat{Y}}_{aj}(t)$ is the first derivative of the corresponding relative lateral displacement $\hat{Y}_{aj}(t)$.

The lateral equivalent wheel loads for the i^{th} vehicle can be written in the p^{th} modal coordinate of the bridge subsystem, as shown in Eq. (3.6).

$$F_{ewl}^{piy}(t) = \sum_{j=1}^{na} ((K_{lyL}^{j} \hat{Y}_{aj}(t) + C_{lyL}^{j} \dot{\hat{Y}}_{aj}(t) + K_{lyR}^{j} \hat{Y}_{aj}(t) + C_{lyR}^{j} \dot{\hat{Y}}_{aj}(t)) y_{j}^{pi}(t)$$
(3.6)

in which, $y_j^{pi}(t)$ is the p^{th} modal coordinates in the lateral direction for the j^{th} axle of the i^{th} vehicle at time instant *t*.

3.2.3 EMTL-FE hybrid strategy

The EMTL-FE hybrid strategy makes it possible to conduct nonlinear analysis by taking advantage of the strong nonlinear analysis functionality provided by commercial FE software, such as SAP2000 in this study. It is known that commercial FE software is hard to directly conduct the bridge-traffic dynamic interaction. Similar to the idea of the EDWL approach (Chen and Cai, 2007; Chen and Wu, 2010), equivalent traffic time history excitations from the stochastic traffic, including bridge-traffic dynamic interaction effects, are applied on the bridge FE model of the commercial software. The proposed strategy includes the following three steps: (1) the fully-coupled bridge-traffic interaction analysis is conducted to provide the time histories of the vertical and lateral equivalent wheel loads (EWL) for each vehicle of the traffic flow under combined extreme and service loads; (2) the EWL of each vehicle is linearly distributed in both longitudinal and transverse directions to the adjacent nodes of the bridge deck in the bridge finite element model in order to generate the cumulative traffic loads acting on the bridge deck, referred to as equivalent moving traffic loads (EMTL). The time-history excitations in vertical and lateral directions for each bridge deck node of the bridge finite element (FE) model can be defined from the equivalent moving traffic loads (EMTL); (3) extreme load excitations (e.g. seismic loads) can be applied on the bridge FE model as usual and the time-history nonlinear dynamic analysis is then conducted. Such a strategy is thus

called "EMTL-FE hybrid strategy" because it integrates fully-coupled bridge-traffic dynamic analysis model developed with MATLAB and the nonlinear finite element (FE) analysis with commercial FE software.

As discussed previously, compared to in-house finite element program, the adoption of commercial finite element software for the nonlinear analysis has some benefits such as well-calibrated accuracy, easiness for other people to follow and validate, and flexibility to transfer to other commercial finite element counterparts if needed. Although any major commercial FEM software will work, in this study, the popular commercial software SAP2000 is selected to demonstrate the EMTL-FE hybrid approach. Despite the fact that the proposed EMTL-FE strategy was inspired from the existing EDWL approach (Chen and Cai 2007; Chen and Wu 2010), it should be noted that the proposed approach is different from the existing EDWL approach in following two major aspects: (1) the proposed EMTL is obtained from the fully-coupled dynamic interaction analysis including the bridge and all the vehicles of the traffic flow at the same time, while the existing EDWL approach quantifies the equivalent wheel load of each vehicle through dynamic interaction analysis of the bridge and one single vehicle at a time; (2) the proposed approach applies the EMTL on the bridge FE model in commercial FE software enabling comprehensive nonlinear dynamic analysis under extreme loads, while the existing EDWL approach applies the EDWL on the modal-based bridge model with reduced DOF focusing on service loads only.

3.2.4 Implementation of the spatially varying seismic loads in the SAP2000 program

In the EMTL-hybrid simulation analysis, the SAP2000 program is utilized to conduct the nonlinear dynamic analysis on the bridge. To implement the spatially varying earthquake ground motions in the SAP2000 program, the seismic loading should be introduced at the bridge supports as displacement time histories instead of acceleration time histories. Numerical double integration is conducted on the non-uniform acceleration histories to obtain the displacement time histories at each support through the cumulative trapezoidal rule. To eliminate potential errors in generating the displacement time history data, the very low and very high frequency contents have been filtered. Band-pass Butterworth filter is applied

to remove frequency contents that are below 0.1 Hz and above 20 Hz. The offset at zero time and slope with respect to time axis is minimized through linear regression analysis.

3.3 Numerical demonstration

3.3.1 Prototype bridge and vehicle models

The prototype cable-stayed bridge in the present study has a total length of 836.7 m, with a main span, two side spans and two approach spans as shown in Fig. 3.1. The cable-stayed bridge has a bridge deck with a constant steel twin-box cross-section, which has a width of 28 m and a height of 4.57 m. The two steel pylons have A-shaped cross-sections with a height of 103.6 m. The bridge superstructure is partially supported by the reinforced concrete bridge piers with sliding bearing at the side span. The two cable planes of the bridge are placed in a fan-shaped arrangement with 12 sparsely located cables in each cable plane. The support locations are labeled as support 1 to 6 to represent the six locations for the spatially varying ground motion inputs, which are shown in Fig. 3.1. The west and east bounds are indicated in Fig. 3.1 in order to define the two driving directions.



Figure 3.1 Elevation view of the cable-stayed bridge system

The mechanical and sectional properties of the bridge girder, piers and the strands used for the stay cables are shown in Table 3.1. The cable-stayed bridge is modeled as a three-dimensional member-based finite element model in the SAP2000 program. The bridge girder, towers and piers are modeled as beam elements considering bending, torsion and shear deformation effects. The cables are modeled with catenary cable elements, highly nonlinear ones including both the tension-stiffening effects and large-deflection effects (CSI 2008). The equivalent static soil stiffness is considered at each pier support in this study, in which a set of six stiffness coefficients are calculated for a rigid foundation supported on flexible

soil site according to FEMA 356 (2000). The frequencies and mode shape features of the first 20 modes (excluding those of the tower and piers) for the bridge are determined by means of the eigenvalue analysis and the results are listed in Table 3.2. In order to maximize the accuracy and consider contributions from higher modes, the first 100 modes of the bridge are adopted in the present study to develop the reduced-DOF bridge dynamic equations and the fully-coupled bridge-traffic dynamic interaction model.

In the present study, the travelling vehicles are classified into three types: heavy truck, light truck and light car. The heavy truck model has two rigid bodies and three axle blocks with a total of 19 DOFs. The dynamic model for the light truck and light car has one rigid body and two axle blocks with 12 DOFs. The dynamic parameters for each type of vehicles involved in the present study, including the mass, mass moment of inertia, stiffness coefficients and damping coefficients and the size parameters for each type of vehicles can be referred to Sec. 2.5.1.

3.3.2 Simulated stochastic traffic flow

The busy stochastic traffic flow with a density of 31 vehicles/km/lane is simulated for the fullycoupled bridge-traffic interaction analysis. The percentages of the three types of vehicles in the traffic flow are chosen as 20%, 30% and 50% for heavy trucks, light trucks and light cars, respectively. The roadway-bridge-roadway system contains 112 cells for the bridge path and 28 cells for each of the two roadway paths with a cell length of 7.5 m. The total length of the roadway-bridge-roadway path is 1,260 m, including two roadways with a length of 210 m each and the bridge with a length of 840 m. The traffic flow is simulated in the inner and outer lanes in both moving directions of the bridge. When the vehicles are moving from the west to the east on the bridge as indicated in Fig. 3.1, the moving direction is defined to be positive. The simulated stochastic traffic flow is constituted of a total of 160 vehicles on the roadway-bridge-roadway path, including 32 heavy trucks, 48 light trucks and 80 light cars. The total DOFs of the bridge/traffic system are 2,244, in which the first 100 DOFs are in the modal coordinates of the bridge corresponding to the first 100 modes and the later 2,144 DOFs are in the physical coordinates for all the vehicles. The busy traffic flow is simulated stochastically with duration of 100 seconds. Figs. 3.2 (a) and (b) show the longitudinal locations of the vehicles over time in the positive direction on the inner lane or outer lane of the bridge in the traffic flow, respectively.



(a) Vehicle locations on the inner lane of the bridge



(b) Vehicle locations on the outer lane of the bridge

Figure 3.2 Vehicle locations on the bridge in the positive driving direction

 $(\bigcirc$ - heavy truck; + - light truck; • -light car)

3.3.3 Service and seismic loads

3.3.3.1 Simulation of road surface roughness

The road roughness coefficient is taken as 20×10^{-6} m³/cycle for "good" roughness condition according to the International Organization for Standardization (ISO) specifications. The length of simulation is 1,260 m with a length step of 0.1 m, including the length of the bridge and two approaching roadways. The roughness displacement on the bridge deck is shown in Fig. 3.3.





3.3.3.2 Earthquake ground motion input

Since the present study is to introduce the methodology of considering extreme loads, rather than to conduct a site-specific seismic analysis, only one set of earthquake records from Imperial Earthquake is selected as the scenario earthquake for demonstration purposes. The NS and EW components are synthesized as the horizontal component of the seismic shear wave, as shown in Fig. 3.4. The attack angle of the seismic wave is assumed to be 45 degrees to the longitudinal axis of the bridge structure. The vertical to horizontal ratio is selected as 2/3 as complied with most regulations to incorporate the vertical earthquake ground motion excitations (ASCE7 2010). Through the STFT, the evolutionary PSDF is obtained with respect to time and frequency, which is demonstrated in Fig. 3.5.



Figure 3.4 Acceleration time history of the horizontal component for Imperial scenario earthquake



Figure 3.5 Evolutionary PSDF of Imperial earthquake using STFT

Six sets of synthetic ground motion acceleration time histories for the horizontal component are generated as non-stationary stochastic processes for the six different support locations as numbered in Fig. 3.1, respectively. The N-S, E-W and vertical components of ground motions are applied in the transverse, longitudinal and vertical directions of the bridge, respectively. Incoherence effects are considered in the simulation to model the spatial variability by incorporating the complex coherence functions in the cross-spectral density matrix formulation. The coherence function $\gamma_{ij}(\omega)$ that relates the spatial variation of the

ground motions among different supports is chosen as the Harichandran and Vanmarcke model (Harichandran and Vanmarcke 1986), as expressed in the following equation.

$$\gamma_{ij}(\omega) = A \exp\left[-\frac{2d_{ij}}{\alpha\theta(\omega)}(1-A+\alpha A)\right] + (1-A)\exp\left[-\frac{2d_{ij}}{\theta(\omega)}(1-A+\alpha A)\right]$$
(3.7)

in which, A, α are the constant model parameters; $\theta(\omega)$ is the frequency dependent spatial scale fluctuation, which is expressed as follows.

$$\theta(\omega) = \kappa \left[1 + \left(\frac{\omega}{2\pi f_0} \right)^b \right]^{-1/2}$$
(3.8)

in which, κ , f_0 , b are the model parameters. In the present study, the model parameters are chosen as A = 0.636, $\alpha = 0.0186$, $\kappa = 31200 m$, $f_0 = 1.51 Hz$ and b = 2.95 (Harichandran and Vanmarcke 1986).

The soil frequency response function can be idealized as the Kanai-Tajimi (K-T) filter function to consider the contribution of only the first mode and ignore the high-frequency components of the ground motion, as shown in the following equations (Clough and Penzien 2003).

$$F_{j}^{K-T}(\omega) = \frac{\omega_{j}^{2} + 2i\zeta_{j}\omega_{j}\omega}{\omega_{j}^{2} - \omega^{2} + 2i\zeta_{j}\omega_{j}\omega}$$
(3.9)

in which, ω_j and ζ_j are the soil characteristic frequency and damping ratio at support *j*, respectively. The supports 1, 2, 5 and 6 are assumed to be founded on the medium soil site. The supports 3 and 4 are assumed to be founded on the soft soil site. The filter parameters, including the soil frequency and damping ratio, for the medium and soft soil types are adopted as suggested in the publication (Der Kiureghian and Neuenhofer 1992). The soil characteristic frequency and damping ratio for medium soil are 10.0 rad/s and 0.4, respectively. The soil characteristic frequency and damping ratio for soft soil are 2.0 rad/s and 0.2, respectively.

The simulated ground motion acceleration histories of the horizontal component for the support locations 1, 3 and 5 are demonstrated in Figs. 3.6 a, b and c, respectively. The support locations are indicated in the elevation view of the prototype bridge in Fig. 3.1.



Figure 3.6 Simulated ground motion acceleration at support locations 1, 3 and 5

Through the double integration and post-processing technique, the displacement time histories for the supports are obtained which can be used in the nonlinear dynamic analysis in the SAP2000 program. Fig. 3.7 demonstrates the ground motion displacement time histories at support locations 1, 3 and 5.



Figure 3.7 Simulated ground motion displacement at support locations 1, 3 and 5

3.3.4 *ETWL* subjected to seismic loads

The dynamic responses of the bridge-traffic system are investigated in this study under earthquake ground motion excitations. To take into account the initial dynamic states induced by the bridge-traffic interactions, the spatially varying earthquake ground motions are assumed to take place at the 50 second for time duration of 50 seconds. The fully-coupled bridge-traffic interaction analysis is conducted to obtain the dynamic responses of the bridge and the vehicles. At the same time, the cumulative loads from the moving traffic can be derived from the interaction analysis, which are called "Equivalent Moving Traffic Loads" (EMTL). Figs. 3.8 (a-b) and Figs. 3.9 (a-b) demonstrate that the EMTL results in the vertical and lateral directions for the girder joint at the middle point and quarter point of the main span with respect to time, respectively. The negative sign of the vertical EMTL for the bridge girder indicates that the loads are applied downwards to the finite element model. It is seen that the vertical bridge-traffic interaction forces are dominant along the bridge span before earthquake occurs. After earthquake occurs, the extreme lateral bridge-traffic interaction forces increase remarkably along the bridge span, while the increase of the extreme vertical bridge-traffic interaction forces largely depends on the bridge joint location, vehicle location, vehicle speed and the non-stationary earthquake excitations.



(a) Vertical EMTL



(b) Lateral EMTL

Figure 3.8 The vertical and lateral EMTL for the girder joint at the middle of the bridge main span





Figure 3.9 The vertical and lateral EMTL for the girder joint at the quarter point of the bridge main span

3.3.5 Validation of the seismic analysis using acceleration and displacement histories

For structures like long-span bridges, the consideration of spatially variability has been proved to have significant influence on the seismic response (Zerva 1990; Der Kiureghian 1996). The spatially-

varying earthquake ground motions are implemented in the developed program based on the mode superposition approach using the non-uniform acceleration histories for each support location. In the SAP2000 program, the spatially-varying earthquake ground motions can only be implemented through the displacement histories to consider the spatial variations among different supports. Linear dynamic analysis using the mode superposition is conducted in the SAP2000 program starting from un-deformed initial state. The constant modal damping with a damping ratio of 0.01 is incorporated in both programs. Validation analysis is conducted to demonstrate the effectiveness and accuracy of the transformation from acceleration time histories to displacement time histories, as well as the dynamic response from seismic analysis using the two programs.

Figs. 3.10a, b, c and d show the seismic responses at the mid-span of the bridge girder in the vertical, lateral, longitudinal and torsional directions, respectively, through the developed mode-based program and the SAP2000 program. It is seen from the time-history response of the bridge structure that the dynamic responses obtained from the two programs are close in vertical, lateral, longitudinal and torsional directions with small discrepancy in the extreme values and phase angles. The comparison results suggest that the analytical models in the developed mode-based program and the finite element model in the SAP2000 program are overall equivalent in terms of deriving global response, especially when the nonlinear effect is no significant. With the post-processing techniques for the simulated earthquake ground motions, the dynamic response shows that the transformation from acceleration to displacement maintains the general equivalency without inducing notable errors.



(a) Vertical displacement



(d) Torsional displacement



The spectral properties of the seismic responses are further analyzed in the vertical, lateral, longitudinal and torsional directions to demonstrate the equivalency of the analysis using the two programs in terms of frequency contents, as shown in Figs. 3.11a, b, c and d, respectively. The Discrete

Fourier Transformation is used to obtain the frequency contents of the dynamic response in each direction. It is seen that the frequency magnitudes at the most-excited frequencies in each directions are generally close using the two programs. In the low frequency range, the frequency magnitudes from the SAP2000 program are slightly larger than those from the developed mode-based program. The seismic responses in both the time and frequency domains obtained from the dynamic models in the developed mode-based program and the SAP2000 program are found to be similar in the corresponding directions.





(b) Lateral displacement





Figure 3.11 Spectral properties of the seismic response at the mid-span of the bridge girder in the vertical, lateral, longitudinal and torsional directions

3.3.6 Validation of EMTL-FE approach on obtaining global response

As discussed previously, the fully-coupled bridge-traffic interaction analysis can generate the bridge response at any location following the mode superposition concept. Because of the limitation associated with nonlinearity considerations of the fully-coupled bridge-traffic interaction model, the hybrid EMTL-FE approach is introduced to provide some local response details especially when nonlinearity is a concern. After the EMTL is quantified, the nodal force time-history inputs in both vertical and lateral directions can be generated and applied on the FE bridge model developed in SAP2000. The earthquake ground motions are applied to the supports of the bridge FE model in the form of displacement time histories with the built-in engine of SAP2000 to generate the bridge response. A comparison of the response results for the same bridge with both approaches is made and the dynamic response time histories at the middle joint of the bridge in the main span are demonstrated in Figs. 3.12a-d for vertical, lateral, longitudinal and torsional displacements at the mid-span joint of the bridge girder, respectively.






(d) Torsional displacement

Figure 3.12 The dynamic response histories at the middle joint using different approaches

It is shown that the dynamic responses have a significant increase in each direction starting from the 50 second when the earthquake happens. It is found from the result comparisons as shown in Fig. 3.12 that the dynamic responses show general consistency between the two different approaches, before and after the earthquake occurs. The results suggest that both approaches can predict the global bridge response of the major bridge component (e.g. girder) reasonably well. There is some small difference of the details of time-history response as shown in Fig. 3.12, of which an accurate assessment is hard to be made at this point in terms of which result is more accurate due to the lack of the field-monitored response for a comparison.

The spectral features of the dynamic responses of the bridge are investigated through the Discrete Fourier Transformation of the time-history results from both the fully-coupled analysis and the hybrid EMTL-FE approach. The Discrete Fourier Transformation is conducted for the dynamic response after earthquake occurs in each direction, respectively. Figs. 3.13 (a-d) show the magnitude results of the spectrum for the vertical, lateral, longitudinal and torsional response, respectively. The results from both the fully-coupled analysis and the hybrid EMTL-FE approach are plotted together for comparison purposes. It is seen that both approaches can capture the spectral characteristics of both vertical and lateral responses with generally similar results. As shown in Fig. 13a, there exist notable peaks on the vertical response spectrum at the frequencies of 0.40 Hz, which is in the vicinity of the 1st symmetric

vertical mode. For the lateral responses, the notable peak values in the magnitude spectrum occur at around 0.44 Hz and 0.62 Hz, which are corresponding to the 1st and 2nd symmetric lateral modes (Fig. 3.11a), respectively. The longitudinal response is mostly excited at the 1st and 2nd longitudinal mode, which are at 0.58 Hz and 0.72 Hz, respectively. The most notable peak value for the frequency magnitude in the torsional direction occurs at around the frequency of 1.19 Hz, which is corresponding to the 1st symmetric torsional mode. By taking advantage of the well-calibrated nonlinear analysis functions of the commercial software (e.g. SAP2000), it is reasonable to conclude that local response details of some typical locations (e.g. connections, columns) vulnerable to seismic can be more rationally captured with the EMTL-FE approach.





(d) Torsional displacement

Figure 3.13 Spectral properties of the dynamic response at the middle joint using different approaches

3.3.7 Nonlinear dynamic response of the bridge

The nonlinear dynamic analysis of the prototype bridge in SAP2000 starts from the deformed bridge configuration from nonlinear static analysis under gravity loads and initial cable strain. The time histories of EMTL from fully-coupled bridge-traffic interaction analysis are applied on the bridge joints to account for the dynamic effects from moving traffic. The material nonlinearity and the geometric nonlinearity due to P-delta effects and the large displacement effects are considered in the nonlinear dynamic analysis. Newmark average acceleration method is chosen as the direct integration scheme with a time step of 0.02 second. The spatially varying earthquake ground motions are assumed to take place at the 50 second for a

time duration of 50 seconds. The seismic wave is assumed to have an attack angle of 45 degrees regarding to the longitudinal axis of the bridge. The vertical ground motions are applied to the bridge structure as 2/3 of the horizontal component of the seismic wave. The dynamic responses in the vertical, lateral, longitudinal and torsional directions are compared with the corresponding response from linear modal history analysis in SAP2000, in which no geometric and material nonlinearities are considered. It is found that the vertical dynamic responses have remarkable difference from the nonlinear and linear dynamic analysis, while lateral, longitudinal and torsional responses are close in the comparing cases. The vertical displacement time histories at the mid-span joint of the bridge girder are shown in Fig. 3.14 for the nonlinear and linear dynamic response. It is noted that the linear dynamic response displayed in Fig. 3.14 was obtained as the summation of the linear dynamic response and the initial deformed displacement under gravity and initial prestressing forces from static analysis.



Figure 3.14 Vertical displacement histories at the mid-span of bridge girder by nonlinear and linear dynamic analysis

It is found that at around 70 second, the plastic hinges are observed to occur at the bridge piers connecting to bridge foundations. The bridge girder and pylons remain in the elastic range without inducing material nonlinearity during the whole simulation process. In the current simulation case, geometric nonlinearity contributes mostly the increase in the vertical nonlinear dynamic response compared with that from linear dynamic analysis. The response difference between those from nonlinear and linear dynamic analyses is more notable after earthquake occurs. It is indicated that the consideration

of nonlinearity is important in the dynamic analysis of the bridge-traffic system subjected to earthquake excitations.

3.4 Conclusions

The present study proposed a new time-domain hybrid simulation platform for the long-span bridge and traffic system subjected to earthquake ground motions. Such a hybrid platform integrates mode-based bridge-traffic interaction model with reduced degrees of freedom and nonlinear FEM commercial software SAP2000. Numerical simulation was conducted on a prototype bridge with stochastic traffic considering the excitations from road roughness and earthquake ground motions. The spatially-varying earthquake ground motions are simulated through the spectral representation approach as onedimensional, n-variate, non-stationary Gaussian random processes, in which the incoherence and wave passage effects, are considered. The fully-coupled bridge-traffic interaction analysis is firstly conducted through the developed program using mode superposition approach, in which the bridge system and all the individual vehicles from the stochastic traffic flow are coupled simultaneously considering earthquake ground motions. The equivalent moving traffic loads (EMTL) are obtained from the fully-coupled bridgetraffic interaction analysis and then applied to the joints of the bridge finite element model in the proposed hybrid EMTL-FE strategy to conduct nonlinear dynamic analysis of long-span bridges under seismic excitations when nonlinearity is a concern. In addition, considering that the EMTL includes the coupling effects in the form of wheel forces that are computationally efficient, it can also be used as the engineering application of traffic service loads for the bridge system with sufficient accuracy. Validation analysis of the EMTL-FE approach is firstly conducted through the comparative analyses using the two models subjected to simulated spatially-varying earthquake ground motions. In addition, the hybrid EMTL-FE approach is validated by comparing the time-domain and frequency-domain contents of the dynamic response using the fully-coupled program and the commercial FE program. By taking advantage of the functions from both in-house bridge/traffic dynamic interaction model and the commercial FE software, the hybrid strategy offers a useful tool to study the nonlinear response of the bridge.

CHAPTER 4 Time-progressive dynamic assessment of abrupt cable breakage events on cablestayed bridges^{*}

4.1 Introduction

Like other civil structures, long-span bridges experience increasing risk of progressive failure subjected to some hazardous loading scenarios. Although long-span bridges are usually designed with sufficient structural redundancy, particular concerns arise about the abrupt breakage of bridge cables, which may cause progressive failure such as zipper-like collapse (Starossek 2007). As the primary guideline for the design of bridge cables in the US, the Post Tensioning Institute (PTI 2007) issued the recommendations for stay cable design, testing and installation. The recommendations by PTI (PTI 2007) provide two load application methods in order to quantify the dynamic effects due to the loss of cable: one is the pseudo-dynamic method, in which the equivalent static analysis is performed with a pair of impact pseudo-dynamic forces, resulting from 2.0 times the static forces, applied at the top and the bottom anchorage locations of the ruptured cable; the other one is the nonlinear dynamic analysis, in which the dynamic cable forces due to the cable breakage are applied.

As compared to the pseudo-dynamic analysis focusing on the maximum responses during the whole process of the abrupt loss of a cable, nonlinear dynamic analysis can provide more detailed and accurate information throughout the cable breakage process. As a relatively new area, there have been a couple of studies on nonlinear dynamic analysis of cable breakage in recent years. Ruiz-Teran and Aparicio (2009) evaluated the response of under-deck cable-stayed bridges due to accidental breakage of stay cables and investigated the influence on the dynamic response due to different parameters, such as breakage time, type of breakage, load combinations, deviators and damage grade. Mozos and Aparicio (2010a, b) conducted the parametric study on the dynamic response of cable-stayed bridges subjected to a sudden failure of a cable. Ten cable-stayed bridges with various layouts of the stay and cable patterns were

^{*} This chapter is adapted from a published paper by the author (Zhou and Chen 2014a) with permission from ASCE.

studied by conducting both dynamic analysis and simplified static analysis (Mozos and Aparicio 2010a, b). Wolff and Starossek (2010) applied the pseudo-dynamic approach in the accidental cable breakage analysis and conducted the collapse analysis for a cable-stayed bridge. Numerical and experimental studies were conducted by Mozos and Aparicio (2011) to investigate the influence of rupture time on the dynamic response of a cable structure and the load-time relationship in the rupture process. Cai et al. (2012) compared four analytical procedures: linear static, nonlinear static, linear dynamic and nonlinear dynamic analyses for the cable breakage events after taking into account the initial state of a cable-stayed bridge.

Cable-breakage events on cable-stayed bridges, either accidental or intentional, can happen at any time when considerable traffic may still remain on the bridge. The dynamic impact on the bridge from the cable-breakage event, particularly the local performance of the connecting nodes of the ruptured cable, is usually very critical. Rational traffic loads are thus important in order to provide more accurate information about the performance at those critical locations on long-span bridge before, during and after the cable breakage event. In almost all the existing studies, however, traffic loads were either not considered or adopted nominal design live load as defined in the AASHTO specifications (e.g. AASHTO 2012), in the form of a uniformly distributed load or a uniformly distributed load plus axle loads. It is known that the design live load (e.g. HL-93) in the AASHTO LRFD specifications was developed from and calibrated for short-span bridges under strength limit state. Moreover, the design traffic loads serve as a convenient design tool to achieve equivalent response, but do not actually represent the realistic traffic loads and distributions on the bridge. Therefore, the application of design traffic loads calibrated for short-span bridges on cable-loss analysis of long-span bridge is not well justified. In recent years, advanced stochastic traffic flow simulation techniques have been applied on quantifying the traffic load on long-span bridges (e.g. Chen and Wu 2011), which can be used on defining the traffic loads on cableloss analysis.

This study proposes the time-progressive nonlinear dynamic analysis methodology based on SAP2000 to investigate the performance of a cable-stayed bridge subjected to the abrupt cable breakage

in the time domain. Alternate load path method is applied to simulate the abrupt breakage event, and more realistic stochastic traffic loads are considered. In order to incorporate the stochastic traffic load and the dynamic bridge-vehicle interactions, several challenges such as "dynamic" initial states with non-zero initial velocity and acceleration of the abrupt breakage are addressed. After the methodology is introduced, a prototype bridge is numerically studied and some discussions associated with the simulation process and results are made.

4.2 Nonlinear dynamic simulation methodology with stochastic traffic loads

4.2.1 Stochastic traffic loads considering bridge-traffic interactions

In order to rationally simulate the moving traffic on long-span bridges, the stochastic traffic flow in this study is simulated with the cellular automaton (CA) traffic simulation model. The CA traffic model is a type of microscopic traffic flow simulation approach, which can generate individual vehicle's behavior in both temporal and spatial domains (Nagel and Schreckenberg 1992). The CA-based traffic flow simulation was performed on a "roadway-bridge-roadway" system to replicate the stochastic traffic flow through the bridge following the approach proposed by Chen and Wu (2011). Accordingly, the number of total vehicles, the instantaneous velocity and the position of each vehicle at any time instant can be identified. The details of the traffic flow simulation including the traffic rules by which the CA model is simulated have been described in several published papers (Chen and Wu 2010; Chen and Wu 2011), hereby not repeated. It is known that the cable breakage accompanied with busy traffic will be more critical to bridge safety than other scenarios of traffic. Therefore, in order to keep a reasonable number of scenarios in this study, only busy traffic flow at a density of 50 veh/mile/lane is considered in the simulation process of a cable breakage event.

When traffic moves on a long-span bridge, the dynamic interaction between the bridge and vehicles needs to be considered in order to obtain the rational estimation of the dynamic response of the bridge (Xu and Guo 2003; Cai and Chen 2004). The dynamic wheel load ratio R (Chen and Cai 2007), defined as the ratio of dynamic wheel load to vehicle gravity, is adopted in this study to quantify the dynamic

forces on the bridge due to moving traffic. With the data generated from the CA-based traffic flow simulation, the information of each vehicle, including the vehicle speed and location at each time step, will be gathered to obtain the R values corresponding to each vehicle at different time instants as well as the corresponding locations. The total wheel load for each vehicle in the stochastic traffic flow can be defined as the summation of the dynamic wheel load and the vehicle gravity load, as defined in Eq. (4.1) (Chen and Cai 2007):

$$F_j(t) = \left(1 + R_j(t)\right) * G_j(t) \tag{4.1}$$

where, $R_j(t)$ is the dynamic wheel load ratio of vehicle *j* at time *t*; $G_j(t)$ is the gravity of the vehicle *j* at time *t*.

The proposed approach will apply the total wheel load $F_j(t)$, including the static weight and dynamic effects due to the interaction between the bridge and vehicles, in the form of time history load functions to the finite element model of the bridge.

4.2.2 Selection of time-domain simulation approach for the cable breakage event

The commercial software SAP2000 is thoughtfully chosen in the proposed study for several reasons. Compared to in-house FEM software and some other commercial software, SAP2000 is popular and reliable engineering analysis and design software, which is relatively more accessible by many researchers and practitioners. The adoption of the software allows more people to follow and take advantage of the proposed approach in order to tackle the complex cable-breakage event on bridges. In addition, the convenient and robust functionalities built in the program such as time-domain analysis, design function and nonlinearity consideration offer good flexibility on engineering design and further studies. Since SAP2000 program doesn't allow adding or removing structural members during a single time-history analysis of a certain structural system, the cable to be considered for abrupt breakage cannot be easily eliminated in the middle of the time-history simulation process. Therefore, in order to simulate the cable breakage scenario with SAP2000, two modeling procedures could be adopted: (1) modeling with the breaking cable remaining on the bridge model; and (2) modeling with the breaking cable

removed from the bridge model. Both procedures are demonstrated in Fig. 4.1 and the selection of the most appropriate one has to be made.



(a) Procedure 1: modeling with the breaking cable remaining on the bridge



(b) Procedure 2: modeling with the breaking cable removed from the bridge

Figure 4.1 Demonstration of different load application concepts in the two modeling approach

For the procedure 1, the ruptured cable will not be physically eliminated from the bridge model throughout the whole dynamic simulation period. Before the cable-breakage occurs, the cable to be considered for abrupt breakage is subjected to a pair of time-varying tension forces on the two ends, noted as $T_{11}(t)$ and $T_{21}(t)$, in which the first subscript represents the end number and the second subscript represents the procedure number (Fig. 4.1 (a)). After the breakage occurs, a pair of forces with the same magnitudes as $T_{11}(t)$ and $T_{21}(t)$ but in opposite directions will be applied at the two anchorage nodes of the ruptured cable to counteract the cable tension forces. As a result, the ruptured cable will have zero effective axial force acting on the bridge, although the cable still physically remains on the bridge after the breakage occurs. Different from the procedure 1, the cable to be considered for abrupt breakage is eliminated from the bridge during the whole dynamic simulation process in the procedure 2. Before the cable ruptures, the two anchorage nodes of the breaking cable are loaded with a pair of forces with the same magnitudes as the time-varying tension force $T_{12}(t)$ and $T_{22}(t)$ on the two ends, representing the axial loads applied on the bridge from the intact cable before the rupture occurs (Fig. 4.1 (b)). After the cable rupture occurs, the originally applied tension force $T_{12}(t)$ and $T_{22}(t)$ at the two ends can be simply removed to mimic the scenario after the cable ruptures.

The comparison between the two procedures demonstrates that each one has its advantages and also limitations. The procedure 1 has a realistic representation of the bridge before the cable ruptures, but not after the cable breakage occurs due to the inclusion of the cable element in the bridge FEM model, which actually should not remain in the bridge dynamic system after the cable breakage occurs. On the other hand, the procedure 2 can capture the reality of the cable after the breakage, but not before the cable ruptures due to the lack of the physical cable element in the bridge FEM model. Since the performance of the bridge after the cable breakage is of more concern than that before the cable ruptures, the procedure 2 seems to be more rational. However, since the procedure 2 cannot realistically represent the actual bridge before the rupture occurs, it is possible that it may not provide accurate initial states for the cable breakage event (bridge configuration immediately before the rupture occurs). Therefore a comparative study using both procedures and an evaluation of the associated error are needed in order to validate the selection, which will be conducted in the following sections.

4.2.3 *Scope of the study and assumptions*

In order to capture the most representative scenarios of the cable breakage events for cable-stayed bridges, some assumptions are made and the scope of the study is defined in the following.

4.2.3.1 <u>Dynamic loads considered in the present study</u>

Although some slender long-span cable-stayed bridges may experience significant vibrations caused by strong wind, most conventional cable-stayed bridges with bluff cross-sections and those with shorter spans are not very wind-sensitive especially under low and moderate wind speeds. Considering that the odds of having strong wind, busy traffic and cable breakage at the same time are low, the present study does not consider the wind impact on the bridge when the cable breakage event occurs. Such an assumption is valid for non-slender cable-stayed bridges and some slender ones as long as wind is mild. Due to the complexness related to the simulation of cable breakage, this assumption can also help make better observations of the mechanism associated with cable rupture and the interaction with traffic without mixing contributions from wind. For most slender long-span cable-stayed bridges or very rare cases when strong wind and cable breakage occur at the same time, a separate study considering the complex windtraffic-bridge interactions will be conducted and reported in the future.

4.2.3.2 <u>Cable breakage duration</u>

When a cable experiences abrupt breakage, the tension force in the cable is lost over a very short period of time. There is currently very little research regarding the exact duration of cable breakage. Ruiz-Teran and Aparicio (2009) conducted a parametric study on the influence of different breakage time versus fundamental period of the bridge and concluded that the maximum value of dynamic amplification factor is reached if the breakage time lasts less than one hundredth of the fundamental period of the bridge. Mozos and Aparicio (2010a, b) applied 0.005 seconds as the rupture time of a seven wire steel strand of 0.60'' after an experimental program. Mozos and Aparicio (2011) conducted rupture experiments on a seven wire strand of specific steel type with a nominal diameter of 0.6 inches. The authors concluded that the observed average rupture time is 0.0055 s for the undamaged wires. In this study, the abrupt breakage duration is assumed to be 0.01 seconds which is also the time step adopted in the numerical integration of the dynamic equations.

4.2.3.3 Single-cable breakage and the corresponding breakage initial states

The PTI recommendations require all cable-stayed bridges to withstand the sudden loss of one cable without reaching any limit states. Therefore, the abrupt breakage of single cable is the focus of the present study, which also serves an important basis of any further study on breakage of more cables. Multiple-cable breakage at the same time is beyond the current scope and deserves a separate study. As discussed earlier, the abrupt breakage of a cable, regardless of being caused by intentional attack, accidental vehicle collision, or simply cable deterioration, is usually accompanied by moving traffic. The stochastic-traffic-induced vibrations will cause dynamic response (e.g. forces, displacements or stress) of the cables, girders or other structural members varying over time. As a result, the initial states of the cable-breakage events, i.e. the bridge response (e.g. forces, displacements or moments) immediately before the cable breakage, are actually "dynamic" (time-varying) rather than "static" (time-independent) as adopted in most existing analyses. In other words, in order to have a more realistic simulation, the cable breakage event should not

occur from a static initial state (i.e. zero initial velocity and acceleration) but rather, a dynamic state with a non-zero initial velocity and acceleration. Yet how to consider the "dynamic" initial conditions caused by the stochastic traffic in the time-history simulation poses some new technical challenges for the first time, which need to be addressed.

4.2.3.4 <u>Time duration of the simulation</u>

In order to provide accurate dynamic initial conditions of a cable breakage event, the undamaged bridge with stochastic moving traffic will be simulated for the time duration t_1 . The cable rupture event occurs at t_1 second and lasts for the next 0.01 second. After the cable breaks, the simulation continues another time duration t_2 with moving stochastic traffic remaining on the bridge. According to the typical reaction time of drivers and the estimation of perception delay by the vehicle drivers far away from the ruptured cable to detect the incident, the traffic speeds and patterns are assumed not being considerably affected during 3-7 seconds immediately following the cable breakage. Therefore, the adoption of the time duration t_2 more than 10 seconds should be able to provide sufficiently long time period of information about the modified bridge system in the most critical moments following the cable breakage. Therefore, during the time duration t_2 , stochastic traffic with the same traffic density as that before the cable breakage is assumed to be present on the bridge based on the assumption that the stochastic traffic has not yet been considerably changed due to the delay of perception of response to the incident as discussed above.

4.2.4 Time-progressive dynamic simulation strategy of cable breakage with dynamic initial states

In this section, the time-progressive dynamic simulation strategy of cable breakage is described in details. As discussed above, the nonlinear dynamic analysis of the whole process in the time domain consists of three time durations: in the first duration of the process, the dynamic vibration on the undamaged bridge system is performed for a time period of t_1 seconds under the busy stochastic traffic flow; the second duration refers to the rupture time of 0.01 seconds, during which the cable has an abrupt breakage; in the third duration, the dynamic vibration is carried out on the damaged bridge system, which

starts at the time of $(t_1+0.01)$ second just after the sudden loss of a single cable and lasts for a time duration of t_2 seconds. The busy stochastic traffic load is present at the bridge system throughout all the three time intervals.

In order to simulate the whole process with the proposed methodology, the computer modeling process should be composed of two stages: the force generation stage and the breakage simulation stage. In the force generation stage, the nonlinear static analysis and the nonlinear dynamic analysis are both conducted on the undamaged bridge system to generate the static cable forces and the time histories of the dynamic cable forces at the two anchorage nodes of the ruptured cable, respectively. In the breakage simulation stage, the forces generated from the nonlinear static analysis and the force time histories from the nonlinear dynamic analysis in the previous stage will be applied appropriately on the modified bridge system. The simulation steps for both the force generation and the breakage simulation stages are introduced as follows:

Stage 1 (force generation): generating the forces for the breakage simulation

(Step 1) Conduct nonlinear static analysis on the undamaged bridge, in which the dead load and the prestress load of the cables are applied to obtain the static axial forces F_{s1} and F_{s2} on the two anchorage nodes of the breaking cable. The step impulse forces F_{s3} and F_{s4} during the last t_2 seconds have the same magnitudes as the axial forces F_{s1} and F_{s2} but in the opposite directions, respectively. This step generates a static state of the bridge named "initial state 1".

(Step 2) Perform the nonlinear dynamic analysis under stochastic traffic flow in the time domain for t_1 seconds starting from "initial State 1" at rest, to generate the force time histories of the first t_1 seconds $F_{d1}(t)$ and $F_{d2}(t)$ at two anchorage nodes of the breaking cable under only stochastic traffic.

Stage 2 (breakage simulation): simulating the cable breakage scenario

(Step 1) Remove the cable element from the original bridge model and the new bridge model is called "modified bridge". Conduct nonlinear static analysis on the modified bridge, in which the dead load and prestress load are applied. The static cable forces F_{s1} and F_{s2} generated from stage 1 are applied on the

two anchorage nodes of the cable being removed. This step generates the initial static state for the modified bridge system, noted as "initial state 2".

(Step 2) Perform the nonlinear dynamic analysis of the modified bridge system for t_1 seconds starting from "initial state 2" to determine the initial state just before the breakage with non-zero velocity and acceleration, noted as "initial state 3". During the simulation of the first t_1 seconds, the force time histories $F_{d1}(t)$ and $F_{d2}(t)$ from the Step 2 of Stage 1 are applied at the two anchorage nodes of the cable being removed and the stochastic traffic load time histories are applied to the girders in the modified bridge system at the same time.

(Step 3) Perform the nonlinear time history analysis for t_2 seconds starting from "initial state 3". In this step, the step impulse loads F_{s3} and F_{s4} as defined in Step 1 of Stage 1 (i.e. same magnitude of F_{s1} and F_{s2} from stage 1 but in the opposite directions) are applied on the two anchorage nodes of the cable being removed. The step impulse loads F_{s3} and F_{s4} are applied to counteract the static force F_{s1} and F_{s2} in each time step of the last t_2 seconds because the previously applied loads in step 1 are retained in the current dynamic simulation step. The traffic load time histories in the last t_2 seconds are also applied on the modified bridge system in this step.

The whole time-progressive simulation process for cable breakage scenarios is summarized in Fig. 4.2.



Stage 1 Generating the forces for the breakage simulation

Figure 4.2 Time-progressive dynamic simulation process of cable breakage scenarios

4.3 Numerical example and discussions

4.3.1 Bridge system

The cable-stayed bridge in the present study has a total length of 836.7 m, with a main span, two side spans and two approach spans shown in Fig. 3. Two cable planes are placed in a fan-shaped arrangement with 12 sparsely located cables in each cable plane. The cables are spaced with a distance of 53.80 m and 51.63 m for the main and the two side spans, respectively. Due to the symmetry of the cable layout, only 6 cables in one plane are involved in the following analysis as shown in Fig. 3. The bridge has steel girders with twin-box shaped cross sections and two steel pylons with A-shaped cross sections. The

mechanical properties of the bridge girder, the pylons and the strands used for the stay cables are shown in Table 4.1. The sectional properties of the bridge girder, pylons and stay cables are listed in Table 4.2.



Figure 4.3 Elevation view of the cable-stayed bridge system

Table 4.1 Mechanical properties of the steel of	on the	bridge system
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Properties	Unit	Girder and pylons	Stay cables
Weight density, γ	kN/m^3	74.868	74.868
Elasticity modulus, E	kN/m^2	1.999E+8	1.999E+8
Poisson's ratio, ρ	-	0.27	0.27
Shear modulus, G	kN/m^2	7.872E+7	7.872E+7
Yield stress, F_y	kN/m^2	2.482E+5	1.493E+6
Ultimate stress, F_u	kN/m^2	3.999E+5	1.724E+6

Table 4.2 Sectional properties of the bridge system

Properties	Unit	Girder	Pylons	Stay cables
Cross sectional area, A	m^2	1.113	0.455~1.099	0.0130~0.0343
Moment of inertia, vertical, I_y	m^4	3.101	1.144~7.068	-
Moment of inertia, lateral, I_z	m^4	41.944	1.144~7.068	-
Moment of inertia, torsional, J	m^4	2.635	1.182~4.504	-
Plastic modulus, vertical, S_{py}	m^3	0.717	0.254~1.044	-
Plastic modulus, lateral, S_{pz}	m^3	4.227	0.254~1.044	-

4.3.2 *Finite element modeling*

A three-dimensional finite element model of the prototype cable-stayed bridge is developed with the SAP2000 program. The main twin-box girder, cross girders and A-shaped pylons of the bridge are

modeled using frame elements. The stay cables are modeled as catenary cable elements in which the geometric nonlinearity is inherently considered in the nonlinear time domain analysis. Rayleigh damping is adopted in the simulation process, in which the participating factors for stiffness and mass matrix are obtained from the first two bending frequencies by assuming a damping ratio of 0.01.

4.3.3 Validation of the procedure 2 adopted in time-domain analysis

For the prototype bridge, the time duration before the breakage t_1 and the time duration after the breakage t_2 are chosen to be 15 and 14.99 seconds, respectively. As a result, the total simulation time period in this study will be 30 seconds, including 15, 0.01 and 14.99 seconds before, during and after the cable rupture event, respectively. As discussed in Sec. 2.2, the procedure 2 (i.e. with the ruptured cable physically removed from the bridge system) seems to represent more realistic situation of the modified system after the cable rupture occurs as compared to the procedure 1. To confirm the intuitive observation and quantify the associated errors, a comparative study is conducted between the results adopting the procedure 1 and the procedure 2 respectively for the same simulation period of 30 seconds, during which cable 1 has an abrupt breakage at the time of 15 second. Busy dynamic traffic loads are applied in the whole time domain analysis. Fig. 4.4 shows the comparison results in terms of the vertical displacement at the girder node connecting cable 1 (the ruptured cable) from the following three cases: dynamic simulation of undamaged bridge system under busy dynamic traffic flow without the occurrence of cable breakage, dynamic simulation of abrupt breakage of cable 1 with the presence of busy traffic loads using the procedure 1 and the procedure 2, respectively.



Figure 4.4 Response time history results of the dynamic simulation using different procedures

As shown in Fig. 4.1(b), a pair of equivalent forces is applied on the modified bridge to virtually represent the physical cables before the cable breakage (0-15 seconds) in the procedure 2. It can be found in Fig. 4.4 that the procedure 2 can actually give very close results of the displacement at the girder joint connecting to the ruptured cable before the cable ruptures (t = 0.15 seconds), as compared to the undamaged case and also the procedure 1. Specifically, the vertical displacements of the connecting girder joint on the undamaged bridge (t = 0 second) (i.e. initial states of the breakage events) are -1.511 m, -1.505 m, -1.511 m for the procedure 1, the procedure 2 and the undamaged bridge, respectively. The initial states of the vertical displacement on the damaged bridge system (t = 15 seconds, right before the cable ruptures) in the three cases are-1.566 m, -1.641 m, -1.621 m for the procedure 1, the procedure 2 and the undamaged bridge, respectively. The associated errors of the initial states generated in the procedure 2 as compared with the undamaged case (the real initial states) are 0.42% at t = 0 second (initial state 2 as defined in the section describing the time-progressive dynamic simulation strategy).

The above results indicate that by applying the equivalent forces to replace the physical cables before the cable ruptures as adopted in the procedure 2, pretty accurate initial conditions can be provided for the following cable breakage events. The comparisons of the displacements after the cable breakage (after 15 seconds) suggest that there is significant displacement amplification due to the cable breakage. Comparatively, the procedure 1 provides much less accurate estimation of the displacements after the cable breakage. Based on the comparison of the results as shown above, it becomes obvious that the errors of adopting the initial states generated through the procedure 2 are small as compared to the exact initial states. Moreover, the responses starting at the cable breakage are our major concern in order to investigate the post-incidental performance and potential risk of progressive collapse. Therefore, the procedure 2 is adopted in the hereafter simulation.

4.3.4 Influence of different dynamic breakage initial states

Cable breakage event theoretically can occur anytime when the bridge is vibrating due to the excitation of stochastic traffic. Depending on the exact instant when the breakage event occurs, the initial states of the cable breakage event (i.e. the bridge performance immediately before the event) are not only dynamic, but also vary over time in a stochastic way. To evaluate the impact from the stochastic nature of initial states, the present study performs several breakage simulation analyses of cable 1 starting from different time instants. Under the excitation from the stochastic traffic loads, the vertical bridge displacement response of the connecting girder node of cable 1 (to be considered for abrupt breakage) varies over time from a local minimum value, to the following local maximum value and then to the next local minimum value (one "cycle") in a stochastic manner. In this study, one typical response cycle is selected between the time instants of 13.02 s and 15.76 s. Within the selected response cycle, the maximum and minimum values of the bridge response are found to occur at 14.39 seconds and 13.02 seconds for the undamaged bridge system, respectively. Therefore, five initial breakage time instants within the selected cycle, which are 13.02 s, 13.82 s, 14.39 s, 15.00 s and 15.76 s, are selected to represent five breakage initial states with each of which, the cable breakage simulation analysis is conducted for 30 seconds following the methodology introduced previously. The vertical displacements at the connecting girder joint of cable 1 starting with these 5 different breakage initial states are compared in Fig. 4.5, in which the corresponding cable breakage time instants are labeled. Fig. 4.6 demonstrates the extreme response S_m for each case after the cable breakage and the response of the corresponding breakage initial state S_0 (right before the cable breakage).



Figure 4.5 Dynamic response time histories with different breakage initial states in the breakage scenario

of cable 1



Figure 4.6 The comparison responses for scenarios of breakage of cable 1 with different breakage initial

states

As shown in Fig. 4.6, the extreme dynamic response reaches the largest value in the breakage scenario starting at 14.39 seconds while the smallest value in the breakage scenario starting at 13.02 seconds. The dynamic response values with other initial breakage time (i.e. 13.82 seconds, 15.00 seconds and 15.76 seconds) lie between the largest and the smallest values. As shown in Fig. 5, at the initial breakage time of 14.39s, the bridge girder node connecting the ruptured cable is moving downwards from the upper extreme position, which is in the same direction as the impact force due to the cable breakage. Consequently, the displacement of the girder node has the largest increase after the cable ruptures due to

the "accelerating" effect. Nevertheless, at the initial breakage time of 13.02 s when the displacement has the minimum value after the cable breakage, the bridge girder node is moving upwards from the lower extreme position, which is in the opposite direction of the impact force caused by the cable breakage. Therefore, depending on the instant when the cable rupture actually occurs, the bridge response may vary considerably within a certain range. The dynamic analysis with realistic stochastic traffic load in the proposed approach enables simulations with more details such as time-varying nature of the bridge performance subjected to the cable breakage. So the cable design covering the worst-case scenarios may require more comprehensive analysis of different initial states for each cable.



Figure 4.7 The frequencies and the corresponding magnitudes in the frequency domain in the breakage scenarios of cable 1

The time-histories of the post-breakage response on the modified bridge with different initial breakage states are transformed to the spectral domain with Fourier transformation functions. The spectral frequencies and the corresponding magnitudes of the response are shown in Fig. 4.7. It is found that the

largest magnitude of the response in each breakage scenario of cable 1 occurs around the natural frequency of the first vertical bending mode of the modified bridge system. The energy in the frequency domain is concentrated in the range close to the frequencies of the first two symmetric vertical bending modes, which are 0.406 Hz and 0.951 Hz, respectively. Depending on the time instants when the cable breakage occurs, the spectral results in Fig. 4.7 show different energy distributions between the two frequencies. It also clearly shows in Fig. 4.7 that the vibrations corresponding to the frequencies of the 1st and the 2nd vertical modes are both heavily excited in the breakage event starting at 14.39 s. While for the breakage event starting at 13.02 s, both modes receive the least excitations among all the five cases. These observations are consistent with the results obtained in the time domain (Fig. 4.5).

4.3.5 *Influence of various traffic load characterizations*

Different from static traffic commonly adopted in the existing studies, stochastic moving traffic is considered in this study. The significance of stochastic traffic as compared to static traffic will be evaluated. In this section, the dynamic response of cable-stayed bridge with different types of traffic loads is analyzed using the proposed methodology through conducting nonlinear direct integration time history analysis. Three types of traffic load scenarios are considered in the analysis: no traffic load, static traffic load and stochastic traffic flow. For the static traffic load scenario, static initial states, i.e., zero initial velocity and acceleration, are adopted. The static traffic load is simulated as a uniformly distributed load along the span, which is obtained from the trial-and-error analysis in order to equate the static deflection of the mid-span under the static uniform load and the mean value of the dynamic deflection with busy stochastic traffic flow. Such an equivalency between the equivalent static load and the stochastic dynamic loads. The equivalent static traffic load for the busy traffic flow is found to be 3.867 kN/m from a series of trial-and-error assessments.

The results of the four cases (1-4) are summarized in Table 4.3 with corresponding traffic loads and initial states for the breakage event of cable 1 defined in the same table. It is not surprising to show

through the comparison between the cases with traffic (case 2-4) and the case without considering traffic (case 1) that the inclusion of traffic in the cable rupture study is necessary, as evidenced by considerably larger extreme response on the bridge before the cable rupture (initial state) ($S_0 = -1.521$ m vs. -1.694 m) and after the cable rupture ($S_m = -1.762$ m vs. -1.948 m). For each traffic case, the absolute difference between the extreme value after the cable breakage S_m and the corresponding initial breakage condition S₀ suggests that cable rupture causes pretty large amplification of the response at the bridge girder node connecting to the ruptured cable. For example, 0.472 m more displacement occurs at the same point for Case 3 as a result of the cable rupture event. The response time histories of the girder node with different traffic loads are shown in Fig. 4.8, along with the response of the undamaged bridge subjected to stochastic traffic. Cases 3-4 all adopt stochastic traffic with two different breakage initial states, which respectively correspond to the maximum and the minimum bridge response based on the analysis in the previous section. The displacement caused by the static traffic load (case 2) is between the maximum (case 3) and the minimum values (case 4) of the bridge response subjected to stochastic traffic load. So it is clear that the stochastic traffic load with dynamic initial states can provide more detailed and comprehensive information about the response on the bridge than that based on the static traffic load as adopted in most existing studies.

Cases	Service loads *	Breakage initial state	Breakage start time (s)	Initial state S ₀ (m)	Extreme value S _m (m)	S _m - S ₀ (m)
Case 1	G, P	Static	15	-1.520	-1.761	0.241
Case 2	G, P, StaT	Static	15	-1.608	-1.861	0.253
Case 3	G, P, StoT	Dynamic (maximum)	14.39	-1.477	-1.949	0.472
Case 4	G, P, StoT	Dynamic (minimum)	13.02	-1.694	-1.838	0.144

Table 4.3 Applied Service loads and breakage initial states for the comparative cases

* G-Gravity load; P-Prestress load; StaT- Static traffic; StoT- Stochastic moving traffic.



Figure 4.8 Response time histories of the girder node in the comparing cases with different types of traffic

4.3.6 Evaluation of nonlinearity involved in the analysis

In linear elastic structural analysis, deflections at the joint ends of the elements are assumed to be small enough so that the overall stiffness of the structure system remains unchanged from that of the undeformed structure. Although in principle actual structural response should be obtained from the deformed configuration of the structure, linear analysis, independent of applied load and resulting deflection, is frequently adopted in bridge analysis with the assumption of small stress and deformation. Nevertheless, in the process of progressive collapse analysis of a cable-stayed bridge, nonlinear analysis often becomes essential for both static and dynamic analyses as a result of following reasons: (1) large strains beyond the elastic limit may be triggered in the collapse process. The material will then enter the plastic range and behave nonlinearly; (2) geometric nonlinearity should not be ignored for cable elements considering the fact that cables are highly nonlinear and that the cable sag effect under its own self-weight results in nonlinear axial stiffness; (3) the interaction between the axial and flexural deformations in bending members should not be ignored because large stresses may be present in a structure and result in significantly different structural matrix formulations. For long-span cable-stayed bridges, cables usually experience large tension while bridge decks and towers experience large combined compression and bending moments. Under extreme events such as cable breakage, the bridge performance can be very

complex with possible large deformations, which warrant comprehensive investigation of various types of nonlinearity. In the present study, three sources of nonlinearity are considered: material nonlinearity, large-stress nonlinearity and large-deformation nonlinearity, among which the latter two sources of nonlinearity are believed to be the primary forms of geometric nonlinearity for cable-stayed bridges. The nonlinear analysis is carried out through imbedded iteration process of SAP2000 program and the results are discussed as below.

4.3.6.1 <u>Material nonlinearity</u>

The effects of material nonlinearity in the simulation of cable breakage events are investigated in this section. Material nonlinearity of cable elements is inherently included in a nonlinear analysis in SAP2000 program by means of the designation of nonlinear material properties. Nonlinear material behavior of frame elements is modeled using concentrated plastic hinges in the program. The coupled P-M2-M3 hinge, which yields based on the interaction of axial force and bi-axial bending moments at the hinge location, is applied to model the yielding and the post-yielding behavior of the bridge. Three hinges, at two ends and the mid-point, are inserted into each frame element to characterize the plastic deformation. The yield moment of each frame section is determined according to the Federal Emergency Management Agency guidelines (FEMA-356). In order to evaluate the effects due to material nonlinearity, the nonlinear dynamic response of the bridge model with hinges at the frame ends is compared to that of the bridge model without hinges in the loss scenario of cable 1. The comparison results indicate that no plastic hinge will form in the frames or cables under the current single-cable breakage scenario and the material in the bridge system behaves linearly in the elastic range. Similar single-cable breakage case studies are conducted for the other 5 cables respectively. All the results suggest that no plastic hinge will form for the single-cable breakage scenario of the prototype cable-stayed bridge.

4.3.6.2 Geometric nonlinearity

Geometric nonlinearity includes both large-stress nonlinearity and large-deformation nonlinearity (CSI 2008). In a cable-stayed bridge system, large-stress nonlinearity is generally expressed as tension-stiffening effects for cables and beam-column effects (P-Delta effects) for frames. Due to the sag

phenomenon under the cable self-weight, the axial stiffness of cables varies nonlinearly with the cable stresses, known as the tension-stiffening effects (CSI 2008). The nonlinear behavior due to large stress of bending members lies in the fact that the axial force-bending interaction will enable the effective bending stiffness of the frame to decrease for a compressive axial force and to increase for a tensile axial force (Nazmy and Abdel-Ghaffar, 1990). In addition to large stress nonlinearity, large deformation, i.e. deflection or rotation, may occur in the cable breakage event of a cable-stayed bridge. Therefore, the structural matrix formulations of the system need to be updated based on the deformed configurations. Since large deformation does not necessarily occur at the presence of large stress, it is usually treated as a different type of nonlinearity.

The geometric nonlinearity of cables, in terms of tension-stiffening effects and large deflection effects, is inherently included in the finite element formulation of a nonlinear analysis in the SAP2000 program due to the highly nonlinear properties of cables. The influence of geometric nonlinearity of frames is investigated in this study with the cable breakage case of cable 1. In order to demonstrate the relative significance of various nonlinearity considerations of frames to the dynamic response at the cable anchorage locations on the bridge, three comparative cases with different geometric nonlinearity consideration of P-Delta effects (large stress) and frames with the consideration of both P-Delta effects and large deformation effects. Fig. 4.9 shows graphically the time histories at the connecting girder point in the three comparative cases.



Figure 4.9 Response time histories in the cases with different geometric nonlinearity considerations of frames

It is found in Fig. 4.9 that non-negligible differences of the simulation results exist with different geometric nonlinearity considerations of frames. Since the consideration of both sources of geometric nonlinearity generates more conservative (larger) response, it is suggested that both large-deformation and large-stress nonlinearity of frames should be considered in the cable-breakage simulation process.

4.3.7 Bridge performance subjected to various abrupt single-cable breakage scenarios

After investigating several critical factors for the cable breakage event of cable 1, a comprehensive analysis of single-cable breakage scenarios for cable 2-6 is also conducted. In order to understand the impact on the bridge performance from the abrupt cable breakage, the vertical displacement time history of the respective connecting girder node with the cable where the breakage occurs is studied through the comparison with the time history of the same girder joint on the undamaged bridge.

As discussed earlier, two extreme breakage initial states are found to be at the two time instants when the bridge reaches the upper and lower extreme positions, respectively. For the breakage scenarios of each cable, the representative maximum and minimum initial states are selected during the response "cycle" of 15 second. Therefore, the maximum and minimum breakage initial states for the breakage scenarios of each cable vary correspondingly among breakage scenarios of different cables in a small range. In order to cover the whole range of possible response of the bridge due to the single-cable breakage event, the breakage simulation analyses starting with the corresponding two extreme dynamic initial states are performed for each cable breakage event (cable 1-6). For each case, the busy stochastic traffic flow is present during the whole dynamic simulation time of 30 seconds for both the undamaged and the modified bridge models. For all the scenarios, the material nonlinearity and the geometric nonlinearity for both the frame and cable elements, including the P-delta effect and large displacement, are considered in the nonlinear dynamic analysis in the time domain. It should be noted that the girder node connecting cable 6 is restrained at the support in the vertical direction, and thus the vertical displacements in the breakage scenario of cable 6 cannot be displayed.

The vertical displacement responses of the comparison figures from various single-cable breakage scenarios are plotted in Fig. 4.10 (Cable 2 and 3) and Fig. 4.11 (Cable 4 and 5). The vertical response for the cable breakage event of Cable 1 can be referred to Fig. 4.8. It is shown that the response from the modified bridge system in each scenario has little difference from that of the undamaged scenario before the cable ruptures. In other words, the initial state of each cable breakage scenario is very close to the corresponding value on the undamaged bridge. These results confirm again that the proposed approach can provide accurate initial states for the dynamic analysis of various cable breakage scenarios.



(a) Comparison results for the breakage scenario of Cable 2



(b) Comparison results for the breakage scenario of Cable 3





(a) Comparison results for the breakage scenario of Cable 4



(b) Comparison results for the breakage scenario of Cable 5

Figure 4.11 Response time histories of single cable breakage simulation in the side span

For each cable breakage event as shown in Figs. 10 and 11, significant increase of the vertical displacement at the connecting node on the bridge girder is observed after the cable ruptures. The breakage scenarios of the cables in the main span (Cable 1-3) cause much larger response than those on the side span (Cable 4-6). Similar to the results for cable 1, it is also found that the dynamic response will have the largest value if the breakage starts from the initial state (named "maximum dynamic initial state) where the girder node reaches the upmost location and starts to move downwards. While if the breakage occurs starting with the initial state when the girder node starts to move upwards, the dynamic impact from the breakage of a cable will be the least.

4.4 Response envelope analysis

Recommended by the regulations (e.g. PTI 2007), the structural response of a cable stayed bridge due to abrupt breakage of a single cable can either be obtained from a nonlinear dynamic approach or a nonlinear pseudo-dynamic approach. For the nonlinear pseudo-dynamic approach, the impact dynamic force from the loss of a cable shall be equal to a dynamic amplification factor (DAF) of 2.0 times the static force before the cable breakage occurs. The static forces at the two anchorage nodes of the breaking cable are obtained from the undamaged bridge system under the gravity load, the prestress load and the equivalent static traffic load (as defined in Sec.3.5). In this chapter, the abrupt breakage scenarios of each single cable are performed by means of the proposed nonlinear dynamic analysis, the static analysis and the pseudo-dynamic analysis following the PTI regulations, respectively. The proposed dynamic simulation approach with the maximum dynamic initial states will be applied in the loss events of each cable in order to generate response envelopes of the bridge girder. The response envelopes from the proposed dynamic method, the static analysis and the pseudo-dynamic analysis with a DAF of 2.0 are compared.

By means of different methods, the envelopes of vertical bending moments and stresses along the bridge girder of the cable-stayed bridge to undergo the abrupt loss of a single cable are demonstrated in Figs. 4.12 and 4.13, respectively. It is shown that the pseudo-dynamic method with a DAF of 2.0

recommended by the PTI guidelines can provide similar response envelopes from the nonlinear dynamic analysis for the most part along the bridge, especially at the locations between the cable anchorage points. At the locations where the cables and the girder are connected, the pseudo-dynamic method generates larger moments and provides more conservative design values. It, however, may provide values below those from the nonlinear dynamic analysis and becomes unsafe at other locations, e.g. at the mid-span. Although the pseudo-dynamic approach can reflect the overall shape of the response envelopes, it has limitations on predicting the extreme response in the abrupt loss scenario of a single cable as compared to the nonlinear dynamic approach. As shown in Figs. 4.12 and 4.13, the static analysis cannot capture the extreme values for both the moments and the stresses.



Figure 4.12 Envelopes of vertical bending moments along the bridge girder by means of different simulation methods



Figure 4.13 Envelopes of stresses along the bridge girder by means of different simulation methods

4.5 Conclusions

Cable-stayed bridges are important structures that are usually designed with adequate redundancy to withstand the sudden breakage of a single cable without reaching any limit states. The time-progressive nonlinear dynamic simulation procedures dealing with the abrupt cable breakage has been developed in this chapter. A prototype bridge with cable breakage event of each representative cable was numerically studied. Although only single-cable breakage was demonstrated in the numerical example, the proposed procedure can easily be extended to multi-cable breakage events occurring simultaneously or progressively. The originality of the proposed simulation approach can be reflected in the following aspects: (1) the stochastic dynamic traffic loads including the dynamic interaction effects between bridge and vehicles are considered during the cable breakage events for the first time; (2) the methodology adopts dynamic initial states of abrupt cable breakage occurs at a random time instant. As a result, more comprehensive and detailed information about the bridge performance during cable breakage events can be obtained than those with static initial states in the existing studies; and (3) the simulation process enables consideration of various nonlinearity (e.g. material and geometric nonlinearity) of cables and

frame elements and therefore, dynamic responses during the extreme cable breakage events can be more accurately and realistically evaluated.

After the time-progressive analysis methodology was introduced, the study investigated several critical issues associated with the proposed methodology, such as the selection of the simulation approach of cable breakage, dynamic initial states, traffic loads and nonlinearity. In order to quantify the impacts from these issues on the simulation accuracy, numerical validations and comparative studies of all representative cable-breakage events on a prototype bridge were conducted. Finally, the moment and stress response envelopes along the whole bridge girder were generated after investigating all representative single-cable breakage events. The response envelope results are compared among those with dynamic analyses under stochastic traffic loads and those from the quasi-dynamic analysis under the equivalent static traffic loads with dynamic factors suggested by PTI recommendations. The main conclusions drawn from these studies are summarized as follows:

- According to the comparative study, the proposed time-progressive analysis methodology using the procedure 2 can provide reasonable predictions of the bridge response before, during and after the cable breakage event.
- Through adopting dynamic stochastic traffic loads and the interaction effects, dynamic initial states of the cable breakage events can be considered. The studies show that the dynamic initial states can provide more comprehensive and detailed information about the impact from the cable breakage event on the bridge response than the traditional static initial states. Depending on the instant when the cable breakage occurs, the bridge girder joint linking the ruptured cable will have different response values, varying around those obtained with the static initial states.
- The comparative study shows that to consider traffic loads during cable rupture events is necessary. Depending on the instant when the cable breakage occurs, the stochastic traffic load may cause larger bridge response than that from the static traffic load, which could control the design of the cable breakage event.

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- It was found that material nonlinearity in the cables and frames is not significant and progressive collapse will not occur after the single-cable-breakage scenario for the bridge being studied.
 However, geometric nonlinearity needs to be considered in the simulation to provide more accurate results.
- The dynamic response envelopes for the stresses and vertical bending moments along the bridge were made through static method, nonlinear dynamic method and pseudo-dynamic method with a DAF of 2.0 recommended by the PTI guidelines, respectively. It is found that the pseudo-dynamic method with the DAF of 2.0 can generate the response envelope for the girder with a similar shape as that from the nonlinear dynamic analysis when equivalent static traffic load is considered. It, however, may provide values below those of the extreme response from the dynamic analysis and becomes unsafe at some locations, e.g. at the mid-span. The static analysis, however, cannot capture the extreme values for both the moment and the stress and should not be used in the response analysis of the cable breakage events.

CHAPTER 5 Framework of fully-coupled nonlinear dynamic simulation of a long-span cablestayed bridge and traffic system and application on studying cable-loss incidents^{*}

5.1 Introduction

Cable loss (breakage) is a critical extreme event for most cable-supported bridges. Post-Tensioning Institute's "PTI Recommendations for Stay Cable Design, Testing and Installation" (PTI 2007) (called "PTI recommendations" hereafter) is the primary design guidance related to cable loss, in which cablestayed bridges are required to design against single-cable loss. Cable loss can happen as a result of various causes (e.g., fire, wind, blast or deterioration), which could lead to catastrophic consequences. Similar to other extreme events, to rationally assess the bridge performance subjected to cable-loss incidents becomes pivotal for not only designing new cable-stayed bridges, but also avoiding collapse progression after initial failure of one or multiple stay cables on existing bridges. Long-span cable-stayed bridges are often built across major rivers, straits or mountains supporting important transportation with slender girders and streamlined cross-sections. As a result, strong dynamic coupling effects among longspan bridges, wind and traveling vehicles may significantly influence the local and global bridge behavior (Xu and Guo 2003; Chen and Wu 2010). Like other extreme events without sufficient warning time (e.g., barge collision, earthquake), most cable breakage events may occur when different combinations of traffic and wind loads are also present. Therefore, dynamic simulation incorporating coupling effects of the bridge-traffic-wind system becomes necessary for a rational prediction of the bridge performance subjected to these extreme events. In addition, cable-stayed bridges are generally considered as geometrically nonlinear structures due to the cable sag effect for the cables and beam-column effect for the bridge girder and towers. In an extreme event like cable breakage, existing studies have suggested that nonlinear dynamic analysis with consideration of both geometric and material nonlinearity is most suitable to trace the collapse progression (Cai et al. 2012; Mozos and Aparicio 2010a and b). Therefore, in

^{*} This chapter is adapted from a published paper by the author (Zhou and Chen 2015c) with permission from ASCE.
addition to dynamic coupling, appropriate consideration of nonlinear effects is also critical for dynamic simulation of long-span bridges, especially subjected to cable loss incidents.

In recent years, there has been some progress on considering dynamic interactions of stochastic traffic, bridge and wind for long-span bridges (e.g., Chen and Wu 2010). These studies, however, were conducted using mode-superposition method, which is not ideal for studying bridge performance subjected to extreme events with various sources of nonlinearities. Commercial finite element software SAP2000 has been used to investigate cable-loss events with only traffic service load in Chapter 4 (Zhou and Chen 2014a) by approximately considering bridge/traffic interactions with "equivalent wheel loading approach (EDWL)" (Chen and Cai 2007). Although SAP2000, like other commercial FE software, can conveniently consider nonlinearities associated with the bridge structure, complex dynamic interactions between the bridge, traffic and wind cannot be appropriately incorporated at the same time (Zhou and Chen 2014a). There is no simulation tool of cable-loss incidents which can consider the full dynamic coupling effects of the bridge-traffic-wind system and the nonlinear effects associated with cable-loss incidents of long-span bridges.

In the present study, a new nonlinear dynamic analysis framework is introduced to study the bridgetraffic-wind system subjected to cable loss based on finite-element (FE) formulation. Such an approach is innovative because it directly simulates cable-loss incidents through nonlinear iteration, in which the cable breakage process is realized by applying the counteracting forces and also physically reducing the area of the breaking cable. In this way, both the structural elemental configuration and force condition during a general cable loss incident can be characterized more realistically than existing studies. In addition, the full dynamic interactions of the bridge-traffic-wind system and also geometric and material nonlinearities are considered. A prototype bridge is numerically investigated to demonstrate the new cable-loss simulation approach. Some important factors related to cable breakage simulation process are investigated and discussed.

5.2 Fully-Coupled Bridge-Traffic-Wind Simulation based on Finite Element method

A new finite-element (FE)-based dynamic simulation platform is firstly developed using MATLAB by considering the complex dynamic interactions between the bridge structure, stochastic traffic and wind as well as associated nonlinearities in the time domain. The details of the proposed platform are introduced in the following sections.

5.2.1 Stochastic traffic flow simulation

The cellular automaton (CA) model has been a popular tool to simulate the instantaneous behavior of vehicles in the stochastic traffic flow during the past two decades (e.g., Nagel and Schreckenberg 1992; Barlovic et al. 1998). It is a computationally efficient microscopic simulation methodology in the sense that time advances in discrete steps and space is discretized into multiple cells, each of which is either empty or occupied with one vehicle at a time. By applying a set of probabilistic rules regulating accelerating, decelerating, lane changing and braking, the discrete variables in each cellular automaton are updated based on the vehicle information in the adjacent cells. Same technology applied on a long-span bridge by the second author (Chen and Wu 2010) is adopted in this study and the stochastic traffic flow is simulated over the bridge and two approaching roadways for both driving directions in order to take into account the initial vibration of vehicles on the roadways before getting on the bridge. More details are not repeated here for the sake of space and can be referred to Ref (Chen and Wu 2010).

5.2.2 Structural idealization and finite element modeling of bridge

The cable-stayed bridge system is established as a three-dimensional finite element (FE) model in this study using two types of FE elements. The bridge superstructure and pylons are modeled with nonlinear three-dimensional spatial beam elements (Reddy 2006). The stay cables are modeled with two-node catenary cable elements based on the analytical explicit solution obtained from the differential equations and boundary conditions for a cable with elastic catenary. The connections between the bridge girders and piers are simplified as pinned supports with partial release in the longitudinal direction. Rayleigh damping

is adopted in the simulation process, in which the damping coefficient for the mass and stiffness matrices are obtained by assuming a damping ratio of 0.005 for the first two fundamental modes, respectively.

5.2.3 Modeling of road vehicles

When bridge-specific traffic data is sufficiently available, the vehicles in the stochastic traffic flow can be categorized into several representative vehicle types based on the site-specific traffic characteristics. Although more categories of representative vehicles can always be assumed, three categories of typical vehicles (i.e. heavy trucks, light trucks and light cars) were found to achieve a good balance between efficiency and coverage for most general situations without detailed site-specific traffic data (Chen and Wu 2010). The representative vehicle of each category is modeled as several rigid bodies and wheel axles connected by series of springs, dampers and pivots. The upper and lower springs are used to model the suspension system of each axle and elastic tires, respectively. Viscous dampers are used to model the energy dissipation system. The mass of the suspension system is assumed to be concentrated on the secondary rigid body at each wheel axle while the springs and dampers are assumed massless. Each main rigid body contains four DOFs, including two translational and two rotational DOFs. The numerical dynamic model for the heavy truck includes two main rigid bodies, three wheel axle sets, and twenty-four sets of springs and dampers in vertical and lateral directions (Fig. 3). The displacement vector d_v for the heavy truck model is defined in Eq. (5.1) with 19 DOFs including 8 vertical, 8 lateral, 2 rolling and 1 pitching DOFs that are independent to each other.

$$d_{v} = \{Z_{r1} \ \theta_{r1} \ \beta_{r1} \ Z_{r2} \ \beta_{r2} \ Z_{a1L} \ Z_{a1R} \ Z_{a2L} \ Z_{a2R} \ Z_{a3L} \ Z_{a3R} \ Y_{r1} \ Y_{r2} \ Y_{a1L} \ Y_{a1R} \ Y_{a2L} \ Y_{a2R} \ Y_{a3L} \ Y_{a3R} \}$$
(5.1)

where Z_{ii} and Y_{ii} represent the vertical and lateral displacements of the *i*th rigid body, respectively; θ_{ii} and β_{ii} are the rotational displacements of the *i*th rigid body in the *x-z* plane (pitching) and the *y-z* plane (rolling), respectively; $Z_{ail(R)}$ represents the vertical displacement of the *i*th wheel axle on the left (right) side; $Y_{ail(R)}$ represents the lateral displacement of the *i*th wheel axle on the left (right) side. As shown in Fig. 5.1, the spring stiffness coefficient *K* and damping coefficient *C* for the springs and dampers of the vehicle models are labeled with subscripts according to the axle number, "*u*" (upper) or "*l*" (lower) position, "*y*" (lateral) or "*z*" (vertical) direction and "*L*" (left) or "*R*" (right) side of the vehicle. The dynamic models of the light trucks and cars are similar to those of the heavy trucks and consist of one main rigid body, two wheel axle sets, sixteen sets of springs and dampers vertically or laterally. The displacement vector d_{y} for the light truck or car has 12 DOFs including 5 vertical, 5 lateral, 1 rolling and 1 pitching independent DOFs, as expressed in Eq. (5.2).



$$d_{v} = \{Z_{r1} \ \theta_{r1} \ \beta_{r1} \ Z_{a1L} \ Z_{a1R} \ Z_{a2L} \ Z_{a2R} \ Y_{r1} \ Y_{a1L} \ Y_{a1R} \ Y_{a2L} \ Y_{a2R}\}$$
(5.2)

(a) Elevation view



(b) Side view

Figure 5.1 The numerical dynamic model for the heavy truck with one trailer

The point contact theory without separation between the road surface and the tires of each vehicle is assumed in the bridge-vehicle interaction model. The vertical displacement of the vehicle is related to the

vertical bridge deck displacement and the deck surface roughness at the contact point. The road surface roughness is simulated as a stationary Gaussian random process with zero mean value (Huang and Wang 1992; Chen and Wu 2010).

5.2.4 Modeling of wind forces on the bridge and vehicles

The aerodynamic forces acting on the bridge are commonly separated into three components: steadystate forces resulting from average wind speed, self-excited forces resulting from bridge-wind interactions and buffeting forces resulting from the unsteady wind velocity components.

5.2.4.1 Steady-state wind forces on bridge

The steady-state wind forces acting on the bridge deck consist of drag force, lift force and torsional moment, as defined in Eqs. (5.3a-c), respectively.

$$D = \frac{1}{2}\rho U_w^2 C_d(\alpha) H \tag{5.3a}$$

$$L = \frac{1}{2}\rho U_w^2 C_l(\alpha) H \tag{5.3b}$$

$$M = \frac{1}{2}\rho U_w^2 C_m(\alpha) B^2$$
(5.3c)

in which, U_w is the mean wind speed; ρ is the mass density of the air; H and B are the depth and width of the bridge girder, respectively; α is the effective wind attack angle, which is the summation of initial wind attack angle and the rotational displacement of the bridge deck at current iteration step; $C_d(\alpha)$, $C_l(\alpha)$ and $C_m(\alpha)$ are the static wind coefficients for the bridge girder in horizontal, vertical and torsional directions, respectively, which vary with the effective wind attack angle. It is noted that only the drag component of the steady-state wind force corresponding to initial wind attack angle is considered for bridge cables and pylons in this study.

5.2.4.2 <u>Self-excited wind forces on bridge</u>

Based on the linear aerodynamic theory that self-excited wind forces due to bridge-wind interactions are generated by a linear mechanism, each component of self-excited forces can be expressed as the summation of the response associated with the structural motion in lateral, vertical and torsional directions. In particular, the self-excited drag force, lift force and torsional moment on a unit span can be expressed by the convolution integral between the arbitrary bridge deck motion and the associated impulse functions, as shown in Eqs. (5.4a-c) (Lin and Yang 1983).

$$D_{se}(t) = D_{sep}(t) + D_{seh}(t) + D_{sea}(t) = \int_{-\infty}^{t} f_{Dp}(t-\tau)p(\tau)d\tau + \int_{-\infty}^{t} f_{Dh}(t-\tau)h(\tau)d\tau + \int_{-\infty}^{t} f_{Da}(t-\tau)\alpha(\tau)d\tau$$
(5.4a)

$$L_{se}(t) = L_{sep}(t) + L_{seh}(t) + L_{sea}(t) = \int_{-\infty}^{t} f_{Lp}(t-\tau)p(\tau)d\tau + \int_{-\infty}^{t} f_{Lh}(t-\tau)h(\tau)d\tau + \int_{-\infty}^{t} f_{L\alpha}(t-\tau)\alpha(\tau)d\tau$$
(5.4b)

$$M_{se}(t) = M_{sep}(t) + M_{seh}(t) + M_{sea}(t) = \int_{-\infty}^{t} f_{Mp}(t-\tau)p(\tau)d\tau + \int_{-\infty}^{t} f_{Mh}(t-\tau)h(\tau)d\tau + \int_{-\infty}^{t} f_{M\alpha}(t-\tau)\alpha(\tau)d\tau \quad (5.4c)$$

in which, p(t), h(t) and $\alpha(t)$ are the lateral, vertical and rotational displacements of the bridge deck, respectively; $f_{Dp}(t)$, $f_{Dh}(t)$, $f_{D\alpha}(t)$, $f_{Lp}(t)$, $f_{Lh}(t)$, $f_{L\alpha}(t)$, $f_{Mp}(t)$, $f_{Mh}(t)$ and $f_{M\alpha}(t)$ are the impulse response functions of the self-excited wind components; The first subscript "D", "L" and "M" refer to the response contribution to the self-excited drag force, lift force and torsional moment, respectively; The second subscripts "p", "h" and " α " specify the impulse response function with respect to unit impulse displacement in the lateral, vertical and torsional directions, respectively.

5.2.4.3 <u>Buffeting forces on bridge</u>

Buffeting forces on the bridge girder are induced by the fluctuating components of the oncoming wind velocity, which are usually represented by the time-variant horizontal and vertical components. Similar to the formulation of self-excited wind forces, the buffeting forces on a unit span can be expressed as the convolution integral associated with impulse response functions and the wind velocity fluctuations in both horizontal and vertical directions. The buffeting drag force, lift force and torsional moment can be expressed as:

$$D_{b}(t) = D_{bu}(t) + D_{bw}(t) = \int_{-\infty}^{t} f_{Du}(t-\tau)u(\tau)d\tau + \int_{-\infty}^{t} f_{Dw}(t-\tau)w(\tau)d\tau$$
(5.5a)

$$L_{b}(t) = L_{bu}(t) + L_{bw}(t) = \int_{-\infty}^{t} f_{Lu}(t-\tau)u(\tau)d\tau + \int_{-\infty}^{t} f_{Lw}(t-\tau)w(\tau)d\tau$$
(5.5b)

$$M_{b}(t) = M_{bu}(t) + M_{bw}(t) = \int_{-\infty}^{t} f_{Mu}(t-\tau)u(\tau)d\tau + \int_{-\infty}^{t} f_{Mw}(t-\tau)w(\tau)d\tau$$
(5.5c)

in which, u(t) and w(t) are the horizontal and vertical turbulent wind velocity components, respectively; $f_{Du}(t)$, $f_{Lu}(t)$, $f_{Lu}(t)$, $f_{Lw}(t)$, $f_{Mu}(t)$ and $f_{Mw}(t)$ are the impulse response functions related to the buffeting forces; The subscripts "u" and "w" indicate the impulse response function with respect to turbulent wind velocities in the horizontal and vertical directions, respectively.

5.2.4.4 Wind forces on vehicles

The aerodynamic wind forces acting on the vehicles are determined with quasi-static approach proposed by Baker (1986). The aerodynamic forces and moments on the vehicles have six components, which are drag force, side force, lift force, rolling moment, pitching moment and yawing moment, as expressed in Eqs. (5.6a-f), respectively.

$$F_{vx}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{D}(\Psi)A \qquad (5.6a) \qquad F_{vy}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{S}(\Psi)A \qquad (5.6b) \qquad F_{vz}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{L}(\Psi)A \qquad (5.6c)$$

$$M_{vx}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{R}(\Psi)Ah_{v} \quad (5.6d) \qquad M_{vy}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{P}(\Psi)Ah_{v} \quad (5.6e) \qquad M_{vz}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{Y}(\Psi)Ah_{v} \quad (5.6f)$$

in which, $U_R(t) = \sqrt{(U_w + u(x,t))^2 + U_{ve}^2(t)}$; u(x,t) is the unsteady wind speed at longitudinal location x at time instant t; $U_{ve}(t)$ is the vehicle driving speed at time instant t; Ψ is the yaw angle, $\psi = \arctan[(U_w + u(x,t))/U_{ve}(t)]$; $C_D(\Psi)$, $C_S(\Psi)$, $C_L(\Psi)$, $C_R(\Psi)$, $C_P(\Psi)$ and $C_Y(\Psi)$ are the drag force coefficient, side force coefficient, lift force coefficient, rolling moment coefficient, pitching moment coefficient and yawing moment coefficient, respectively; $_A$ is the reference area, which is usually taken as the frontal area of the vehicle; h is the reference height, which is usually taken as the distance between the gravity of the vehicle and the ground.

5.2.5 Dynamic motion equations for the fully-coupled bridge-traffic-wind system

The bridge and vehicles in the traffic flow are established as two subsystems in the fully-coupled bridge-traffic-wind interaction model. The bridge subsystem is constructed corresponding to the physical degrees of freedom (DOFs) of the bridge finite element model. The vehicle subsystem is established based on the DOFs of each vehicle dynamic model. When vehicles are traveling on the bridge, the interaction forces between the bridge and the vehicles are formulated in terms of coupling matrices and external forces, which will be incorporated in the motion equations of the whole bridge-traffic-wind system. The two approaching roadways for both driving directions are assumed to be completely rigid such that a vehicle is excited by road roughness only without considering the flexibility of the pavement. The coupled motion equations of the bridge-traffic-wind system can be built as follows:

$$\begin{bmatrix} M_{b} & 0 & \cdots & 0 \\ 0 & M_{v_{1}} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & M_{v_{s}} \end{bmatrix} \begin{bmatrix} \ddot{q}_{b} \\ \ddot{q}_{v_{1}} \\ \vdots \\ \ddot{q}_{v_{s}} \end{bmatrix} + \begin{bmatrix} C_{b} & 0 & \cdots & 0 \\ 0 & C_{v_{1}} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & C_{v_{s}} \end{bmatrix} \begin{bmatrix} \dot{q}_{b} \\ \dot{q}_{v_{1}} \\ \vdots \\ \dot{q}_{v_{s}} \end{bmatrix} + \begin{bmatrix} K_{b} & 0 & \cdots & 0 \\ 0 & K_{v_{1}} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & K_{v_{s}} \end{bmatrix} \begin{bmatrix} q_{b} \\ q_{v_{1}} \\ \vdots \\ \dot{q}_{v_{s}} \end{bmatrix}$$

$$= \begin{cases} F_{b}^{\ G} + \sum_{i=1}^{n} F_{v_{i}}^{\ G} + F_{b}^{\ R} + F_{b}^{\ C} + F_{b}^{\ SW} + F_{b}^{\ Se} + F_{b}^{\ Bu} \\ \vdots \\ F_{v_{1}}^{\ R} + F_{v_{1}}^{\ C} + F_{v_{1}}^{\ W} \\ \vdots \\ F_{v_{n}}^{\ R} + F_{v_{n}}^{\ C} + F_{v_{n}}^{\ W} \end{cases}$$

$$(5.7a)$$

in which,

$$F_{b}^{C} = \sum_{i=1}^{n} C_{bci}\{\dot{q}_{b}\} + \sum_{i=1}^{n} K_{bci}\{q_{b}\} + \sum_{i=1}^{n} C_{b,v_{i}}\{\dot{q}_{v_{i}}\} + \sum_{i=1}^{n} K_{b,v_{i}}\{q_{v_{i}}\}$$
(5.7b)

$$F_{v_i}^{C} = C_{v_i,b} \{ \dot{q}_b \} + K_{v_i,b} \{ q_b \}$$
(5.7c)

where *n* is the total number of vehicles involved in the interaction analysis; q_b and q_{v_i} are the displacement vectors of the bridge and the *i*th vehicle, respectively; the subscripts *b* and v_i (*i* = 1, 2, ... *n*)

indicate the parameters are for the bridge and the *i*th vehicle, respectively; one-dot and two-dot superscripts of the displacement vector denote the corresponding velocity and acceleration, respectively; $M_{b(v_i)}$, $K_{b(v_i)}$ and $C_{b(v_i)}$ are the structural mass, stiffness and damping matrices for the bridge subsystem (*i*th vehicle); K_{bci} and C_{bci} are the stiffness and damping contributions to the bridge structural matrices due to the coupling effects between the *i*th vehicle in the traffic flow and the bridge; K_{b,v_i} and C_{b,v_i} are the coupled stiffness and damping matrices contributing to the bridge vibration from the *i*th vehicle in the traffic flow; $K_{v,b}$ and $C_{v,b}$ are the coupled stiffness and damping matrices contributing to the vibration of the *i*th vehicle in the traffic flow from the bridge structure; F_b and F_v denote the nodal force vectors due to externally applied loads for the bridge and the *i*th vehicle, respectively; the superscripts *G*, *R*, *C*, *SW*, *Se*, *Bu* and *W* define the external loads caused by gravity, road surface roughness, coupling bridge-vehicle interaction forces, static wind, self-excited, buffeting forces and total wind forces, respectively.

5.3 Nonlinear Effects Considered during Cable Breakage Incidents

When a cable-stayed bridge suffers from the loss of stay cables, the incorporation of geometric and material nonlinearity often becomes essential considering the following aspects: (1) Cable elements may experience large change of the nonlinear axial stiffness due to the cable sag effect under its own self-weight, and therefore the geometric nonlinearity of cable elements cannot be ignored; (2) the interaction between the axial and flexural deformations in bending members may result in significantly different structural matrix formulations, and therefore the geometric nonlinearity of beam elements should be considered; (3) Large stress and strain may be triggered during the breakage process and the material may behave nonlinearly in the plastic range. The bridge performance can be very complex with possible large deformations during the cable breakage process. So as a general simulation tool for cable-loss incidents, geometric and material nonlinearity are considered for the cable and beam elements in the present model.

5.3.1 Nonlinearity involved in the analysis

5.3.1.1 Geometric nonlinearity

Due to the sag phenomenon under the cable self-weight, the axial stiffness of cables varies nonlinearly with the cable stress, which is also referred to as the tension-stiffening effects. By formulating the equilibrium equations for a catenary cable, various sources of geometric nonlinearity of cable elements are incorporated in the simulation process, including cable sag effect, large displacement effect and tension-stiffening effect. The geometric stiffness of the beam elements on the bridge girders and pylons are considered in the tangent stiffness which can also evaluate the stability performance of the cable-stayed bridge during the nonlinear dynamic analysis.

5.3.1.2 <u>Material nonlinearity for beam elements</u>

The concentrated plastic hinge approach is applied to consider the material nonlinearity of the beam elements, which may be incurred during the cable breakage simulation process. The yielding state of the beam element is determined according to the prescribed interacting yielding surface of axial force and bi-axial bending moments at the hinge location. The plastic hinge for the beam element is defined as *PMM* hinge, named for the three components forming the interacting yielding surface, which include axial force *P* and two bending moments *MM* around the two orthogonal axes of the cross section. The plastic tangent stiffness matrix of the beam element K_{pl} is expressed in Eq. (5.8).

$$K_{pl} = K_e - K_e N (N^{t} (K_p + K_e) N)^{-1} N^{t} K_e$$
(5.8)

in which, K_e is the elastic tangent stiffness matrix; K_p is the plastic stiffness matrix, which depends on the hardening slope of the element stiffness; N is the gradient matrix of the yield surface function. The interaction yielding surface Ψ is conservatively selected as an octahedron surface, which can be expressed in the following equation.

$$\Psi(P, M_{y}, M_{z}) = \frac{|P|}{P_{yield}} + \frac{|M_{y}|}{M_{yyield}} + \frac{|M_{z}|}{M_{zyield}} = 1$$
(5.9)

in which, P, M_y and M_z are the axial force, bending moment around y-axis and z-axis of the member, respectively; P_{yield} , M_{yyield} and M_{zyield} are the axial force, bending moment around y-axis and z-axis of the member at yield state, respectively; $P_{yield} = f_y \cdot A$; $M_{yyield} = f_y \cdot S_{py}$; $M_{zyield} = f_y \cdot S_{pz}$; f_y is the yield strength; A is the sectional area; S_{py} and S_{pz} are the plastic sectional modulus around y-axis and z-axis of the member cross section, respectively.

5.3.1.3 Material nonlinearity for cable elements

The inelastic behavior of a cable element is modeled using the concentrated plastic hinge approach through which plastic hinges are lumped at the two ends of the cable element. Different from the interacting P-M2-M3 hinge for beam elements, the plastic hinge model for cable elements can only bear pure axial force, known as P hinge. The cable element is considered to enter the yield state if the axial force of cable is greater than the yield force of cable. After plastic hinge forms at one end or both ends, the yield force is applied and the elastic modulus of the cable element will be replaced with zero at the yielding end of the cable. The yielding state Ω for cable elements is expressed in the following equation.

$$\Omega(P) = \frac{|P|}{P_{yield}} = 1$$
(5.10)

5.3.2 Nonlinear static analysis of bridge

Nonlinear static analysis is firstly conducted on the bridge structure under the bridge self-weight and static wind loads in order to provide the initial displacement vector of the bridge subsystem for the following nonlinear dynamic analysis. The tangent stiffness of the bridge structure is formulated and updated based on newly calculated position of each element. The equilibrium equation can be expressed as:

$$([K_{E}(q_{j-1})]+[K_{\sigma}(q_{j-1})]) \{q\}_{j} = \{F\}_{j}$$
(5.11)

in which, $[K_E(q_{j-1})]$ and $[K_{\sigma}(q_{j-1})]$ are the structural elastic stiffness and geometric stiffness matrices formulated using the response vector from the previous step, respectively; the tangent stiffness matrix [K] is the summation of structural elastic stiffness matrix $[K_E]$ and geometric stiffness matrix $[K_\sigma]$. $\{q\}_j$ and $\{F\}_j$ are the response vector and the external applied load vector in the iteration step *j*, respectively. Newton-Raphson iteration technique is used to accelerate the convergence of the iteration process.

5.3.3 Dynamic simulation strategy of cable breakage scenario with dynamic initial states

In most of existing studies about cable breakage events, cable breakage scenarios were often simulated by applying counteracting forces at the two ends of the cable for certain periods of the time to mimic the scenarios before or after the cable loss in an equivalent sense (Ruiz-Teran and Aparicio 2009; Mozos and Aparicio 2010a, b; Mozos and Aparicio 2011; Cai et al. 2012; Zhou and Chen 2014). In those studies, the failed cable either was physically removed from the model even before the cable rupture occurs or still remained in the model after the cable loss event. As studied and discussed by Zhou and Chen (2014), either way of handling the physical cable as summarized above causes some error on prebreakage or post-breakage performance. To overcome these limitations, the cable breakage process in the proposed time-progressive analysis is simulated by both applying the counteracting forces and also physically reducing the area of the failing cable following certain process function within total time duration t_{total} of cable breakage. The total counteracting forces are adopted as the cable internal forces right before the instant of cable breakage with the same amplitude but in the opposite directions. The counteracting forces, applied only during the breakage time period, represent the dynamic impact loads involved in the cable-loss events. The concept of cable area reduction is applied to more realistically model post-breakage elemental condition of the bridge model and also offer flexibility of characterizing different cable-loss events. It is known that cable loss may be caused by different reasons, such as fire, lighting, collision, blast, or deterioration, etc. As a general approach to simulate cable-loss incidents caused by different possible extreme events, the cable breakage process function is introduced:

The relative breakage time of cable t_{rel} is defined as the ratio of the elapsed time t_{elap} after breakage starts to the total breakage time t_{total} . The relative loss of cable l_{rel} is defined as the ratio of lost cable area

to total cable area. The breakage process function $f(t_{rel})$ is expressed as the function of t_{rel} and represent the relation of l_{rel} and t_{rel} during the cable breakage incident, as shown in Eq. (5.12).

$$l_{rel} = f(t_{rel})$$
(5.12)

As the cable area decreases gradually during the breakage process following the cable breakage process function, the tension force originally provided by the lost cable area will also be eliminated during a very short time period. The dynamic impact caused by the sudden removal of the cable forces is modeled by adding the counteracting forces F_{b1t} and F_{b2t} at the cable ends, which are quantified at each time step following the cable breakage process function. As a result, both the structural elemental configuration and force condition during a general cable loss incident can be characterized more realistically than existing studies. The counteracting forces F_{b1t} and F_{b2t} will not be applied on the cable joints before the breakage starts or after the breakage completes, which is different from existing studies.

Such a cable-loss simulation procedure is described as follows:

Step 1: Conduct the nonlinear static analysis of the bridge system under the bridge self-weight and static wind load to obtain the initial deformed position of the bridge. The initial displacements for vehicles are set to be zero.

Step 2: Conduct nonlinear dynamic analysis of the bridge-traffic system under dynamic loads from stochastic traffic and wind for t_1 seconds before cable breakage occurs. Obtain the nodal axial forces of the cable to fail at t_1 second, which are denoted as F_{b1} and F_{b2} for the upper and lower ends of the cable, respectively. The displacement, velocity and acceleration vectors at t_1 second are obtained as the initial dynamic breakage states.

Step 3: Determine the elapsed breakage time t_{elap} and relative breakage time t_{rel} at current time step after breakage occurs. Obtain the effective area of the cable for failure and the relative loss of cable l_{rel} at current time step using Eq. (5.12). Update the current effective area of cable for failure in the global structural formulation. The counteracting nodal forces F_{b1t} and F_{b2t} at current time step are determined by multiplying l_{rel} with F_{b1} and F_{b2} , respectively, as shown in Eq. (5.13).

$$F_{bl(2)t} = l_{rel} \cdot F_{bl(2)}$$
(5.13)

Apply the nodal forces F_{b1t} and F_{b2t} with the same amplitude but in the opposite directions to the two anchorage nodes of the cable for failure at time instant t_1+t_{elap} . Starting with the initial breakage state, the displacement, velocity and acceleration vectors at time instant t_1+t_{elap} can be obtained.

Step 4: Repeat step 3 in each time step during the cable breakage duration t_{total} and obtain the global displacement, velocity and acceleration vectors after the breakage process completes at time instant t_1 + t_{total} .

Step 5: Conduct the nonlinear dynamic analysis of the bridge-traffic system staring with the displacement, velocity and acceleration vectors at time instant $t_1 + t_{total}$ for the time duration of t_2 seconds.

Due to the loss of certain cable(s), the simulation analysis will iterate and find a new equilibrium position of the bridge system after the breakage starts in Steps 3-5. Each bridge member will be evaluated at each iteration step about whether it may get into the inelastic range or fail. If the member is determined to behave elastically, the mass matrix, tangent stiffness matrix, damping matrix and external force vector will be constructed based on the motion vector of the current iteration step. If a certain member yields without failing, the element tangent stiffness matrix will be reduced to the plastic stiffness matrix and the fixed end forces will also be updated. If a member is considered to fail following the prescribed failure criterion, such a member will be eliminated from the bridge FE model.

As a summary of the whole procedure as introduced above, the proposed nonlinear dynamic simulation platform for the cable breakage scenario can be illustrated in the following flow chart.



Figure 5.2 Flowchart demonstration of the simulation platform for cable breakage event

5.4 Numerical Demonstration

A prototype long-span cable-stayed bridge, along with passing traffic, is selected to demonstrate the proposed simulation strategy. Subjected to a representative single-cable breakage scenario, the bridge response and the impact on the bridge-traffic and bridge-wind interactions are investigated.

5.4.1 Description of the prototype bridge

The prototype cable-stayed bridge in the present study has a total length of 840 m, with a main span, two side spans and two approach spans as shown in Fig. 5.3a. The steel twin-box girder cross-section has a width of 28 m and a height of 4.6 m, which is shown in Fig. 5.3b. The two A-shaped steel pylons have a height of 103.6 m and single-box cross-sections. The two cable planes of the bridge are placed in a fan-shaped arrangement with 12 sparsely distributed cables in each cable plane with intervals of 54 m and 52 m for the main and two side spans, respectively. The six cables of the cable plane in the north and south sides are labeled as Cable 1a-6a and Cable 1b-6b, respectively. Wind is assumed to blow from the north to the south side of the bridge.



(a) 3-D sketch of the cable-stayed bridge



(b) Cross section of the bridge girder

Figure 5.3 Prototype long-span cable-stayed bridge

The mechanical properties of the bridge girders, cross beams, pylons and strands used for the stay cables are shown in Table 5.1. The sectional properties of the bridge girders, cross beams, pylons and stay cables are listed in Table 5.2.

Table 5.1 Mechanical properties of the bridge system

Properties	Unit	Girders, cross beams and pylons	Stay cables
Weight density, γ	kN/m^3	76.97	74.86
Elasticity modulus, E	kN/m^2	2.0E+8	2.0E+8
Poisson's ratio, ρ	-	0.30	0.27
Shear modulus, G	kN/m^2	7.69E+7	7.87E+7
Yield stress, F_y	kN/m^2	2.48E+5	1.49E+6
Ultimate stress, F_u	kN/m^2	3.45E+5	1.72E+6

Properties	Unit	Girders	Cross beams	Pylons	Stay cables
Cross sectional area, A	m^2	0.556	0.457~1.045	0.212~0.549	0.0130~0.0343
Moment of inertia, torsional, J	m^4	1.317	1.238~2.156	0.681~2.792	-
Moment of inertia, vertical, I_y	m^4	1.548	1.087~2.122	0.687~3.532	-
Plastic modulus, vertical, S_y	m^3	0.717	0.523~1.021	0.352~1.368	-
Moment of inertia, lateral, I_z	m^4	20.97	1.087~2.122	0.687~3.532	-
Plastic modulus, lateral, S_z	m^3	4.227	0.523~1.021	0.315~1.126	-

Table 5.2 Sectional properties of the bridge system

5.4.2 Description of road vehicles

The stochastic traffic flow is simulated on the roadway-bridge-roadway system with a total length of 1260 m, including two roadways with a length of 210 m each and the bridge with a length of 840 m. The busy stochastic traffic flow is simulated with a vehicle density of 31 veh/km/lane. The vehicles simulated in the stochastic traffic flow are classified into three types: heavy truck, light truck and light car, which account for 20%, 30% and 50% of the total vehicles, respectively. The dynamic parameters for each type of vehicles involved in the present study, including mass, mass moment of inertia, stiffness coefficients and damping coefficients, are listed in Table 5.3. The dimension parameters for each type of vehicles are listed in Table 5.4.

Table 5.3 Dynamic parameters of the vehicles used in the case study

Dumomio noromotor	Linit	Vehicle type			
Dynamic parameter	Unit	Heavy truck	Light truck	Light car	
Mass of the 1 st rigid body	kg	4000	6500	1600	
Pitching moment of inertia of the 1 st rigid body	$kg \cdot m^2$	10500	9550	1850	
Rolling moment of inertia of the 1 st rigid body	$kg \cdot m^2$	3200	3030	506	
Mass of the 2 nd rigid body	kg	12500	-	-	
Pitching moment of inertia of the 2 nd rigid body	$kg \cdot m^2$	28500	-	-	
Rolling moment of inertia of the 2 nd rigid body	$kg \cdot m^2$	10500	-	-	
Mass of the 1 st axle block	kg	370	800	39.5	

Dynamic parameter		Init	Vehicle type			
		Unit	Heavy truck	Light truck	Light car	
Mas	s of the 2 nd axle block	kg	1250	800	39.5	
Mas	s of the 3 rd axle block	kg	1100	-	-	
Stiffness	The 1 st axle $(K_{uzL}^{1}, K_{uzR}^{1})$	kN/m	100	250	109	
coefficient in the vertical upper	The 2 nd axle (K_{uzL}^2 , K_{uzR}^2)	kN/m	250	250	109	
springs	The 3 rd axle (K_{uzL}^3 , K_{uzR}^3)	kN/m	400	-	-	
Stiffness	The 1 st axle $(K_{uzL}^{1}, K_{uzR}^{1})$	kN/m	150	175	176	
coefficient in the vertical lower	The 2 nd axle (K_{LL}^2, K_{LR}^2)	kN/m	400	175	176	
springs	The 3^{rd} axle (K_{LL}^{3} , K_{LR}^{3})	kN/m	500	-	-	
Stiffness coefficient in the lateral upper springs	The 1 st axle (K_{uyL}^1, K_{uyR}^1)	kN/m	75	187.5	79.5	
	The 2 nd axle (K_{uyl}^2, K_{uyR}^2)	kN/m	187.5	187.5	79.5	
	The 3^{rd} axle $(K_{uyl}^{3}, K_{uyl}^{3})$	kN/m	300	-	-	
Stiffness coefficient in the lateral lower springs	The 1 st axle $(K_{i_{j,l}}^{1}, K_{i_{j,R}}^{1})$	kN/m	72	100	58.7	
	The 2 nd axle $(K_{i_{pL}}^2, K_{i_{pR}}^2)$	kN/m	131	100	58.7	
	The 3^{rd} axle $(K^3_{b,L}, K^3_{b,R})$	kN/m	167	-	-	
Damping	The 1 st axle $(C_{uzL}^{I}, C_{uzR}^{I}, C_{uyL}^{I}, C_{uyR}^{I})$	kN∙s/m	5	2.5	0.8	
coefficient in the vertical/lateral upper dampers	The 2 nd axle (C_{uzL}^2 , C_{uzR}^2 , C_{uyL}^2 , C_{uyR}^2)	kN∙s/m	30	2.5	0.8	
	The 3 rd axle (C_{uzl}^{3} , C_{uzR}^{3} , C_{uyl}^{3} , C_{uyR}^{3})	kN∙s/m	40	-	-	
Damping coefficient in the vertical/lateral lower dampers	The 1 st axle $(C_{LL}^{1}, C_{LR}^{1}, C_{bl}^{1}, C_{lyR}^{1})$	kN∙s/m	1.2	1	0.8	
	The 2 nd axle (C_{LL}^2 , C_{LR}^2 , C_{lyL}^2 , C_{lyR}^2)	kN·s/m	4.5	1	0.8	
	The 3^{rd} axle $(C_{LL}^3, C_{LR}^3, C_{bL}^3, C_{bL}^3)$	kN∙s/m	4.5	-	-	

Table 5.4 Dimensions of the vehicles used in the case study

Parameter	Unit	Heavy truck	Light truck	Light car
Distance between axle 1 and rigid body 1 (D_1)	m	1.83	2.8	1.34
Distance between axle 2 and rigid body 1 (D_2)	т	1.83	4.8	1.34

Parameter		Heavy truck	Light truck	Light car
Distance between axle 2 and rigid body $2(D_3)$		3.60	-	-
Distance between axle 3 and rigid body 2 (D_4)	т	2.60	-	-
Distance between pin and rigid body 1 (D_5)	т	1.83	-	-
Distance between pin and rigid body 2 (D_6)		3.60	-	-
Distance between left axle and rigid body 1 (W_1)		1.10	1.0	0.8
Distance between right axle and rigid body 1 (W_2)		1.10	-	-
Frontal area (A)	m^2	10.5	7.5	1.96
Reference height (h_v)	т	2.0	1.85	1.1
Total length of vehicle (L)		13.4	11.6	4.54

5.4.3 Validation of the basic modules of the framework

To validate the basic modules of the proposed finite element framework, the same prototype bridge is also modeled with the commercial finite element program SAP2000. As partial validation efforts, nonlinear static analysis, modal analysis and dynamic analysis have been conducted using both the proposed program and SAP2000 program and the representative results are listed below.

5.4.3.1 Validation analysis using nonlinear static analysis

The nonlinear static analysis is conducted on the prototype bridge under self-weight from the proposed finite element framework and SAP2000 program. The vertical displacement at the mid-span of the bridge girder and the longitudinal displacement at the west pylon top are obtained and compared using the two programs, and the results are listed in Table 5.5. The difference of the primary displacement response using the two programs is within 1 %, indicating the acceptable accuracy of the proposed finite element program in terms of static analysis.

Table 5.5 Comparison of primary displacement response of the bridge from nonlinear static analysis

Displacement response	Proposed finite element framework	SAP2000 program	Difference (%)
Vertical displacement at mid-span girder joint (m)	-1.408	-1.415	0.5
Longitudinal displacement at west pylon top (m)	0.293	0.295	0.6

5.4.3.2 Validation analysis using modal analysis (eigenvalue analysis)

The frequency properties of the bridge are obtained from modal analyses using the two programs. The lowest several representative modes of the bridge in terms of mode frequencies and mode shapes are listed side-by-side in Table 5.6. More detailed comparison results regarding the modal frequencies and mode shapes can be found in the reference (Zhou and Chen 2013).

Mada	Proposed finite element framework		SAP20	Difference	
Frequency (Hz)		Shape	Frequency (Hz)	Shape	(%)
1	0.4321	1 st sym. Vertical	0.4232	1 st sym. Vertical	2.00
2	0.6522	1 st sym. Lateral	0.6095	1 st sym. Lateral	6.55
3	0.6829	1 st anti-sym. Vertical	0.6744	1 st anti-sym. Vertical	1.24
4	0.9884	2 nd sym. Vertical	0.9881	2 nd sym. Vertical	0.03
5	1.2424	1 st sym. Torsional	1.2052	1 st sym. Torsional	2.99

Table 5.6 Comparison of mode properties of the bridge from modal analysis

5.4.3.3 Validation analysis using time history analysis (dynamic analysis)

The coupled bridge-traffic interaction analysis is firstly conducted to obtain the equivalent traffic force time history on each bridge girder joint due to bridge-traffic interactions. The moderate traffic flow is involved in the analysis with a density of 22 vehicles per kilometer per lane. The total simulation time is 50 seconds. The equivalent time-history nodal forces acting on the bridge due to the presence of moving vehicles are obtained and then applied to the respective bridge models in the proposed finite element program and SAP2000 program. Dynamic analyses are conducted on both bridge models under the same suite of equivalent nodal forces using the Newmark- β integration method for both programs. The time histories of vertical displacement response at the mid-span joint of the bridge girder obtained from the proposed finite element program and SAP2000 program and SAP2000 program are compared in Fig. 5.4. It is found that the instant dynamic response is very close using the two programs, indicating that the developed finite element program can provide valid dynamic response from dynamic analysis.



Figure 5.4 Comparison of time history response at the mid-span of the bridge girder from dynamic

analysis

As discussed earlier, to date, none of the existing approaches can conduct similar cable breakage simulation as proposed, in which bridge-traffic and bridge-wind interactions are incorporated. Therefore, a direct and full validation of the proposed nonlinear dynamic simulation framework of cable breakage events with another simulation tool is not yet possible at the moment. It is expected that a full validation of the proposed framework can be made in the future when either more experimental study results or other advanced simulation tools become available.

5.4.4 Simulation of dynamic excitations

5.4.4.1 Road surface roughness

"Good" roughness condition is assumed in the present study with the road roughness coefficient being 20×10^{-6} m³/cycle according to ISO specifications (Dodds 1972). The road surface roughness profiles are simulated for two approaching roadways and the bridge (Chen and Wu 2010).

5.4.4.2 <u>Turbulent wind speeds</u>

The correlated turbulent wind velocities are simulated as multi-variate stationary random processes (Deodatis 1996a) at eight locations uniformly distributed along the bridge. The sampling frequency used in the simulation of turbulent wind speed is 25 Hz. The power spectral density components of vertical and

horizontal turbulent wind vectors used herein are given by Kaimal spectrum and Panofsky-McCormick spectrum, respectively.

5.4.4.3 <u>Cable breakage simulation</u>

As discussed earlier, the cable loss process caused by a specific extreme event can be very different from another one caused by different reasons. The cable breakage process function f and total breakage time t_{total} are proposed in the present study to offer a flexible tool to simulate cable loss incidents caused by various reasons. Although it is not yet available, the data to describe the cable loss process due to different causes can be obtained by conducting some experiments. It is, however, beyond the scope of the present study to characterize the specific cable-loss processes.

Due to the absence of existing data about the cable breakage process function and total breakage time, some assumptions need to be made in order to carry out the following demonstration. The cable-loss process is typically believed to last for a very short time period, while there is no consensus on typical time duration for various cable-loss events (Zhou and Chen 2014). Without aiming to characterize cable loss process caused by a specific extreme event, the total breakage duration t_{total} of 0.04 s is assumed in the following demonstration after referencing some existing studies (e.g., Zhou and Chen 2014a). The breakage of the cable is assumed to follow a linear loss process, in which the breakage process function is expressed in the following equation.

$$f(t_{rel}) = t_{rel} \tag{5.14}$$

In the numerical demonstration, only single-cable breakage is studied and specifically, cable 1a is assumed to fail at around 20 s. The total dynamic simulation time period is selected to be 50 seconds with an integration time step of 0.04 s, which is the same value as the total breakage time. Traffic and/or wind load condition are assumed to be unaffected by the cable breakage event in this study.

5.4.5 Simulation results in a representative study for demonstration

In the present demonstrative study, cable 1a is assumed to break at 20.00 second when busy stochastic traffic and mild wind (10 m/s) are present. The nonlinear dynamic analysis of the first 20.00 s

before cable breakage occurs provides the performance information of the bridge-traffic system in normal service conditions. After the breakage occurs, cable 1a is removed at 20.04 s considering the breakage completes within one time step. In the meantime, the cable forces at two ends of cable 1a at 20.00 s are applied to the corresponding anchorage nodes of cable 1a at 20.04 s with same amplitude but in the opposite direction for the time duration of one time step (0.04 s). Nonlinear dynamic analysis continues for another 29.96 s to demonstrate the post-breakage dynamic response of bridge-traffic system.

It is found that plastic hinges will not form and the materials of the bridge behave linearly during the whole simulation process when only a single cable breaks under the service load excitations from wind and traffic. The dynamic responses of the bridge, vehicles and the interaction forces from wind and traffic will be demonstrated in the following sections.

5.4.5.1 Dynamic displacement response of the bridge

The dynamic response of the bridge structure can be obtained through nonlinear dynamic analysis in time history. The time histories of the vertical, lateral and torsional displacements of the leeward girder joint (on south side of the bridge in Fig. 5.3a) connecting cable 1a are shown in Figs. 5.5a, b and c, respectively. It is observed that the bridge local dynamic responses close to the ruptured cable experience abrupt amplifications in vertical, lateral and torsional directions after cable breakage occurs. There is a sudden change of the mean displacement in each direction after the breakage occurs, showing that a new balance position of the bridge is reached after the cable loss. The dynamic responses reach the largest extreme value soon after the instant when the cable rupture occurs (i.e. 20.00 s in this example) and then decay gradually.





Figure 5.5 Dynamic displacement histories at joint of leeward girder connecting the breaking cable 5.4.5.2 *Impact of cable-loss on self-excited wind forces*

To further look into the mechanism of cable-breakage events, the impacts from cable-breakage events on the bridge-traffic-wind interaction system are investigated. Self-excited wind forces are motiondependent and can reflect the interaction between bridge and wind. The time histories of the self-excited wind forces under wind speed 10 m/s throughout the cable breakage event, including drag, lift force and torsional moment, at the joint of leeward girder connecting the breaking cable are shown in Figs. 5.6a-c. For comparison purposes, the results of the self-excited forces under the normal service conditions without the occurrence of cable breakage are also plotted. It is found that self-excited wind forces all exhibit abrupt increase after the cable breakage occurs and then decay gradually, indicating considerable influence on the bridge-wind interaction. Since self-excited wind forces are usually related to aerodynamic instability, the results suggest that cable breakage event occurring under higher wind speeds may potentially influence the instantaneous aerodynamic stability of a slender long-span bridge. Specific studies about the impact on aerodynamic stability from cable loss events may be conducted in the future.



(a) Drag force



(b) Lift force





Figure 5.6 Self-excited forces at the girder joint connecting the breaking cable throughout the analysis 5.4.5.3 *Impact of cable-loss on bridge-traffic interaction forces*

The forces on the bridge related to vehicles can be divided into three components, which are vehicle gravity force, force due to road surface roughness and force due to coupling effects from bridge-traffic interaction, as shown in Eq. (5.7). The cable-breakage events have no effect on the forces due to vehicle gravity and road surface roughness while may significantly influence the motion-dependent coupling forces between the bridge and moving traffic. The coupling bridge-traffic interaction forces in the vertical, lateral and torsional directions at the girder joint connecting to the breaking cable are shown in Fig. 5.7a, b and c, respectively. Due to the movement of stochastic traffic, it is found that the interaction forces fluctuate remarkably around the zero mean value with respect to time. It is indicated that the coupling bridge-traffic interaction forces are significantly influenced by the cable-breakage events, especially in the vertical and torsional directions.



(a) Vertical interaction forces



(c) Torsional interaction moments

Figure 5.7 Forces and moments from bridge-traffic interaction at the mid-span of leeward girder

5.4.5.4 Dynamic response of the vehicles

Based on the fully-coupled bridge-traffic-wind interaction analysis, the dynamic response of each vehicle in the traffic flow can be readily obtained from the nonlinear dynamic analysis of cable breakage scenarios. The representative vehicle is selected as a light truck traveling on the inner lane in the positive driving direction. The traveling path of the representative vehicle is displayed in Fig. 5.8, which shows that the light truck travels on the roadway-bridge-roadway system with different instantaneous driving speeds following the stochastic traffic flow simulation, evidenced by varying curve slopes over time. When the cable breakage occurs, the vehicle is at the longitudinal location of 581.6 m from the west end of bridge, 20 meters away from the cable-breakage location.



Figure 5.8 Vehicle traveling path in the longitudinal direction

The time histories of the vertical and lateral displacement and acceleration responses for the representative vehicle are demonstrated in Figs. 5.9 and 5.10, respectively. It is noted that the vehicle displacement responses are measured from the equilibrium position of the bridge under gravity and static wind loads in each time step. After the breakage of Cable 1a occurs at 20.00 s, the vehicle exhibits significantly larger dynamic displacement and acceleration in the vertical direction than those without cable breakage. In contrast to the vertical displacement response, the lateral displacement of the vehicle doesn't exhibit considerable difference upon the cable breakage. By comparing the responses of different vehicles, it is found that the vehicle response is more likely to be influenced by the cable-breakage incident when the vehicle is close to the failed cable at the moment of cable breakage. Although beyond the current scope, it is apparent that the proposed platform can provide detailed time-history response of any individual vehicle, which is critical to studying vehicle driving safety and driving comfort issues on bridges under both normal service load condition and extreme cable-breakage events.



(a) Displacement response



(b) Acceleration response

Figure 5.9 Vertical response histories of the representative vehicle





Figure 5.10 Lateral response histories of the representative vehicle

5.4.6 Influence of the time instant when cable breakage occurs-dynamic initial states of cable breakage

Long-span cable-stayed bridges vibrate considerably under service wind and traffic loads. Therefore, depending on the exact instant when cable breakage occurs, the initial state of cable breakage is not only dynamic with non-zero velocity and acceleration, but also exhibits stochastic characteristics. In the previous study by the authors (Zhou and Chen 2014), the dynamic initial states of the cable breakage events caused by stochastic traffic only were studied. In the present study, more comprehensive studies are conducted to investigate the influence from the specific time instant as well as the associated dynamic initial states of cable breakage subjected to both traffic and wind service loads. The vertical and torsional displacements at the mid-span joint of the south bridge girder are investigated assuming cable 1a fails abruptly at around 20 second when busy traffic and wind excitations of 10 m/s are applied on the bridge.

Two dynamic extreme initial states for vertical displacement are selected at 18.92 second and 20.04 second, when the mid-span joint of the bridge girder moves to the uppermost and lowest positions, respectively. For comparison purposes, another time instant, 19.48 second, is chosen between the two selected time instants. Thus there are totally three representative dynamic initial states for vertical response, i.e. breakage of cable 2a starting at 20.04 second (dynamic initial state 1), 19.48 second (dynamic initial state 2) and 18.92 second (dynamic initial state 3). Nonlinear dynamic analyses are conducted for the three representative dynamic initial states with the vertical and torsional displacements at the mid-span joint of bridge girder as presented in Figs. 5.11a and b, respectively. It is found in Fig. 5.11a that vertical displacement response reaches the largest and the smallest extreme values with initial states 1 and 3, respectively. Similar to the observations made in the previous study (Zhou and Chen 2014), vertical displacement of the bridge girder joint has local maximum and minimum values when the breakage occurs at the time instant when the joint reaches the uppermost position and starts to move downward, and the lowermost position and starts to move upward, respectively. As shown in Fig. 5.11b, the maximum and minimum extreme initial states (i.e. initial state 1 and 3) identified based on vertical response do not cause the maximum and minimum extreme values of torsional displacement response. A comprehensive search among different breakage instants is made to identify the critical initial states for

torsional displacement. It is found that at the breakage instants of 19.88 s and 19.52 s, the bridge torsional displacement reaches the local largest and smallest values with the presence of service loads, respectively.



(a) Vertical displacement



(b) Torsional displacement

Figure 5.11 Dynamic displacement at mid-span of bridge girder with breakage initial states 1-3

Although critical initial states causing the maximum and minimum extreme responses in different directions (e.g., vertical and torsion) are not the same, some general rules can be identified from the previous results. Cable breakage will cause an instantaneous unbalance of the bridge system, accordingly exhibiting a sudden increase of the bridge response in particular directions before the bridge is able to rebalance. In the meantime, the bridge usually experiences dynamic vibrations under service loads when cable breakage occurs. The maximum response in any direction (e.g., vertical or torsional) will be achieved when the instantaneous bridge response under service loads reaches a local peak value, and starts to move in the opposite direction coinciding with the sudden movement direction of the bridge

caused by the instantaneous unbalance as a result of cable loss. In other words, the instantaneous unbalance and rebalance process of the bridge structure subjected to cable breakage may reinforce or suppress the existing bridge vibration caused by service loads, depending on the exact time instant the cable loss occurs. For various bridge components such as bridge girder, pylons and remaining cables, it is not realistic to determine universally unfavorable initial states of cable breakage events. Since the instant when cable loss occurs is nearly impossible to predict, the bridge response exhibits certain randomness with different breakage dynamic initial states in a cable breakage event, which require appropriate probabilistic simulation based on reliability theory.

5.4.7 Influence of the service load conditions

To simulate an unfavorable service condition, busy traffic and wind loads with a steady-state wind speed of 20 m/s are applied on the prototype bridge. The combined effects of stochastic traffic and wind loads are investigated assuming cable 1a fails at 20.00 second in the three comparative cable-breakage cases: with stochastic traffic and wind, with only stochastic traffic and with only wind. The time histories of vertical, lateral and torsional displacement at the girder joint connecting to the breaking cable are shown in Figs. 5.12a, b and c, respectively. It is found that the extreme vertical dynamic response of the bridge after cable breakage reaches the largest values when both stochastic traffic and wind are applied. The largest extreme lateral response occurs in the breakage event when only wind excitations are applied. The extreme torsional responses are close for the breakage events with both traffic and wind excitations and with only wind excitations, both of which are much larger than that in the breakage event with only traffic excitations. Before cable-breakage occurs, lateral and torsional dynamic response of the bridge is dominated by wind excitations while vertical response is influenced by both wind and traffic excitations. By comparing the post-breakage dynamic responses in the case with both traffic and wind and the case with only wind, it is found that the inclusion of traffic on top of wind excitations causes suppression of the lateral peak response, as evidenced by the reduced extreme value and standard deviation. The extreme post-breakage response in terms of vertical response in the wind-only case has considerably smaller value

than that in the other two cases. Such a phenomenon suggests that although wind load dominates lateral and torsional response, stochastic traffic load is essential to capturing the extreme post-breakage response of the bridge girder in the vertical direction. It can be concluded from the comparative studies that fully coupled consideration of the bridge-wind-stochastic traffic is important to cable-breakage simulation of long-span bridges by capturing the extreme values of both pre-breakage and post-breakage responses.



(c) Torsional displacement

Figure 5.12 Vertical, lateral and torsional displacement histories in the breakage cases with traffic and/or

wind

5.5 Conclusions

A new nonlinear time-progressive dynamic simulation framework was established to simulate cable breakage extreme events on cable-stayed bridges in the time domain based on finite-element (FE) formulation. Unlike existing studies, such a simulation methodology directly simulates a cable breakage event through nonlinear iterations, which can start from dynamic initial states at random time instants. The cable breakage process is realized by both applying the counteracting forces and also physically reducing the area of the breaking cable within the total breakage duration. Through the simulation methodology, the general cable loss incident can be more realistically characterized in terms of both the structural elemental configuration and force condition. Starting from the initial displacement vectors from nonlinear static analysis, the nonlinear dynamic analysis is conducted to simulate the cable-breakage incidents, in which the coupling effects among the bridge, individual vehicles from stochastic traffic and wind, as well as various sources of geometric and material nonlinearities are fully considered.

After the framework is developed, a prototype long-span cable-stayed bridge as well as the associated traffic, was selected to demonstrate the methodology. Although only the breakage of a single cable was demonstrated as the numerical example, the proposed methodology can be readily applied to simulate the multi-cable breakage events occurring simultaneously or progressively. The displacement responses before and after the breakage occurs are demonstrated for the bridge and the representative vehicle. The influence on the self-excited wind force and coupling bridge-traffic interaction forces due to the breakage of stay cables are investigated. Several critical factors, such as the dynamic initial states at breakage, service load condition from wind and traffic, are investigated for the simulation of cable-breakage events. Main findings drawn from the parametric studies are summarized as follows:

Bridge local dynamic responses close to the ruptured cable experience abrupt amplifications in vertical, lateral and torsional directions after cable breakage occurs. The vehicle vertical response is more likely to be influenced by the cable-breakage incident when the vehicle is close to the failed cable at the moment of cable breakage. Unlike the vertical response, the lateral response of the vehicle doesn't have significant change due to the breakage of the stay cable.

- The breakage of stay cables significantly influences the motion-dependent interaction forces, including the self-excited forces due to the bridge-wind interaction and the coupling forces due to the bridge-traffic interaction, especially at the girder locations close to the breaking cable.
- Cable breakage can occur at any time instant on a vibrating bridge under service loads with different dynamic initial conditions. In both vertical and torsional directions, the maximum response will be achieved when the instantaneous bridge response under service loads reaches a local peak value, and start to move in an opposite direction coinciding with the direction of movement caused by the instantaneous unbalance as a result of the cable loss. The instantaneous unbalance and rebalance process of the bridge structure subjected to cable breakage may reinforce or suppress the existing bridge vibration caused by service loads, depending on the exact moment the cable loss occurs. The bridge response exhibits certain randomness with different breakage dynamic initial states in a cable breakage event, highlighting the needs of probabilistic simulation and appropriate consideration of various uncertainties based on reliability theory.
- The comparative studies with different service load conditions show that to consider service load excitations from stochastic traffic and wind during cable rupture events is necessary. At certain time instants when the cable breakage occurs, the presence of stochastic traffic and wind may cause larger bridge response, which could control the design of the cable breakage event.

As a summary, cable loss incidents are important to cable-supported bridges, deserving more comprehensive studies. Such incidents can be caused by many different reasons, making the efforts on better understanding the process and consequences very challenging. As a part of explorative efforts, the present work introduced a new framework for cable-loss studies, which may serve as an important stepping-stone for future studies. A lot of work needs to be done in the future to achieve better understanding of this complex phenomenon, such as experimental characterization of the cable-loss process caused by different reasons, progressive failure analysis of various cable-loss events etc.

CHAPTER 6 Numerical investigation of cable breakage events on long-span cable-stayed bridges under stochastic traffic and wind^{*}

6.1 Introduction

Stay cables of cable-stayed bridges may experience damages due to various reasons, such as natural hazards (e.g., wind, lightning), man-made hazards (e.g., vehicle collision, explosion, fire and intentional cutting) or structural deterioration (e.g., fatigue cracks). Without being exhaustive, as shown in Table 1.3, five recent cable-loss incidents with various causes during the past ten years are summarized, which led to one bridge collapse and mild to moderate damage for the rest. Owing to various causes of cable loss events, considerable variations and uncertainties exist on the time-dependent cable breakage process, such as time duration of breakage, time-transient process of breakage and also time instant when cable breakage occurs. So far, very little information is available about the impact from these possible variations and uncertainties on the performance assessment of cable-loss events on cable-stayed bridges.

In Chapter 5 (Zhou and Chen 2015c), a new finite-element-based nonlinear dynamic simulation platform for breakage of stay cables was proposed with several important improvements over the existing simulation techniques. These include improved modeling of cable-loss process, consideration of realistic wind and traffic loads through the complex bridge-traffic-wind interaction model on a finite-element (FE) basis and incorporation of geometric and material nonlinearities from various sources. Such a platform offers so far the most advanced and versatile simulation tool applicable to various kinds of cable-stayed bridges, including those slender ones susceptible to wind. The objective of the present study is to carry out a comprehensive investigation of cable-loss phenomena with the recently developed platform to improve the knowledge about various cable-loss events and their impacts on bridge safety. Specifically, a comprehensive parametric investigation on a prototype long-span cable-stayed bridge is carried out to study the impact on the post-breakage bridge response from cable-breakage parameters such as cable-

^{*} This chapter is adapted from a published paper by the author (Zhou and Chen 2015d) with permission from Elsevier.
breakage process, duration and initial state. The influences of dynamic excitations from stochastic traffic and wind as well as the coupling effects on the post-breakage response of the bridge are also investigated. Response envelopes on the bridge girders, pylons and stay cables due to breakage of single stay cable are obtained using the nonlinear dynamic simulation methodology in the present study and then compared to those obtained with the equivalent static analysis approach as introduced by the Post-Tensioning Institute (PTI) in order to shed some lights on the applicability of the popular codified analytical approach on longspan bridges.

6.2 Parametric Analysis of Single-cable Breakage Scenarios

As discussed in the previous section, a rational characterization of cable-loss events involves the uncertainties about cable breakage, such as duration, time-transient processes, and the time instant of occurrence. Besides, various possible combinations of service loads for different cable-breakage events present another challenge. In order to better understand the nature of cable-loss, a parametric study is conducted for these variables associated with single-cable breakage events. To facilitate the readers understanding the following sections, a 3-D sketch of the prototype bridge is shown in Fig. 6.1. The mechanical and sectional parameters are given in Table 5.1 and Table 5.2, respectively, in Chapter 5. The three lowest frequencies of the bridge are 0.405, 0.652 and 0.682 Hz, which correspond to the symmetric vertical, symmetric lateral and anti-symmetric vertical mode shapes, respectively.



Figure 6.1 3-D sketch of the prototype long-span cable-stayed bridge

6.2.1 Influence of cable breakage durations and time-transient processes

Depending on the causes, the actual cable failure processes could vary significantly from one to another. Some breakage may occur abruptly and the whole capacity of the cable gets lost within a very short time period, while others may partially lose some capacity until finally reaching total failure. As a relatively new topic, there is currently lack of sufficient information on defining the specific parameters associated with various cable-loss scenarios from different causes, such as fire, lightning, blast and vehicle impact. Although some parametric studies were conducted on several parameters of cable-loss events (Ruiz-Teran and Aparicio 2009), realistic descriptions of cable-loss processes caused by different incidents are not yet available. In the present study, general breakage phenomenon of stay cables is assumed to occur independently from the causes. Based on the observations made in some existing studies (Ruiz-Teran and Aparicio 2009; Mozos and Aparicio 2011), a cable loss process is defined in the present study with two variables: total breakage time duration t_{total} and time-transient cable area loss function l_{rel} . Four total breakage time durations t_{total} are assumed in the following sensitivity analyses: no longer than 0.04 seconds, exactly 1.0 seconds, 2.5 seconds and 4.0 seconds. These time durations are selected based on some reasonable assumptions developed in the limited existing studies. Since there is little data or existing knowledge about the actual cable breakage process, the same assumption of exponential function once adopted by Ruiz-Teran and Aparicio (2009) is followed here primarily because of its computational convenience. The time-dependent cable area loss function l_{rel} is the ratio of lost cable cross-section area to total cable cross-section area, which is assumed as an exponential function of the relative breakage time t_{rel} for the latter three t_{total} values. The time-dependent cable area loss function l_{rel} can then be defined as:

$$l_{rel}(t_{rel}) = t_{rel}^{a} \tag{6.1}$$

where t_{rel} is defined as the ratio of the elasped time t_{elap} after breakage starts over the total breakage time duration t_{total} . *a* is the exponential factor, assumed to be 0.2, 0.05, 1, 5 and 20.

By changing the exponential factor α as defined in Eq. (6.1), a breakage process can be modeled with different patterns within the same breakage duration. For example, when α is set to be 1, the function is a linear breakage process indicating that the breakage occurs uniformly during the breakage duration. Since the smallest t_{total} is less than or equal to 0.04 seconds which is same as the time step used in the nonlinear dynamic analysis, it is considered as a type of abrupt cable breakage scenario. The timedependent cable area loss function l_{rel} for abrupt breakage scenario is defined as two similar cases: (1) the cable loses 100% area at the instant when the breakage starts (t = 0) (Process 1 as shown in Fig. 6.2), or (2) the cable remains intact at the instant when the breakage starts (t = 0) and the cable loses 100% area at the beginning of next time step ($t = t_{total} = 0.04$ s) (Process 7).

Therefore, a total of 7 processes are defined: Processes 1 and 7 represent the two abrupt cable breakage cases and Processes 2-6 represent the cases defined in Eq. (6.1) with the five α values as defined above, respectively. The results of time-dependent cable area loss function l_{rel} versus t_{rel} are shown in Fig. 6.2. Considering the symmetric nature of cable loss processes as shown in Fig. 6.2, only Processes 1-4 are studied in the following sensitivity studies.



Figure 6.2 Demonstration of different cable loss processes

To better study the impact of cable loss duration and process without mixing contributions from other factors, no service load is considered and cable breakage starts at rest. To explore the influence from breakage duration, Cable 1a is assumed to break within four different total breakage time durations: abrupt (within 0.04 second), 1.0 second, 2.5 second and 4.0 second. For the abrupt breakage case (within 0.04 s), the time-dependent cable area loss function I_{rel} follows the Process 1 (i.e. 100% area lose at t = 0). For the other three total breakage time durations (1.0, 2.5 and 4.0 s), I_{rel} follows Eq. (6.1) with a linear process ($\alpha = 1$; Process 4). Assuming cable breakage starts at t = 0, the time history responses of vertical displacement at the mid-span joint of the south bridge girder are shown in Fig. 6.3 for different total breakage time durations. It is clear that abrupt breakage durations. Larger dynamic response will be induced in the breakage cases with shorter breakage durations especially when the breakage duration is less than the fundamental period of the bridge, the dynamic response doesn't have remarkable change when the total breakage duration increases.



Figure 6.3 Vertical dynamic responses at mid-span of south bridge girder with different breakage

durations

In addition to total breakage time duration, the impact from time-dependent cable area loss function l_{rel} is also investigated. It is assumed that Cable 1a breaks within the total time duration of 1.0 s following Processes 1 through 4 as defined previously. Assuming cable breakage starts at t = 0, the vertical displacement time histories at the mid-span of the bridge girder are displayed in Fig. 6.4 for different

processes. It is found that Process 1 with abrupt loss of cable and Process 4 with linear loss of cable cause the largest and smallest dynamic response among all four breakage processes, respectively. Compared to the total breakage duration, the time-dependent cable area loss function I_{rel} has relatively less impact on the bridge response. The extreme dynamic responses induced in the cases with breakage processes 2 and 3 are between the corresponding values in the case with breakage processes 1 and 4. Dynamic response is observed to become larger as the breakage process approaches abrupt loss process while it becomes smaller as the breakage process approaches linear loss process. Since an accurate characterization of the total breakage duration as well as the time-transient cable area loss function is not yet available, uncertainties associated with these two factors are found necessary to be considered appropriately.



Figure 6.4 Vertical dynamic response at mid-span of south bridge girder with different breakage

processes

In the parametric studies introduced above, abrupt breakage of cable is found to generally induce the largest dynamic response compared with other breakage processes, thus becoming the most critical breakage process of cable loss. The simulations in the following sections will be based on the abrupt cable loss assumption unless otherwise noted.

6.2.2 Influence of the time instant when cable breakage occurs-dynamic initial states of cable breakage

Long-span cable-stayed bridges vibrate considerably under service wind and traffic loads. Therefore, depending on the exact instant when cable breakage occurs, the initial state of cable breakage is not only dynamic with non-zero velocity and acceleration, but also exhibits stochastic characteristics. In a previous study by the authors (Zhou and Chen 2014a), dynamic initial states of cable breakage events only caused

by stochastic traffic were studied. In the present study, more comprehensive studies are conducted to investigate the influence from the associated dynamic initial states of cable breakage subjected to both traffic and wind service loads. The vertical and torsional displacements at the mid-span of the south bridge girder are investigated assuming Cable 2a fails abruptly at around 20 second when moderate traffic and wind excitations of 20 m/s are applied on the bridge.

Two dynamic extreme initial states for vertical displacement are selected at 19.32 second and 20.52 second, when the mid-span of the bridge girder moves to the uppermost and lowest positions, respectively. For comparison purposes, another time instant, 20 second, is chosen between the two selected time instants. Thus there are totally three representative dynamic initial states for vertical response, i.e. breakage of Cable 2a starting at 19.32 second (dynamic initial state 1), 20 second (dynamic initial state 2) and 20.52 second (dynamic initial state 3). Nonlinear dynamic analyses are conducted for the three representative dynamic initial states with the vertical and torsional displacements at the mid-span joint of bridge girder presented in Figs. 6.5a and 6.5b, respectively. It is found in Fig. 6.5a that the vertical displacement response reaches the largest and the smallest extreme values with initial states 1 and 3, respectively. Similar to the observations made in the previous study (Zhou and Chen 2014a), the vertical displacement of the bridge girder joint experiences local maximum and minimum values when the breakage occurs at the time instant when the joint reaches the uppermost position and starts to move downward, and the lowermost position and starts to move upward, respectively. As shown in Fig. 6.5b, the maximum and minimum extreme initial states (i.e. initial state 1 and 3) identified based on vertical response do not cause the maximum and minimum extreme response of torsional displacement response. A comprehensive search among different breakage instants is made to identify the critical initial states for torsional displacement. It is found that at the breakage instants of 20.08 s and 19.68 s, the bridge torsional displacement reaches the largest and smallest values with the presence of service loads, respectively.



(b) Torsional displacement

Figure 6.5 Dynamic displacement at mid-span of bridge girder with breakage initial states 1-3

Although critical initial states causing the maximum and minimum extreme responses in different directions (e.g., vertical and torsion) are not the same, some general trends can still be identified from the previous results. It is found that cable breakage will cause an instantaneous unbalance of the bridge system, accordingly exhibiting a sudden increase of the bridge response in particular directions before the bridge is able to rebalance. In the meantime, the bridge usually experiences dynamic vibrations under service loads when cable breakage occurs. The maximum response in any direction (e.g., vertical or torsional) will be achieved when the instantaneous bridge response under service loads reaches a local peak value, and starts to move in an opposite direction to the sudden movement direction of the bridge caused by the instantaneous unbalance as a result of cable loss. In other words, the instantaneous unbalance and rebalance process of the bridge structure subjected to cable breakage may reinforce or suppress the existing bridge vibration caused by the service loads, depending on the exact time instant the

cable loss occurs. For various bridge components such as bridge girder, pylons and remaining cables, it is not realistic to determine universally unfavorable initial states of cable breakage events. Since the instant when cable loss occurs is nearly impossible to predict, the bridge response exhibits certain randomness with different breakage dynamic initial states in a cable breakage event, which require appropriate probabilistic simulation based on the reliability theory.

6.2.3 Impact on internal forces from cable loss events – A baseline scenario

It is possible for cable breakage events to start at rest without the presence of excitations from stochastic traffic or turbulent wind, especially when sufficient warning time is available to evacuate the traffic and wind is very mild at the same time. Such a scenario also serves as a baseline case, which provides useful information about the impact on bridge performance from cable breakage only, without mixing contributions from service loads. Cable loss can occur on any cable of the bridge and a comparative study of single-cable loss event occurring on three representative cables is carried out. Single-cable breakage scenario of Cable 1a, 2a or 3a is assumed to occur respectively at 20 second to be consistent with the following studies. However, it is noted that cable breakage in the baseline scenario starts at rest and no dynamic excitations are applied during the whole simulation time period. The impact on the dynamic internal forces of the bridge girder, pylon column and cables is investigated through conducting nonlinear dynamic analyses on the three cable breakage scenarios.

6.2.3.1 Dynamic response on bridge girder

The extreme dynamic responses in terms of vertical bending moment on the bridge girder are obtained from the nonlinear dynamic analysis of the bridge for breakage cases of Cable 1-3a. For comparison purposes, the corresponding static bridge response of the intact bridge without cable loss is obtained from nonlinear static analysis of the bridge. The vertical bending moments along the whole bridge under all the three cable loss events and one undamaged bridge case are plotted in Figs. 6.6a-b. Positive and negative vertical bending moments on the bridge girder are defined as those causing tension on the bottom slab and the top slab of the bridge girder, respectively. It is found from Fig. 6.6a that cable

breakage will generally cause the largest increase of positive bending moments at the girder segments closest to the failed cable. Among all the three cases, breakage of Cable 1a causes the largest extreme positive bending moments at the middle point of the main span, which may control the bridge girder design against single-cable breakage event. For all the three representative cable-breakage scenarios, positive dynamic bending moments at the corresponding bridge girder locations closest to the failed cables all exhibit considerable amplification from those at the same locations when the bridge was intact. Among the three representative cable-breakage events, the largest dynamic amplification is observed in the breakage case of Cable 2a. Different from positive bending moments, as shown in Fig. 6.6b, extreme negative bending moments on the bridge girder closest to the failed cable do not exhibit significant amplifications from those of the intact bridge, rather causing significant increase on the bridge girder locations around the nearest remaining cables. For example, breakage of Cable 1a will cause notably amplified negative bending moment at the girder locations connecting Cable 2a and 3a. Similarly, breakage of Cable 2a will considerably amplify the negative bending moments at the girder locations connecting Cable 3a and 1a. The maximum dynamic response ratios (DRR), defined as the ratio of the response in the breakage case to that in the undamaged case, are -4.17 and 1.24 for vertical bending moments at the girder location connecting Cable 2a and mid-span, respectively. The negative sign of the DRR indicates the change of moment direction. It should be noted that the absolute extreme bending moment at the mid-span is still larger than that at the girder location of Cable 2a in the above case although the DRR of the latter is larger.



(a) Positive vertical bending moments



(b) Negative vertical bending moments

Figure 6.6 Extreme vertical bending moments along bridge girder

Abrupt single-cable breakage usually experiences instantaneous unbalance of the bridge structure before it reaches its new balanced position. During the process, the bridge may experience considerably larger torsional moments. The nonlinear time history analyses provide the information about the extreme positive and negative torsional moments along the bridge girder as shown in Figs. 6.7a and b, respectively. The positive torsional moment is defined following the right hand rule rotating about the positive *x* direction. It is seen from Figs. 6.7a-b that positive and negative extreme torsional moments are generally anti-symmetric about the girder location closest to the cable breakage. For both positive and negative torsional moments, significant increase has been observed at the girder locations near the failed cable and also on nearby cables. Among all the three representative cable loss events, the breakage of Cable 2a generally induces the largest extreme torsional moment and also dynamic amplifications as compared to the undamaged case. The maximum DRRs in the breakage case of Cable 2a for torsional moment are - 11.95 and -24.47 at the girder location connecting Cable 2a and mid-span, respectively. This result shows that some individual cable breakage events could be more critical than others in terms of internal forces on the bridge, and the critical cable-breakage scenario, possibly controlling the design, needs to be appropriately identified on a case-by-case basis.



(b) Negative torsional moments

Figure 6.7 Extreme torsional moments along bridge girder

6.2.3.2 Dynamic response on bridge pylons

Extreme positive and negative longitudinal bending moments of the south column on the west pylon are evaluated and compared with those on the same pylon column of the intact bridge from nonlinear static analysis (Fig. 6.8). Positive longitudinal bending moments of the bridge pylon are defined as those causing tension on the east side and compression on the west side of the pylon column.



Figure 6.8 Extreme longitudinal bending moments along bridge pylon

As shown in Fig. 6.8, breakage events of all the three representative cables will cause significant increase of extreme bending moments on the pylon column in both positive and negative directions, particularly at the base of the pylon column. Compared to Cable 1a and Cable 3a, breakage of Cable 2a induces notably larger extreme bending moments on the pylon column in both positive and negative directions. It is found that the critical location for the pylon column subjected to cable breakage events is the pylon base with the largest positive and negative longitudinal bending moments. As compared to the undamaged case, the DRRs at the pylon base can reach up to -10.9 and 11.0 for positive and negative bending moments, respectively.

The extreme torsional moments along the bridge pylon in positive and negative directions are shown in Fig. 6.9. It is again found that breakage of Cable 2a induces the largest torsional moments along the pylon column among the three breakage cases. The critical location in terms of torsional moments on the pylon column for the three cable breakage cases is at the pylon base and the DRRs for torsional moment are between 19.6 and -18.6, indicating that large dynamic amplification may occur on both sides of the pylon column.



Figure 6.9 Extreme torsional moments along the west bridge pylon

6.2.3.3 Dynamic response on bridge cables

The dynamic responses of the remaining cables on the bridge are investigated in the three single-cable breakage events. The maximum tension forces at the lower ends of remaining cables among Cable 1-6a are evaluated in the breakage cases of Cable 1-3a through nonlinear dynamic analysis and compared with the static cable forces of the undamaged bridge structure (Fig. 6.10). Not surprisingly, it is found that the remaining cables close to the ruptured cable are more likely to be influenced by a cable breakage incident. For instance, abrupt breakage of Cable 1a induces the largest increase of tension force at Cable 2a from that of the undamaged bridge. The DRRs at the locations of Cables 1a and 3a in the breakage case of Cable 2a are 1.77 and 2.76, respectively. Although it seems significant that the remaining cable forces will increase by 77% to 176% subjected to a single-cable loss event compared with those on the undamaged bridge, the local response amplification on the bridge girders and pylons as discussed above

are actually much more substantial in terms of DRR. It is found that the breakage of Cable 2a generally induces the largest extreme tension forces on the remaining cables among the breakage cases of Cable 1-3a. Similar conclusions are drawn for the internal force or moment response of the bridge girder and pylon. The influence of the cable breakage event on the bridge response depends on several factors, such as the stay cable being on the side span or main span, the distance between two adjacent remaining cables, the cable inclination angle and the cable length, etc. In the present study, the distances between any two adjacent remaining cable/pylon supports for the Cable 1a, 2a and 3a are very close, which are 103.5 m, 107.6 m, 107.6 m, respectively. Therefore, the distance between two adjacent remaining cable/pylon supports for the bridge is probably because the ability for the east bridge tower (not connecting with the breaking cable) to withhold additional girder gravity force after cable breakage occurs outweighs the force effects on the west bridge tower (connecting with the breaking cable) to we bridge tower (connecting with the breaking cable) due to the larger inclination angle of the breaking cable. It is noted that the most disadvantageous breakage situation may vary among different bridges and the accurate determination of the most disadvantageous scenario requires finite element analysis of the breakage events on the specific bridge.



Figure 6.10 Maximum tension forces of remaining cables in the breakage case of cable 1-3a

6.2.4 Impact on internal forces/moments due to excitations from stochastic traffic and wind

6.2.4.1 Dynamic excitations from stochastic traffic

The panic driving behavior of the vehicles may exist after the drivers are able to detect the incidents causing the cable loss. The influences of the emergency incidents on the driving behavior are very complicated and unfortunately existing related studies are still very limited. Due to the lack of appropriate model and data to characterize the panic driving behavior, the driving behavior following the cable breakage is assumed to remain unchanged in the present study. Such an assumption is valid for the instants immediately following the cable-loss incidents before the drivers are able to detect and respond to the incidents and may be approximate for the extended time after the cable-loss incidents. It is found in previous sections that breakage of Cable 2a will generally cause the largest dynamic internal forces or moments on bridge girder, pylon and cables among the three breakage cases of Cable 1a, 2a and 3a. In this section, the bridge dynamic performance is investigated for the cable breakage case of Cable 2a with the presence of stochastic traffic. As discussed earlier, stochastic traffic flow is generated using the CA model with two different traffic densities, which are 31 and 16 vehicles per kilometer per lane representing busy traffic and moderate traffic conditions, respectively. The breakage of Cable 2a is assumed to occur at 20 second in the three comparative cases with busy traffic, moderate traffic and no traffic, respectively. In previous sections, the critical locations of study have been identified at several locations when Cable 2a is assumed to fail, such as mid-span joint of the south bridge girder for vertical bending moment, the bridge pylon base for longitudinal bending moment, and Cable 1a for tension force. The dynamic responses of these corresponding critical locations will be studied respectively in the following to evaluate the critical impacts from excitations due to different service loads.

The time histories of vertical bending moment at the mid-span of the bridge south girder, longitudinal bending moment at the pylon base and tension force of Cable 1a are demonstrated in Figs. 6.11a, b and c, respectively. Before the cable breakage occurs at 20 second, the pre-breakage internal forces or bending moments of the bridge girder or cables are generally larger in the case with busy traffic than those in the case with moderate traffic or no traffic. In contrast, the pre-breakage moment at the base of the bridge

pylon is small and exhibits little difference under different traffic densities. As shown in Fig. 6.11, for the three bridge members (i.e. bridge girder, pylon and Cable 1a), post-breakage (after t = 20 s) responses all show significant amplification over the respective pre-breakage responses. The largest extreme values of post-breakage responses for all the three bridge members actually occur during the first 1-2 cycles, rather than immediately after cable breakage occurs. For bridge girder (Fig. 6.11a) and Cable 1a (Fig. 6.11c), different traffic densities do cause considerably different extreme post-breakage responses and interestingly, moderate traffic condition, instead of busy traffic condition as expected intuitively, causes the largest post-breakage extreme response at mid-span of bridge girder. This phenomenon may be caused by several possible reasons. Considering that the traffic flow is stochastically simulated using CA model, two different patterns of traffic flow may have different extreme response of the bridge in a cable breakage event even for the same traffic flow density because of the stochastic nature of the simulated traffic flow. The different dynamic excitations from traffic loads may affect the breakage initial state and further influence the extreme response after cable breakage occurs. In addition, due to the fewer vehicles in the moderate traffic flow than in the busy traffic flow, the vehicles in the moderate traffic flow generally move faster than those in the busy traffic flow, which may generate larger interaction effects between the bridge and vehicles. For the bridge pylon (Fig. 6.11b), traffic has little effect on the postbreakage base moment, similar to the observation of pre-breakage response. Unlike the bending moment on bridge girder and tension force of Cable 1a, cable breakage itself causes large cyclic change of the bending moment at pylon base with close-to-zero mean value.



(a) At mid-span of bridge girder



Figure 6.11 Internal force/moment histories at critical location of bridge girder, pylon and cables

6.2.4.2 Dynamic excitations from wind

In this section, the dynamic performance of the prototype bridge subjected to single-cable breakage (Cable 2a) is evaluated when only wind excitation is considered. Two steady-state wind speeds are studied in the cable breakage cases of Cable 2a starting at 20 second: 20 m/s representing moderate wind and 40 m/s representing strong wind in order to investigate the influence on dynamic response of the bridge due to different wind excitations. 40 m/s wind speed is selected to realistically represent the strong wind scenario when the bridge is typically closed to traffic. Under 20 m/s wind speed, traffic may still be remained on the bridge. The wind-only case with 20 m/s wind speed in this section is therefore primarily for investigation and comparison purposes. The time histories of torsional moment at the mid-span of the bridge girder, torsional moment at the pylon base and tension force histories of Cable 1a are shown in Figs. 6.12a, b and c, respectively.







(c) Tension force of cable 1a

Figure 6.12 Internal force/moment of bridge in the breakage cases of Cable 2a

It is found that as wind speed increases, the initial static displacement of the bridge from nonlinear static analysis under steady-state wind load and gravity increases accordingly. Different from the traffic-only cases, the torsional moments of the three bridge components, including bridge pylon, all exhibit increased extreme values for pre-breakage and post-breakage responses when wind speed increases.

Similar to traffic-only case in the previous section, cable breakage causes dramatic increase of the torsional moments on the bridge girder and pylon, as well as tension forces for the cable. Because of the new equilibrium position of the bridge, post-breakage dynamic responses of the three bridge members all maintain different mean values from the respective ones for the pre-breakage responses. Except for bridge cables, of which the tension force reaches the maximum extreme values 1-2 cycles after the occurrence of cable breakage, torsional moments for both bridge girder and pylon base reach the peak values during the first cycle after the occurrence of cable breakage. In addition, it is observed that the dynamic impact from cable breakage increases nonlinearly as wind speed increases. This is mainly due to the fact that the wind forces, including static, self-excited and buffeting forces are closely related to the square of the steady-state wind speed. The demands for internal forces or moments of the bridge structure increase more significantly as the wind speed increase from 20 m/s to 40 m/s than those with wind speed increasing from 0 to 20 m/s.

6.2.4.3 <u>Combined dynamic excitations from stochastic traffic and wind</u>

Although a consistent criterion is not yet available across the United States and the rest of the world, some bridges are often closed to traffic for safety concern when mean wind speed at bridge girder gets high (Chen et al. 2009). For example, AASHTO assumes a bridge may be closed to traffic when wind speed exceeds 25 m/s, although under which many bridges are actually not closed (Chen and Wu 2010). To simulate a typical service condition with both normal traffic and relatively strong wind, moderate traffic and wind loads with a steady-state wind speed of 20 m/s are applied on the prototype bridge. The combined effects of stochastic traffic and wind load are investigated assuming Cable 2a fails at 20 second in the three comparative cable-breakage cases: with stochastic traffic and wind, with only stochastic traffic and with only wind. The time histories of vertical bending and torsional moments at the mid-span joint of the south bridge girder are shown in Figs. 6.13a and 6.13b, respectively. It is found that the extreme dynamic response of the bridge after cable breakage occurs, torsional moment of the bridge is dominated by wind excitations while vertical bending moment is influenced by both wind and traffic

excitations. The post-breakage dynamic responses in the case with both traffic and wind and the case with only stochastic traffic are compared. It is found that the inclusion of wind excitation on top of the stochastic traffic excitation causes slight increase of the peak moments immediately after cable breakage, but gradually suppresses the dynamic responses in both vertical and torsional directions, evidenced by reduced standard deviation. In addition, it is demonstrated that the dynamic impact due to cable breakage damps out faster when wind excitations are included, likely due to the aerodynamic damping effects through bridge-wind coupling. Similar observations are made by comparing the bending/torsional moments at the base of the pylon column and tension forces at Cable 1a among the breakage cases with different combinations of wind and traffic. It is found that the extreme post-breakage response in terms of vertical bending moment in the wind-only case has considerably smaller value than those in the other two cases. Such a phenomenon suggests that although wind load usually dominates torsional moment, stochastic traffic load is essential to capturing the extreme post-breakage response of the bridge girder in terms of both bending and torsional moments. It can be concluded from the comparative studies that fully coupled consideration of the bridge-wind-stochastic traffic is important to cable-breakage simulation of long-span bridges by capturing the extreme values of both pre-breakage and post-breakage responses.



(a) Vertical bending moment





Figure 6.13 Vertical bending and torsional moment in the breakage cases with traffic and/or wind

6.3 Response Envelope Analysis and Comparison with the Design Specification

Without specifying the applicable types and spans of cable-stayed bridges, PTI regulations (PTI 2007) are usually followed to carry out the design, testing and installation of stay cables, in which the equivalent nonlinear static analysis approach is recommended. Due to the unique characteristics such as sensitivity to wind and the large amount of traffic for long-span cable-stayed bridges as compared to those with shorter spans, the applicability of the recommended analytical approach on long-span cable-stayed bridges is not clear. In the following section, a comparison between the results from the proposed nonlinear dynamic analysis approach and those from the approximated approach recommended by the PTI design specification is made. From the design perspective, response envelopes along major bridge elements are crucial in order to capture the critical sections and also the worst-case scenarios. Single cable breakage events with the presence of both stochastic traffic and wind excitations at a steady-state wind speed of 20 m/s are selected for the following study. For comparison purposes, the equivalent nonlinear static analysis with a DAF (Dynamic Amplification Factor) of 2.0 as recommended by PTI (2007) is also conducted on the prototype bridge. The proposed nonlinear dynamic analysis approach adopts stochastic traffic load, which cannot directly be applied to the equivalent nonlinear static analysis. Existing design traffic load in the AASHTO LRFD specification (AASHTO 2012), such as HL-93 primarily calibrated from and used for short-span bridges, is not directly applicable to long-span bridges. In order to make rational comparison, an equivalent uniformly distributed traffic load 3.37 kN/m along the bridge is quantified,

under which the static vertical displacement at the midpoint of the main span equals the mean value of the dynamic vertical displacement of the proposed nonlinear dynamic analysis with stochastic traffic flow. To consider the wind loads on bridges, the steady-state wind forces corresponding to wind speed of 20 m/s are also applied on the bridge structure in the equivalent nonlinear static analysis for cable breakage events.

6.3.1 Response envelopes for bridge girder

The response envelopes along bridge girder in terms of vertical bending moment, lateral bending moment and torsional moment due to single-cable breakage are demonstrated in Fig. 6.14a, b and c, respectively. It is found that the equivalent static analysis with a DAF of 2.0 generally provides smaller values in both positive and negative bending moments and torsional moments with only a few exceptions. It suggests that equivalent static analysis with a DAF of 2.0 for the cable loss event as defined by PTI may not be able to capture the maximum responses on long-span bridge girders and a detailed nonlinear dynamic analysis may be needed for the cable-breakage design.



(a) Vertical bending moments



(c) Torsional moments



6.3.2 Response envelopes for bridge pylons

Figs. 6.15a, b and c presented the response envelopes along the south pylon column in terms of longitudinal bending moment, transverse bending moment and torsional moment due to single-cable breakage, respectively. It is found in Fig. 6.15a that the equivalent static analysis with a DAF of 2.0 will give much smaller longitudinal bending moment envelopes than those from nonlinear dynamic analysis. The transverse bending moment envelopes (Fig. 6.15b) from equivalent static analysis have relatively similar trend to those from nonlinear dynamic analysis but with unsafe results. For the most part, the

torsional moments obtained from nonlinear static analysis are smaller than those from nonlinear dynamic analysis with a few exceptions, as shown in Fig. 6.15c. It is concluded that equivalent nonlinear static analysis with a DAF of 2.0 will incur non-neglegible error in determining the response envelopes of bridge pylon, and therefore is unsafe for the design of cable-stayed bridge pylons subjected to single-cable loss when moderate wind (20 m/s) and moderate traffic exist.



Figure 6.15 Response envelopes of south pylon column using nonlinear dynamic and equivalent static analysis

6.3.3 *Response envelopes for stay cables*

As shown in Fig. 6.16, the response envelopes in terms of tension forces of cables have similar shapes using both nonlinear dynamic and static analyses. Equivalent static analysis with a DAF of 2.0 is found unsafe for the design of Cable 1a-4a, although being safe for the design of Cable 5a and 6a. Considering cable arrangements can vary significantly among different cable-stayed bridges, bridge-specific investigation on the tension forces of the remaining cables during cable-loss events is felt needed.



Figure 6.16 Tension force envelopes for stay cables using nonlinear dynamic and equivalent static

analysis

6.4 Conclusions

Single-cable breakage events have been comprehensively simulated on a representative long-span cable-stayed bridge with the newly proposed nonlinear dynamic methodology considering dynamic excitations from traffic and wind. Firstly, parametric studies on several critical variables during singlecable breakage events were conducted such as cable breakage duration, time-dependent cable area loss pattern, and the time instant of cable breakage occurrence and associated dynamic initial states. The internal forces/moments on bridge girder, pylons and remaining cables due to different single-cable breakage scenarios were then explored based on the nonlinear dynamic analysis results. Furthermore, the dynamic influence of stochastic traffic, wind excitations and their combined effects on the internal force and moments were studied. Due to the lack of relevant data, some assumptions had to be made in order to carry out the present study. These assumptions include the time duration and process of the cable breakage process and the driving behavior following the cable breakage events. Main findings drawn from the parametric studies of single-cable breakage cases are summarized as follows:

 Larger dynamic response will be induced in the breakage cases with shorter breakage durations when the breakage duration is less than the bridge fundamental period. Dynamic response is observed to become larger as the breakage process approaches abrupt loss process while it becomes smaller as the breakage process approaches linear loss process. Given the fact that rational characterization of the total breakage duration as well as the process of cable area loss is not yet available, uncertainties associated with these two factors are found necessary to be considered appropriately.

- Cable breakage can occur at any time instant on a vibrating bridge under service loads with different dynamic initial conditions. In both vertical and torsional directions, the maximum response will be achieved when the instantaneous bridge response under service loads reaches a local peak value, and start to move in an opposite direction coinciding with the direction of movement caused by the instantaneous unbalance as a result of the cable loss. The instantaneous unbalance and rebalance process of the bridge structure subjected to cable breakage may reinforce or suppress the existing bridge vibration caused by service loads, depending on the exact moment the cable loss occurs. The bridge response exhibits certain randomness with different breakage dynamic initial states in a cable breakage event, highlighting the needs of probabilistic simulation and appropriate consideration of various uncertainties based on the reliability theory.
- Compared with the vertical bending moments of bridge girder, the torsional moment of bridge girder, the bending and torsional moments of bridge pylon columns are more likely to be influenced in a cable breakage event. Tension forces of the remaining cables received least influence with relatively smaller amplification factors during the cable breakage event.
- Cable breakage with simultaneous presence of busy stochastic traffic and wind will generally induce larger internal force or moment along bridge girder, pylon and cables than those in the cases which do not consider the combined effects from traffic and wind. Fully coupled consideration of the bridgewind-stochastic traffic is found important to cable-breakage simulation of long-span bridges by capturing the extreme values of both pre-breakage and post-breakage responses.
- The breakage of a single cable on the prototype long-span cable-stayed bridge when service loads from stochastic traffic and wind loads are applied simultaneously will not cause the formation of plastic hinges on the bridge members, nor progressive collapse of the bridge structure. For the prototype long-span cable-stayed bridge being studied, it was found to be capable of withstanding the

breakage of a single cable under service traffic and wind loads without reaching any strength limit state.

Finally, a comparison was made between the response envelopes in terms of bending and torsional moments of different bridge components from the nonlinear dynamic analysis and also the equivalent nonlinear static analysis with a DAF of 2.0 as recommended by PTI. The results of the prototype bridge show that the equivalent nonlinear static analysis with a DAF of 2.0 is unsafe for most of the moment design of bridge girder, unsafe for the force design of some stay cables, and very unsafe and will incur large errors for the design of bridge pylons. It should be noted, however, that some observations about the results of using the PTI approaches may be specific to the bridge being studied and a more general assessment about the PTI specification cannot be made until more bridges are investigated.

CHAPTER 7 Fully coupled driving safety analysis of moving traffic on long-span bridge subjected to crosswind^{*}

7.1 Introduction

Highway vehicles may experience single-vehicle crashes under hazardous driving environments, such as strong crosswind, slippery road surface with rain, snow or ice, etc. (USDOT 2005). For traffic through flexible transportation infrastructures, such as bridges, single-vehicle crash risks were found to increase due to the dynamic coupling effects between the vehicle and the supporting structure (Baker et al. 1994; Guo and Xu 2006). Long-span bridges are usually built across straits or major rivers, and therefore they are more open than most roads with less blocking effects from surrounding environment, exposing vehicles to stronger crosswind. Long-span cable-supported bridges are flexible, susceptible to wind excitations and support considerable amount of vehicles on a daily basis. In addition, vehicles driven on long-span cable-supported bridges may also be temporarily shielded from the crosswind by the bridge tower or other nearby high-sided vehicles, experiencing sharp crosswind gust as well as higher accident risks. When vehicles are driven through a long-span bridge, the complex dynamic interactions among the wind, vehicles and bridge significantly affect not only the performance of the bridge, but also the safety of passing vehicles (Chen and Wu 2010; Chen and Cai 2004; Guo and Xu 2006). Therefore, a rational traffic safety assessment of passing traffic through a long-span bridge requires appropriate modeling of the critical coupling effects within the bridge-traffic-wind system.

In almost all the existing studies about traffic safety on bridges, vehicle safety is assessed with only one single vehicle at a constant driving speed (Baker 1986, 1987, 1991, 1994; Chen and Cai 2004; Guo and Xu 2006; Chen et al. 2009; Chen and Chen 2010). In reality, it is known that the traffic flow on a long-span bridge is typically stochastic and vehicle speeds may vary following some realistic traffic rules (Chen and Wu 2011). This is especially true when the bridge span is long and the total number of vehicles

^{*} This chapter is adapted from a published paper by the author (Zhou and Chen 2015b) with permission from Elsevier.

on the bridge at a time is not small. To assume only one vehicle on the bridge with constant speed cannot reflect the realistic situations on most long-span bridges. Among all the technical hurdles preventing researchers from carrying out more realistic traffic safety assessment, the primary one is the difficulty on reasonably modeling the full dynamic interaction effects of the bridge-traffic system subjected to wind. Chen and Wu (2010) incorporated the stochastic traffic flow simulation into the bridge-traffic interaction analysis based on the equivalent-dynamic-wheel-loading (EDWL) concept (Chen and Cai 2007), which focused on the bridge response and cannot provide accurate estimation of the dynamic response of individual vehicles of the simulated stochastic traffic. As a result, hurdles still remained on assessing vehicles safety on long-span bridges when stochastic traffic is simulated until recently when some advances on modeling techniques were made.

The new mode-based and finite element-based dynamic analysis frameworks of bridge-traffic system are developed in Chapter 2 (Zhou and Chen 2015a) and Chapter 5 (Zhou and Chen 2015c), respectively, by considering the full-coupling effects of all the vehicles, the bridge and the wind simultaneously. As a result, the dynamic response of each individual vehicle of the stochastic traffic can be accurately obtained for the first time, which opens a door to the advanced traffic safety assessment of stochastic traffic. Built based on these advances, the chapter reports the efforts on developing an integrated dynamic interaction and safety assessment model of the fully coupled bridge-traffic system. Developed within the finite-element-based simulation framework (Zhou and Chen 2015c), the dynamic bridge-traffic interaction analysis is conducted to obtain the vehicle response considering road roughness and wind excitations, followed by vehicle safety assessment of different accident types in the windy environment. A prototype long-span cable-stayed bridge and the simulated stochastic traffic are studied as a demonstration of the proposed approach.

7.2 Mathematical modeling of the vehicles and the bridge

7.2.1 Modeling of the long-span cable-supported bridge

A long-span cable-stayed bridge is modeled in this study as a three-dimensional finite element model using two types of finite elements. The bridge girder and pylon are modeled with nonlinear spatial beam element based on Timoshenko beam theory. The axial, bending, torsional warping and shear deformation effects are considered at the same time. The stay cables are modeled with catenary cable elements. They are derived based on the exact analytical expression of differential equations for elastic catenary elements. The geometric nonlinear effect of axial forces on the bridge girder and pylons and the cable tension can be taken into account. The effects of flexibility and large deflection in the cables are also considered in establishing the equilibrium equations of the element. Rayleigh damping is assumed to model the structural damping of the bridge, in which the participating factors for the stiffness and mass matrices are obtained from two structural damping ratios associated with two specific modes.

7.2.2 Modeling of the road vehicles

Three types of vehicles are involved in the present study, which are high-sided heavy trucks, light trucks with medium height and light sedan cars. Each type of vehicle is modeled as several rigid bodies and wheel axles connected by series of springs, dampers and pivots. The suspension system and the elastic tires are modeled as springs in the upper and lower positions, respectively. The energy dissipation is achieved by modeling upper and lower viscous dampers for the suspension system. The masses of the suspension system and the tires are assumed to be concentrated on the mass blocks at each side of the vehicle and the masses of the springs and dampers are assumed to be zero. Each main rigid body contains four degrees of freedom, including two translational and two rotational ones. The numerical dynamic model of the heavy truck is composed of two main rigid bodies, three wheel axle sets, twenty-four sets of springs and dampers vertically and laterally. The numerical dynamic model for the light truck and light cars consists of one main rigid body, two wheel axle sets, sixteen sets of springs and dampers. The

dynamic models of the light trucks and cars are similar to those of the heavy trucks except with only one rigid body.

7.3 Modeling of the dynamic excitations

7.3.1 Modeling of road surface roughness

The road surface roughness is an important source of the coupling effects for a bridge-traffic system. The roughness on the approaching road and the bridge deck is modeled as a stationary Gaussian random process with zero mean value. The power spectral density function suggested by Huang and Wang (1992) is adopted in the present study, shown in Eq. (7.1).

$$S(\overline{\phi}) = A_r \left(\frac{\overline{\phi}}{\overline{\phi}_0}\right)^{-2}$$
(7.1)

in which, A_r is the road roughness coefficient (m³/cycle) representing the road roughness condition; $\overline{\phi}$ is the wave number (cycle/m); $\overline{\phi}$ is the discontinuity frequency (0.5 π cycle/m). The road surface roughness r(x) can be generated by the inverse Fourier Transformation as shown in Eq. (7.2).

$$r(x) = \sum_{i=1}^{n} \sqrt{2S(\overline{\phi}_k)\Delta\overline{\phi}} \cos(2\pi\overline{\phi}_k x + \theta_k)$$
(7.2)

in which, *n* is the number of points in the inverse Fourier Transform; *x* is the location on the road surface; θ_k is the random phase angle with a uniform distribution between 0 and 2π . The dynamic forces between the *i*th vehicle and the contacting element on the bridge due to the road surface roughness can be expressed in the following equation:

$$F_{r}^{i}(t) = \sum_{j=1}^{n_{a}} (K_{lzL}^{j} r(x_{j}) + C_{lzL}^{j} \dot{r}(x_{j}) V_{i} + K_{lzR}^{j} r(x_{j}) + C_{lzR}^{j} \dot{r}(x_{j}) V_{i})$$
(7.3)

in which, $r(x_j)$ is the road surface roughness at the j^{th} axle; $\dot{r}(x_j)$ is the first order derivative of $r(x_j)$ with respect to time; V_i is the driving speed of the i^{th} vehicle; n_a is the number of wheel axles.

7.3.2 Modeling of wind forces on bridge

The total wind forces acting on the bridge are commonly divided into three components: steady-state forces resulting from the average wind speed component, self-excited forces resulting from the bridgewind interactions and buffeting forces resulting from the unsteady wind velocity component.

7.3.2.1 <u>Steady-state wind forces on bridge</u>

The steady-state wind forces acting on the bridge deck are divided into three directions, which are drag force, lift force and twist moment, as defined in Eqs. (7.4a-c), respectively.

$$D = \frac{1}{2}\rho U_w^2 C_d(\alpha) H \tag{7.4a}$$

$$L = \frac{1}{2}\rho U_w^2 C_l(\alpha) H \tag{7.4b}$$

$$M = \frac{1}{2} \rho U_w^2 C_m(\alpha) B^2 \tag{7.4c}$$

in which, ρ is the mass density of the air; U_w is the steady-state wind speed; *H* is the depth of the bridge girder; *B* is the width of the bridge girder; α is the effective wind attack angle, which is obtained by summing the initial wind attack angle and the rotational displacement of the bridge deck; $C_d(\alpha)$, $C_1(\alpha)$ and $C_m(\alpha)$ are the static wind coefficients for the bridge girder, respectively, which change with the effective wind attack angle.

7.3.2.2 <u>Self-excited wind forces on bridge</u>

The components of the self-excited forces can be obtained by summing the response associated with the structural motion in lateral, vertical and torsional directions. Particularly, the self-excited drag force, lift force and twist moment on a unit span of bridge girder can be expressed as the convolution integral between the time-dependent bridge girder motion and the impulse functions associated with the motion, as shown in Eq. (7.5) (Lin and Yang, 1983).

$$D_{se}(t) = D_{sep}(t) + D_{seh}(t) + D_{sea}(t) = \int_{-\infty}^{t} f_{Dp}(t-\tau) p(\tau) d\tau + \int_{-\infty}^{t} f_{Dh}(t-\tau) h(\tau) d\tau + \int_{-\infty}^{t} f_{D\alpha}(t-\tau) \alpha(\tau) d\tau$$
(7.5a)

$$L_{se}(t) = L_{sep}(t) + L_{seh}(t) + L_{sea}(t) = \int_{-\infty}^{t} f_{Lp}(t-\tau)p(\tau)d\tau + \int_{-\infty}^{t} f_{Lh}(t-\tau)h(\tau)d\tau + \int_{-\infty}^{t} f_{L\alpha}(t-\tau)\alpha(\tau)d\tau$$
(7.5b)

$$M_{se}(t) = M_{sep}(t) + M_{seh}(t) + M_{sea}(t) = \int_{-\infty}^{t} f_{Mp}(t-\tau)p(\tau)d\tau + \int_{-\infty}^{t} f_{Mh}(t-\tau)h(\tau)d\tau + \int_{-\infty}^{t} f_{Ma}(t-\tau)\alpha(\tau)d\tau \quad (7.5c)$$

in which, p(t), h(t) and $\alpha(t)$ are time-dependent lateral, vertical and rotational displacement responses of the bridge girder, respectively; $f_{Dp}(t)$, $f_{Dh}(t)$, $f_{D\alpha}(t)$, $f_{Lp}(t)$, $f_{Lh}(t)$, $f_{L\alpha}(t)$, $f_{Mp}(t)$, $f_{Mh}(t)$ and $f_{M\alpha}(t)$ are the impulse response functions of the self-excited wind force components; The first subscripts "D", "L" and "M" define the impulse response functions corresponding to the self-excited drag, lift and twist forces, respectively; The second subscripts "p", "h" and " α " define the impulse response function with respect to lateral, vertical and torsional unit impulse displacement, respectively.

7.3.2.3 <u>Buffeting forces on bridge</u>

Buffeting forces on the bridge girder are induced by the unsteady components of the oncoming wind, including the time-variant horizontal and vertical parts. The buffeting forces on a unit span of bridge girder can be expressed as the convolution integral between the wind fluctuations and the associated impulse response functions in both horizontal and vertical directions using the similar way to that formulating the self-excited forces. The drag force, lift force and twist moment of buffeting forces can be expressed as:

$$D_{b}(t) = D_{bu}(t) + D_{bw}(t) = \int_{-\infty}^{t} f_{Du}(t-\tau)u(\tau)d\tau + \int_{-\infty}^{t} f_{Dw}(t-\tau)w(\tau)d\tau$$
(7.6a)

$$L_{b}(t) = L_{bu}(t) + L_{bw}(t) = \int_{-\infty}^{t} f_{Lu}(t-\tau)u(\tau)d\tau + \int_{-\infty}^{t} f_{Lw}(t-\tau)w(\tau)d\tau$$
(7.6b)

$$M_{b}(t) = M_{bu}(t) + M_{bw}(t) = \int_{-\infty}^{t} f_{Mu}(t-\tau)u(\tau)d\tau + \int_{-\infty}^{t} f_{Mw}(t-\tau)w(\tau)d\tau$$
(7.6c)

in which, u(t) and w(t) are the turbulent wind velocities in the horizontal and vertical directions, respectively; $f_{Du}(t)$, $f_{Dw}(t)$, $f_{Lu}(t)$, $f_{Lw}(t)$, $f_{Mu}(t)$ and $f_{Mw}(t)$ are the impulse response functions; the subscripts "D", "L" and "M" denote the impulse response functions corresponding to drag force, lift force and twist moment, respectively; the subscripts "u" and "w" define the impulse response functions with respect to turbulent wind velocities in the horizontal and vertical directions, respectively.

7.3.3 Modeling of wind forces on vehicles

The total wind forces acting on a road vehicle are determined using the quasi-static approach (Coleman and Baker 1990). The instantaneous wind speed on the vehicle is obtained as the summation of mean wind speed U_w and unsteady horizontal wind speed u(x, t) at location x and time t. The road vehicles on the road or bridge have varying driving speeds following some traffic rules and the instantaneous vehicle driving speed is defined as $U_{ve}(t)$ at time t. The wind velocity relative to the vehicle $U_{re}(t)$ at time t is obtained from the vector subtraction of $U_{ve}(t)$ from $(U_w + u(x, t))$, as shown in Fig. 7.1.



Figure 7.1 Demonstration of the relative wind speed

The wind speed relative to the vehicle $U_{re}(t)$ can be obtained in Eq. (7.7a). φ is the wind attack angle, which is the angle between the wind and vehicle driving directions. ψ is the yaw angle, which is the angle between the direction of the relative wind speed and the vehicle driving direction in the range from 0 to π , as defined in Eq. (7.7b).

$$U_{R}(t) = \sqrt{(U_{w} + u(x,t))^{2} + U_{ve}^{2}(t)}$$
(7.7a)

$$\psi = \arctan\left[(U_w + u(x,t))/U_{ve}(t)\right]$$
(7.7b)

The total wind forces and moments on the vehicles are given in the following equations:

$$F_{vx}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{D}(\Psi)A \qquad (7.8a) \qquad F_{vy}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{S}(\Psi)A \qquad (7.8b) \qquad F_{vz}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{L}(\Psi)A$$

(7.8c)

$$M_{vx}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{R}(\Psi)Ah_{v} \quad (7.8d) \qquad M_{vy}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{P}(\Psi)Ah_{v} \quad (7.8e) \qquad M_{vz}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{Y}(\Psi)Ah_{v}$$

(7.8f)

in which, F_{w}^{w} , F_{yy}^{w} and F_{vz}^{w} are the total drag force, side force and lift force acting at the center of gravity in the positive x, y and z direction, respectively; M_{wx}^{w} , M_{yy}^{w} and M_{wz}^{w} are the total rolling moment, pitching moment and yawing moment about the x, y and z directions, respectively; $C_{D}(\Psi)$, $C_{S}(\Psi)$, $C_{L}(\Psi)$, $C_{g}(\Psi)$, $C_{p}(\Psi)$ and $C_{y}(\Psi)$ are the coefficients of drag force, side force, lift force, rolling moment, pitching moment and yawing moment, respectively; A is the reference area, which is usually taken as the frontal area of the vehicle; h is the reference height, which is usually taken as the distance between the gravity of the vehicle and the ground. The direction of each wind force component on the vehicle is demonstrated in Fig. 7.2. The positive driving direction is in the positive x direction and wind is assumed to blow in the positive y direction.



Figure 7.2 Demonstration of the direction of each wind force component on the vehicle

The total wind forces on the vehicle consist of the effects from both mean wind speed and unsteady wind speed, and therefore include the combined static and dynamic effects. In the coupled bridge-traffic interaction analysis, only the unsteady part of the wind forces contributes to the dynamic coupled analysis between the bridge and vehicles. In this study, the total wind forces on vehicles are partitioned into two parts, which are the static wind force and the aerodynamic wind force. The static wind forces in each direction only consider the static wind effects from the mean wind speed U_w and exclude the contribution of the unsteady wind speed u(x, t) and vehicle speed $U_{ve}(t)$ that change with respect to time. The static

wind forces are used in the nonlinear static analysis to identify the deformed position for the following nonlinear dynamic analysis. The static wind forces and moments in each direction are expressed as follows:

$$F_{vx}^{s} = \frac{1}{2}\rho U_{m}^{2}C_{D}(\Psi')A \quad (7.9a) \qquad F_{vy}^{s} = \frac{1}{2}\rho U_{m}^{2}C_{S}(\Psi')A \quad (7.9b) \qquad F_{vz}^{s} = \frac{1}{2}\rho U_{m}^{2}C_{L}(\Psi')A \quad (7.9c)$$

$$M_{vx}^{s} = \frac{1}{2}\rho U_{m}^{2}C_{R}(\Psi')Ah_{v} \quad (7.9d) \qquad M_{vy}^{s} = \frac{1}{2}\rho U_{m}^{2}C_{P}(\Psi')Ah_{v} \quad (7.9e) \qquad M_{vz}^{s} = \frac{1}{2}\rho U_{m}^{2}C_{Y}(\Psi')Ah_{v} \quad (7.9f)$$

in which, Ψ' is the angle between the wind speed and the longitudinal direction of the bridge. The static wind force on a vehicle does not consider the effect of vehicle motion. The aerodynamic wind force on a vehicle in each direction is obtained by subtracting static wind force from the total wind force, as shown in Eq. (7.10).

$$F_{vx}^{d} = F_{vx}^{t} - F_{vx}^{s} \quad (7.10a) \qquad F_{vy}^{d} = F_{vy}^{t} - F_{vy}^{s} \quad (7.10b) \qquad F_{vz}^{d} = F_{vz}^{t} - F_{vz}^{s} \quad (7.10c) \qquad M_{vz}^{d} = M_{vy}^{t} - M_{vy}^{s} \quad (7.10e) \qquad M_{vz}^{d} = M_{vz}^{t} - M_{vz}^{s}$$

(7.10f)

The drag force can hardly cause an accident on the vehicle because the driver usually can adjust the traction to keep the driving speed and resist the drag force. Since the vehicle model doesn't have the corresponding degree of freedom for the yawing moment, the equivalent lateral forces on the front and rear wheels are obtained based on the force equilibrium conditions in order to resist the yawing moment.

7.4 Dynamic bridge-traffic-wind interaction analysis

7.4.1 Simulation of stochastic traffic flow

The stochastic traffic flow in this study is simulated using the cellular automaton (CA) model to rationally simulate the moving traffic on the long-span bridge. The discrete variables of an individual vehicle in the two-lane traffic flow simulated in the present study include the vehicle-occupied lane, vehicle longitudinal location, vehicle type, vehicle speed, vehicle driving direction. The variables in each cellular are updated based on the vehicle information in the adjacent locations and the probabilistic traffic
rules regulating the accelerating, decelerating, lane changing and braking operations. Detailed traffic rules involved in the traffic flow simulation are referred to the published papers (Chen and Wu 2010; Chen and Wu 2011). The CA-based traffic flow simulation is performed on a roadway-bridge-roadway system to simulate the stochastic traffic flow through the bridge in a realistic way. The periodic boundary conditions are adopted in the simulation of stochastic traffic flow such that the number of each type of vehicles in the roadway-bridge-roadway system remains the same.

7.4.2 Equations of motion for the bridge-traffic system

The fully-coupled bridge-traffic interaction system is established with the bridge and vehicles as two subsystems. The bridge subsystem is built based on the finite element formulation of the bridge and the vehicle subsystem is built based on the numerical dynamic model of each vehicle. The interaction forces between the bridge and vehicles are formulated in terms of the coupling matrices and motion vectors of the bridge and each vehicle on the bridge. It is assumed that the two approaching roadways are completely rigid for both driving directions, and therefore the vehicle vibrates on its own without the interaction between the vehicle and supporting road surface when the vehicle is on the approaching roads. The coupled motion equations of the bridge-traffic system can be constructed as in Eq. (7.11).

$$\begin{bmatrix} M_{b} & 0 & \cdots & 0 \\ 0 & M_{v_{1}} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & M_{v_{n}} \end{bmatrix} \begin{bmatrix} \ddot{q}_{b} \\ \ddot{q}_{v_{1}} \\ \vdots \\ \ddot{q}_{v_{n}} \end{bmatrix} + \begin{bmatrix} C_{b} & 0 & \cdots & 0 \\ 0 & C_{v_{1}} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & 0 & C_{v_{n}} \end{bmatrix} \begin{bmatrix} \dot{q}_{b} \\ \dot{q}_{v_{1}} \\ \vdots \\ \dot{q}_{v_{n}} \end{bmatrix} + \begin{bmatrix} K_{b} & 0 & \cdots & 0 \\ 0 & K_{v_{1}} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & K_{v_{n}} \end{bmatrix} \begin{bmatrix} q_{b} \\ q_{v_{1}} \\ \vdots \\ q_{v_{n}} \end{bmatrix}$$

$$= \begin{cases} \sum_{i=1}^{n} F_{v_{i}}^{G} + F_{b}^{R} + F_{b}^{C} + F_{b}^{Se} + F_{b}^{Bu} \\ F_{v_{1}}^{R} + F_{v_{1}}^{C} + F_{v_{1}}^{dw} \\ \vdots \\ F_{v_{n}}^{R} + F_{v_{n}}^{C} + F_{v_{n}}^{dw} \end{cases}$$

$$(7.11a)$$

in which,

$$F_b^C = \sum_{i=1}^n C_{bci}\{\dot{q}_b\} + \sum_{i=1}^n K_{bci}\{q_b\} + \sum_{i=1}^n C_{b,v_i}\{\dot{q}_{v_i}\} + \sum_{i=1}^n K_{b,v_i}\{q_{v_i}\}$$
(7.11b)

$$F_{v_i}^{\ C} = C_{v_i,b} \{ \dot{q}_{v_i} \} + K_{v_i,b} \{ q_{v_i} \}$$
(7.11c)

 q_{v_i} and q_{v_i} are the displacement vectors of the bridge and the i^{th} vehicle, respectively; one-dot and two-dot superscripts of the displacement vector indicate the corresponding velocity and acceleration, respectively; *n* is the total number of vehicles involved in the traffic flow; $M_{b(v_i)}$, $K_{b(v_i)}$ and $C_{b(v_i)}$ are the structural mass, stiffness and damping matrices for the bridge subsystem (i^{th} vehicle); K_{bci} and C_{bci} refer to the stiffness and damping contributions to the bridge structure due to the coupling effects between the *i*th vehicle in the traffic flow and the bridge system, respectively; K_{b,v_i} and C_{b,v_i} are the coupled stiffness and damping matrices contributing to the bridge vibration from the i^{th} vehicle in the traffic flow, respectively; $K_{v_i,b}$ and $C_{v_i,b}$ are the coupled stiffness and damping matrices contributing to the vibration of the i^{th} vehicle in the traffic flow from the bridge structure; F_b and F_{v_i} denote the external force vector due to the applied loads for the bridge and the i^{th} vehicle, respectively; the superscripts R and C indicate the interaction forces due to road surface roughness and coupling matrices on the bridge or each vehicle, respectively; the superscripts G, Se and Bu represent for the external loads from vehicle gravity, self-excited and buffeting forces on the bridge, respectively; superscript dw indicates the aerodynamic wind force on vehicles. The coupling matrices between the bridge and each vehicle are updated at every time step during the simulation process based on the new contact location of each vehicle traveling on the bridge. The selfexcited forces due to the bridge-structure interactions and coupling interaction forces between the bridge and vehicles are motion-dependent, and thus are unknown at the beginning of each time step. The nonlinear iteration is therefore undertaken starting with an initial motion vector until the prescribed convergence criterion is satisfied at each time step.

7.5 Vehicle accident analysis

7.5.1 The total force approach

The dynamic interaction analysis between the bridge and vehicles assumes that the vehicle wheels and the bridge deck surface have point contact at all times and the wheels will not be separated from the bridge during the simulation. Based on the nonlinear finite element model, the total contact forces in the vertical and lateral directions can be obtained for the vehicles throughout the analysis. The total contact forces in the vertical and lateral directions for each wheel at one side at time *t* are noted as $V_{iL(R)}^{t}(t)$ and $L_{iL(R)}^{t}(t)$ (*i* = 1, 2,..., *n_a*), respectively. These total contact forces are obtained through following steps:

Step 1: Nonlinear static analysis is conducted for the bridge under bridge gravity, initial pretension forces of the cables and static wind forces on the bridge. This step generates the initial deformed position of the bridge for following nonlinear dynamic analysis.

Step 2: Static analysis is conducted for the vehicles under vehicle gravity and static wind forces on vehicles. The initial deformed position of each vehicle on the bridge will be generated and the static vertical and lateral contact forces $V_{il(R)}^s$ and $L_{fl(R)}^s$ are obtained as the summation of the forces of the lower spring and damper on the left (right) side of the *i*th wheel set, respectively, as shown in Eq. (7.12).

$$V_{iL(R)}^{s} = K_{lzL(R)}^{i} h_{L(R)}^{i} + C_{lzL(R)}^{i} h_{L(R)}^{i}$$
(7.12a)

$$L_{iL(R)}^{s} = K_{lyL(R)}^{i} y_{L(R)}^{i} + C_{lyL(R)}^{i} \dot{y}_{L(R)}^{j}$$
(7.12b)

in which, $K_{lzL(R)}^{i}$ and $C_{lzL(R)}^{i}$ are the stiffness and damping coefficients of the lower vertical spring and damper of the *i*th wheel set on the left (right) side; $K_{lyL(R)}^{i}$ and $C_{lyL(R)}^{i}$ are the stiffness and damping coefficients of the lower lateral spring and damper of the *i*th wheel set on the left (right) side; $h_{L(R)}^{i}$ and $\dot{h}_{L(R)}^{i}$ are the vertical displacement and velocity of the mass block of the *i*th wheel set on the left (right) side, respectively; $y_{L(R)}^{i}$ and $\dot{y}_{L(R)}^{i}$ are the lateral displacement and velocity of the mass block of the *i*th suspension and wheel set on the left (right) side, respectively.

Step 3: Starting from the deformed position of the bridge and vehicles, nonlinear dynamic analysis is conducted for the bridge-traffic system, in which the aerodynamic forces on the bridge, vehicles and the interaction forces between the bridge and vehicles are applied.

In this step, the displacement and velocity of the mass block relative to the bridge at each contact point need to be obtained in order to calculate the forces in the lower spring and damper of the wheel set. Considering that the dynamic displacement and velocity of the vehicle and the bridge are unknown at first in each time step, nonlinear iterations are conducted starting from the initial deformed position. The dynamic total contact force $V_{il(R)}^d$ and $L_{il(R)}^d$ in the vertical and lateral directions for each time step are expressed in Eq. (7.13).

$$V_{iL(R)}^{s}(t) = K_{lzL(R)}^{i} \overline{h}_{L(R)}^{i}(t) + C_{lzL(R)}^{i} \overline{h}_{L(R)}^{i}(t)$$
(7.13a)

$$L_{iL(R)}^{s}(t) = K_{lyL(R)}^{i} \overline{y}_{L(R)}^{i}(t) + C_{lyL(R)}^{i} \overline{\dot{y}}_{L(R)}^{i}(t)$$
(7.13b)

in which, $\bar{h}_{L(R)}^{i}(t)$ and $\bar{h}_{L(R)}^{i}(t)$ are the vertical displacement and velocity of the mass block relative to the bridge contact point for the *i*th wheel set on the left (right) side at time *t*, respectively; $\bar{y}_{L(R)}^{i}(t)$ and $\bar{y}_{L(R)}^{i}(t)$ are the lateral displacement and velocity of the mass block relative to the bridge contact point for the *i*th suspension and wheel set on the left (right) side at time *t*, respectively.

Step 4: The total vertical $(V_{iL(R)}^{t}(t))$ and lateral $(L_{iL(R)}^{t}(t))$ contact force time histories are obtained as the summation of the static and dynamic vertical and lateral contact forces at each time instant, respectively, as shown in Eq. (7.14).

$$V_{iL(R)}^{t}(t) = V_{iL(R)}^{s} + V_{iL(R)}^{d}(t)$$
(7.14a)

$$L_{iL(R)}^{t}(t) = L_{iL(R)}^{s} + L_{iL(R)}^{d}(t)$$
(7.14b)

7.5.2 *Characterization of vehicle accident type*

After the time histories of vertical and lateral contact forces at each wheel set are obtained, the vehicle accident analysis is carried out based on the vehicle mechanics theory. The proposed total force approach is able to identify the three vehicle accident types: lift-up, side-slip and yawing.

7.5.2.1 <u>Type I: Lift-up at the wheel</u>

Lift-up accident is defined as the situation when at least one side of a certain wheel set is lifted uplosing contact with the ground. It is known that the lift-up of wheel may or may not eventually lead to overturning accidents. However, given the complexness and uncertainties of the time-transient overturning process (Chen and Chen 2010), lift-up accident criterion can be used as an ideal and convenient evaluation tool to assess the associated risks conservatively. Based on the coordinates defined in this study, lift-up accidents are considered to happen once the total vertical contact force on the ground at one side of one or all wheel sets at certain time instants are over zero. Lift-up accidents for a high-sided heavy truck may include three situations, such as lift-up occurring at one wheel set, two adjacent wheel sets and all wheel sets. For a 3-axle heavy truck, the accident criterion is shown below.

1) Lift-up at i^{th} wheel axle: $V_{iL(R)}^{t}(t) > 0$, i = 1, 2 or 3

- 2) Lift-up at two adjacent wheel axles: $V_{iL(R)}^{t}(t) > 0$, i = 1 and 2; 2 and 3
- 3) Lift-up at all wheel axles: $V_{iL(R)}^{t}(t) > 0$, i = 1, 2 and 3

When the lift-up accident happens to one wheel set, two wheel sets or all the wheel sets, the vehicle may start to rotate about the contact points of the wheels remaining on the ground. The lift-up process may eventually trigger overturning accidents, and therefore should be prevented for the safety of the driver as well as the vehicle. Existing research has shown that lift-up accidents are the most common type for high-sided trucks in the windy environment (e.g. Chen and Cai 2004; Chen and Chen 2010).

7.5.2.2 Type II: Side-slip at the wheel

A side-slip accident is defined as the situation when a certain wheel remains contact on the ground whereas the lateral friction cannot prevent the wheels from sliding laterally. When the left and right wheels of a certain wheel set both remain contact on the ground, the side-slip accident will happen if the total lateral contact force of the left and right wheel exceeds the total static friction force. The static friction force is the multiplication of static friction coefficient μ and the vertical contact force. Similar to

overturning accidents, side-slip accidents for a high-sided heavy truck also have three possible situations: side-slip at one wheel set, at two adjacent wheel sets and at all wheel sets. The corresponding accident criteria for heavy trucks are shown as follows:

1) Side-slip at i^{th} wheel axle: $|L_{il}^{t}(t) + L_{iR}^{t}(t)| > \mu |V_{il}^{t}(t) + V_{iR}^{t}(t)|$, i = 1, 2 or 3

2) Side-slip at two adjacent wheel axles: $|L_{il}^{t}(t) + L_{iR}^{t}(t)| > \mu |V_{il}^{t}(t) + V_{iR}^{t}(t)|$, i = 1 and 2; or i = 2 and 3

3) Side-slip at all wheel axles: $|L_{iL}^{t}(t) + L_{iR}^{t}(t)| > \mu |V_{iL}^{t}(t) + V_{iR}^{t}(t)|$, i = 1, 2 and 3

When the side-slip accident happens along with lift-up accidents at certain wheel sets, the vertical contact force at the lifted side of the wheel set should be set to zero for the above criteria. The criteria for the car and light truck can be easily derived from above based on the actual axle number.

7.5.2.3 <u>Type III: Yawing accident (rotational movement)</u>

A yawing accident is defined as the situation when the vehicle has rotational movements around the vertical axis. It could happen when all or only some wheels experience side-slipping. Particularly, the yawing accident for a heavy truck can happen in terms of rotating between 1^{st} and 2^{nd} wheel sets or between 2^{nd} and 3^{rd} wheel sets. The accident criterion for each situation of a heavy truck is shown as follows.

1) Yawing around vertical axis between i^{th} and j^{th} wheel sets when side-slip occurs to all wheels:

sign $(L'_{il}(t) + L'_{iR}(t)) \neq sign (L'_{jL}(t) + L'_{jR}(t))$, i = 1, j = 2 or i = 2, j = 3,

and

 $|L_{iL}^{t}(t) + L_{iR}^{t}(t)| > \mu |V_{iL}^{t}(t) + V_{iR}^{t}(t)|, i = 1, 2 \text{ and } 3,$

2) Yawing around vertical axis between i^{th} and j^{th} wheel sets when side-slip doesn't happen to all:

 $|L_{iL}^{t}(t) + L_{iR}^{t}(t)| > \mu |V_{iL}^{t}(t) + V_{iR}^{t}(t)|, i = 1, 2 \text{ or } 3$

and

 $|L_{il}^{t}(t) + L_{iR}^{t}(t)| \le \mu |V_{il}^{t}(t) + V_{iR}^{t}(t)|, j = 1, 2 \text{ or } 3 \text{ and } j \ne i$

3) Yawing around vertical axis at the pivot connecting two rigid bodies:

 $|L_{il}^{'}(t) + L_{iR}^{'}(t)| \leq \mu |V_{il}^{'}(t) + V_{iR}^{'}(t)|, i = 1, 2; L_{3L}^{'}(t) + L_{3R}^{'}(t) > \mu(V_{3L}^{'}(t) + V_{3R}^{'}(t)),$

Or,

$$|L_{iL}^{t}(t) + L_{iR}^{t}(t)| > \mu |V_{iL}^{t}(t) + V_{iR}^{t}(t)|, i = 1, 2; L_{3L}^{t}(t) + L_{3R}^{t}(t) \le \mu (V_{3L}^{t}(t) + V_{3R}^{t}(t))$$

Yawing accidents for light trucks and sedan cars only have the first two types of situations as illustrated above.

7.6 Numerical demonstration

7.6.1 Prototype bridge and vehicle models

The prototype cable-stayed bridge in the present study has a total length of 836.7 m, with a main span, two side spans and two approach spans. The cable-stayed bridge has a bridge deck with a steel twin-box cross-section, which has a width of 28 m and a height of 4.57 m. The two steel pylons have A-shaped cross-sections with a height of 103.6 m. The bridge superstructure is supported by the cross beam of bridge pylons at the pylon locations and the reinforced concrete bridge piers at the side spans with longitudinally sliding bearing supports. The 3-D sketch and cross section of the bridge girder are shown in Fig. 5.3a and Fig. 5.3b, respectively.

The travelling vehicles in this study are classified into three types: heavy truck, light truck and light sedan car. The dynamic parameters for each type of vehicles involved in the present study, including mass, mass moment of inertia, stiffness coefficients and damping coefficients, are listed in Table 5.3. The dimension parameters for each type of vehicles are listed in Table 5.4.

7.6.2 Simulation of stochastic traffic flow

The total length of the roadway-bridge-roadway path is 1,260 m, including two roadways with a length of 210 m each and the bridge with a length of 840 m. The speed limit is adopted as 30 m/s. The stochastic traffic flow with a moderate density of 16 vehicles/km/lane is simulated with the CA-based traffic flow simulation model as introduced previously (Chen and Wu 2011). The percentages of the three types of vehicles in the traffic flow are chosen as 20%, 30% and 50% for heavy trucks, light trucks and light sedan cars, respectively. The simulated stochastic traffic flow is comprised of a total of 80 vehicles in the roadway-bridge-roadway system, including 16 heavy trucks, 24 light trucks and 40 light cars.

7.6.3 Simulation of wind forces on bridge and vehicles

The turbulent wind speeds are simulated as multi-variate stationary random Gaussian process with a mean value of zero in the vertical and lateral directions. In this study, wind speed is assumed to be perpendicular to the longitudinal axis of the bridge deck and the effects of other skewed wind directions are not considered. For the vehicle moving in the wind field, the following formulae suggested by Baker (Coleman and Baker 1990) are used to determine the total wind force and moment coefficients as functions of the yaw angle.

$$C_D(\psi) = -0.5(1 + 2\sin 3\psi)$$
 (7.15a), $C_S(\psi) = 5.2\psi^{0.382}$ (7.15b), $C_L(\psi) = 0.93(1 + \sin 3\psi)$ (7.15c)

$$C_{R}(\psi) = 7.3\psi^{0.294}$$
 (7.15d), $C_{P}(\psi) = 2.0\psi^{1.32}$ (7.15e), $C_{Y}(\psi) = 2.0\psi^{1.77}$ (7.15f)

7.6.4 Baseline scenario for vehicle accident analysis

In this section, the vehicle response will be obtained from bridge-traffic interaction analysis and the representative vehicle for each type will be selected for accident analysis using the total force approach. The steady-state wind speed is 20 m/s and moderate traffic flow is moving through the road-bridge-road system. Compared with the high-sided heavy truck and light truck, light sedan car is usually safe from crosswind due to its small frontal area and height. Therefore, only a representative high-sided heavy truck and a light truck are selected for the following vehicle accident investigation. The traveling paths of the two representative vehicles, e.g., the first vehicle of each type in the traffic flow, are presented in Fig. 7.3.



Figure 7.3 Instantaneous vehicle location for representative vehicles

The initial locations for the representative heavy truck and light truck are on the bridge and on the road, respectively. It is seen in Fig. 7.3 that the driving speeds of vehicles may vary with time following traffic rules. However for traffic flow with a moderate number of vehicles, the vehicles can generally keep a stable driving speed. A horizontal line means that the vehicle is not in the motion. A steeper line section indicates that the vehicle is accelerating while a flatter line section indicates that the vehicle is decelerating from the driving speed in the previous time instant.

The total wind forces acting on the vehicles change with respect to time due to the unsteady characteristics of instantaneous turbulent and vehicle speeds. The four components of total wind force histories during the simulation period on the major (2^{nd}) rigid body of the representative heavy truck are given in Fig. 7.4a-d.



(b) Lift force



(d) Yawing moment

Figure 7.4 Total wind force on the major rigid body of the heavy truck

It is seen in Fig. 7.4 that the wind forces on the vehicle bodies are non-stationary as they change with respect to both turbulent wind speed and vehicle driving speed. As the vehicle speed changes, the total wind forces acting on the vehicles change significantly. Compared with the other four wind forces, lift and drag forces are more sensitive to the vehicle driving speed. As the vehicle speed drops, both the lift and drag forces drop significantly. When the vehicle speed drops to zero, the cross wind is applied on the vehicle with a yaw angle of 90° and wind is perpendicular to the longitudinal axis of the driving direction. The time histories of vertical displacement and lateral displacement at the windward side of the 1st wheel set of the heavy truck are demonstrated in Figs. 7.5a and b, respectively. By Referring to the vehicle location in Fig. 7.3, it is seen that the vehicle enters the bridge at around 7.5 s and leaves the bridge at around 41.0 s. The vehicle has the largest dynamic response when the vehicle arrives at around the mid-

span of the bridge. At the start time when wind is firstly applied, the vehicle has reached the local extreme dynamic responses for both vertical and lateral directions.



(b) Lateral displacement

Figure 7.5 Dynamic response at the windward side of the 1st wheel set for the heavy truck 7.6.4.1 <u>Lift-up accident</u>

Being lifted up at certain wheel sets is the most common cause of road accident for moving vehicles in windy environment. The vertical contact force time histories at the windward and leeward side of the 3^{rd} wheel set are given for the heavy truck in Figs. 7.6a and b, respectively.





Figure 7.6 Vertical contact force at windward and leeward side of the 3rd wheel set for the heavy truck

It is seen in Fig. 7.6 that the side of the wheel in the windward position of the vehicle is more likely to be lifted up in the windy environment. The other side of the vehicle usually remains on the ground unless the vehicle is driven at a very high speed while the wind speed is also in the very high range. Therefore, only in very rare cases, both sides of the vehicle wheel set will be lifted up at the same time. The vertical contact forces at windward side of the 1^{st} and 2^{nd} wheel sets are shown in Fig. 7.7a and b, respectively.



(a) At the windward side of the 1st wheel set



(b) At the windward side of the 2^{nd} wheel set

Figure 7.7 Vertical contact force at windward side of the 1st and 2nd wheel set for the heavy truck It is seen in Fig. 7.7 that the 1st and 2nd wheel sets at the windward side of the heavy truck remain contact with the ground and will not be lifted up. The results suggest that the windward sides of the 3rd and 2nd wheel sets are the most and least likely ones to be lifted up in the windy environment for highsided heavy truck, respectively. The windward side of the 1st wheel set is less likely to be lifted up as compared to that of the 3rd wheel set while more likely to be lifted up compared with that of the 2nd wheel set.

Different from the heavy truck with two rigid bodies, the vertical contact forces for the two wheel sets of the light truck are relatively close on the same side (i.e. windward or leeward side), of which the vertical contact force at the 2nd wheel set is a little more likely to be lifted up than that of the 1st wheel set. The vertical contact force histories at the windward side of the 1st and 2nd wheel set for the light truck are shown in Figs. 7.8 a and b, respectively.



(a) At the 1st wheel set



(b) At the 2nd wheel set

Figure 7.8 Vertical contact force at windward side of the wheel sets for the light truck

It is found from Figs. 7.7 and 7.8 that the lift-up accident can happen to the high-sided heavy truck at the 3rd wheel set and can also happen to the light truck at both wheel sets when the vehicles are driven normally in the windy environment at a steady-state wind speed of 20 m/s. Comparatively, the light truck is more likely to have lift-up accidents than the heavy truck. It is possibly because of the similar vehicle frontal area but much smaller self-weight for the light truck as compared to the heavy truck. As the vehicle speeds up and down, the vertical contact force at the windward side exhibits notable non-stationary characteristics for both the heavy and light trucks. For a certain wheel set, the vertical contact force can be influenced by the vehicle driving speed, turbulent wind speed, relative location on the bridge/road system and the response on the contact point of the bridge. It is shown in Figs. 7.7 and 7.8 that the vertical contact force is very unfavorable and reaches local extreme value in the upward direction. Depending on the specific configuration of the vehicles, it is noted that the patterns of lift-up accidents may differ among vehicles with different wheel set configurations as well as mass distributions.

7.6.4.2 Side-slip accident

When the lateral contact force of a certain wheel exceeds the maximum static friction force, the certain wheel will slip laterally on the bridge or the road. When one side of a certain wheel loses contact

with the ground vertically, the side-slip accident will be more likely to happen than in the situation when both sides of the wheel set remain contact with the ground. Unlike the vertical contact force, the leeward and windward sides of the wheel set usually have the similar lateral contact forces at each time instant, which are thus assumed to be the same in the present study. The total lateral contact force is obtained by summing the forces in the lower lateral springs and dampers and the equivalent lateral forces due to yawing moment on the vehicle bodies. The total lateral contact force at the 1st, 2nd and 3rd wheel set of the heavy truck is demonstrated in Figs. 7.9a, b and c, respectively.



(c) The 3rd wheel set

Figure 7.9 The lateral contact force of each wheel set for the heavy truck

As shown in Fig. 7.9, the global maximum lateral force occurs either at 0 s when wind forces are applied on the bridge and road suddenly or at around the bridge mid-span. It is found that both the vertical and lateral contact forces have unfavorable values when the vehicle enters into the crosswind environment. When the lateral contact forces at both the windward and leeward wheels cannot resist the maximum static friction force, side-slip accident will occur. The static friction coefficient varies with different types of road conditions. For normal dry road with concrete surface, the static friction coefficient is commonly adopted as 1.0. For wet or snow-covered ground surface, the static friction coefficient may drop to 0.4. For icy ground surface, the static friction coefficient $\mu_{rl}(t)$ of the *i*th wheel set at time *t* is calculated to resist side-slip at the wheel, which is defined in Eq. (7.16).

$$\mu_{ri}(t) = \frac{L_{il}'(t) + L_{iR}'(t)}{|V_{il}'(t) + V_{iR}'(t)|}$$
(7.16)

in which the vertical contact force needs to be set to zero if it is a positive value. The obtained static friction coefficients may have positive and negative values at different time instants, which indicate the friction forces are in the positive and negative directions, respectively. The minimum required static friction coefficients for the 1st, 2nd and 3rd wheel sets of the heavy truck are marked with horizontal lines in Figs. 7.10a, b and c, respectively.



(a) The 1st wheel set



(c) The 3rd wheel set

Figure 7.10 Minimum required static friction coefficient for the wheel sets of the heavy truck

It is shown in Fig. 7.10 that the 3rd wheel set among all the wheels of the heavy truck is most likely to experience side-slip during the analysis. For the 3rd wheel set, the vehicle is safe without slipping laterally on the dry ground surface with a static friction coefficient around 1.0. If the vehicle is driven on slippery road surface covered with water or snow, the static friction coefficient may drop below 0.4 and the 3rd wheel set will have side-slip accidents from time to time. On the road surface with a static friction coefficient less than 0.15, the 3rd wheel set will have lateral slipping frequently. The minimum required static friction coefficients for the 1st and 2nd wheel sets are very low, and therefore normally no side-slip accident will occur to the two wheel sets unless the friction coefficient on the ground surface drops below 0.2. Compared with the 2nd wheel set, the 1st wheel set will experience side-slip frequently on the icy road surface with a static friction coefficient less than 0.15.

The time histories of the minimum required static friction coefficient at the 1st and 2nd wheel set for the light truck are shown in Figs. 7.11a and b, respectively. It can be seen that the light truck will not experience side-slip accidents on dry road surface for both wheel sets. The 1st wheel set is slightly more disadvantageous for side-slip accidents than the 2nd one. On slippery road covered by rain flow or snow with a static friction coefficient less than 0.4, both wheel sets may experience side-slip at certain time instants. On icy road with a static friction coefficient less than 0.15, the side-slip at both wheels will occur frequently. Therefore, it is apparent that the side-slip accident is influenced by the lateral contact force, vertical contact force and ground friction condition. The lateral and vertical contact forces are closely related to the bridge-vehicle coupling interactions when the vehicle is driven on the bridge. By referring to the vertical contact force time histories in Fig. 7.8, it is found that side-slip accidents may or may not be associated with lift-up accidents at the same wheel set.



Figure 7.11 Minimum required static friction coefficient for the wheel sets of the light truck

7.6.4.3 Yawing accident

Yawing accident is defined as the rotational movement of the vehicle around the vertical axis. For the heavy truck, the 1st and 2nd wheel sets will not slip laterally while the 3rd wheel set may experience sideslip at certain time instants if the friction condition on road surface is between 0.20 and 0.50. In this situation, the vehicle body will have a rotation around the vertical axis of the pivot connecting the two rigid bodies. If the static friction coefficient is around 0.15, the 1st and 3rd wheel sets will have side-slip at certain time instants while the 2nd wheel set remains safe laterally. The yawing accident may be considered to happen around the vertical axis at the 2nd wheel set for the first rigid body and at the pivot for the second rigid body.

For the light truck that consists of only one rigid body, yawing accidents may happen around the vertical axis at the 2nd wheel set considering the fact that the 2nd wheel set stands still while the 1st wheel set experiences side-slip at certain time instants on slippery road surface, e.g., at around 3.0 s. At the time instants when both wheel sets slip laterally on certain road surface conditions, the yawing accident may not occur considering that the side-slip of the wheel happens in the same direction. It is indicated that yawing accident is closely related to the side-slip accident at the wheel sets. Since the side-slip accidents of different wheel sets usually happen in the same direction, yawing accident is typically represented by the situation when at least one wheel set remains still laterally on the ground while other wheel sets slip laterally.

7.6.5 Influence of the presence of multiple vehicles

As discussed earlier, existing studies usually investigated the situation when a single vehicle is driven through a bridge at a constant speed. When more realistic traffic scenario is simulated, the presence of multiple vehicles on the bridge at a time becomes very likely. It is thus becomes interesting to evaluate the effects of adopting more realistic multi-vehicle scenarios on traffic safety versus that under singlevehicle assumption. To achieve this, this study firstly conducted two comparative case studies without considering wind excitations: the case of traffic flow with multiple vehicles and the case considering only one single vehicle. In the traffic flow case, the simulated moderate traffic flow passes the road-bridgeroad system and the response of the representative heavy truck is investigated. In the single-vehicle case, the same heavy truck is assumed to move through the road-bridge-road system with exactly the same instantaneous driving speed at any time instant as those in the traffic flow case of the stochastic traffic flow. Particularly, the time histories of the vertical and lateral contact forces at the 3rd wheel of the representative heavy truck are shown in Figs. 7.12a and b, respectively.



(b) Lateral contact force

Figure 7.12 Vertical and lateral contact force for the two comparative cases (No wind)

It is found in Fig. 7.12a that the vertical contact forces of the vehicle are exactly the same for the two comparative cases when the vehicle is driven on the first approaching road section before entering the bridge. The differences of the vertical contact force between the two cases gradually disappear after the vehicle leaves the bridge and moves on the second approaching road section. When wind excitations are not applied, the vertical contact forces in the single-vehicle case do not have significant variation between

the scenarios when the vehicle is driven on the bridge or on the road. However in the traffic flow case, the variation in the vertical contact force of the heavy truck is much larger when the vehicle moves on the bridge than on road, especially when the vehicle moves at around bridge mid-span.

As shown in Fig. 7.12b, the lateral contact forces of the vehicle in both cases are zero when the vehicle is driven on the first approaching road section before entering the bridge. After the vehicle enters the bridge, the lateral contact force starts to oscillate around its mean value and the variation amplitude gradually increases as the vehicle moves towards the mid-span of the bridge in the single-vehicle case. Such a phenomenon is observed differently for the traffic flow case, likely due to the fact that the lateral contact force of the vehicle being studied is considerably influenced by the vibration of other vehicles. In both cases, the lateral contact forces gradually damp out when the vehicle leaves the bridge and moves on the second approaching road section. When the vehicle is driven on the bridge, the lateral contact forces in the traffic flow case are significantly larger than those in the single-vehicle case. It is indicated that the presence of other multiple vehicles may increase the dynamic response of the vehicle in investigation in both vertical and lateral directions.

As a second step, two similar comparative cases are investigated when wind excitations are applied with a steady-state wind speed of 20 m/s. Figs. 7.13a and b list the comparative time histories of the vertical and lateral contact forces at the 3rd wheel of the heavy truck, respectively.



(a) Vertical contact force



(b) Lateral contact force



By comparing Figs. 7.12 and 7.13, it is found that wind excitations at a steady-state wind speed of 20 m/s contributes to a large portion of the total vertical and lateral contact forces on the vehicle. The relative differences between the two comparative cases for vertical and lateral contact forces are generally smaller than those between the cases without wind excitations. This is partly due to the different characteristics of dynamic oscillation of the bridge and vehicles under windy and no-wind scenarios. It is also partly due to the fact that the wind force acting on vehicles, especially lift force, is much more dependent on the instantaneous vehicle driving speed rather than the movement of the supporting structure, as discussed earlier. It is demonstrated in Figs. 7.13a and b that the vertical and lateral contact forces of the vehicle in the traffic flow case are larger than those in the single-vehicle case at most time instants when the vehicle is driven on the bridge. For instance, at around 14.1 s, the vertical contact forces of the vehicle in the traffic flow case and single-vehicle case are 851 N and -1692 N, respectively. The lift-up at the windward side of the 3rd wheel may happen when multiple vehicles are present whereas it may not happen when only the single vehicle is involved. However, due to the complex interaction in the bridge-traffic-wind system, there are a few exceptions at which the contact forces of the vehicle are more disadvantageous in the single-vehicle case than those in the traffic flow case, especially for the vertical contact force. It becomes apparent that the presence of multiple vehicles on the bridge can significantly influence the vehicle accident condition on the bridge in the windy environment, underscoring the necessity of carrying out the study based on more realistic traffic simulation.

7.6.6 Influence of the dynamic initial states of vehicle when wind excitations are applied

It is found that a high-sided vehicle may be lifted up or slip laterally when the vehicle enters the crosswind gust suddenly. Previous results assume that the vehicle enters the wind environment at the starting point of the analysis. However in reality, the wind gust may occur at any time during the process when vehicles pass through the bridge. To investigate the influence of the initial states when wind excitations are applied, sudden wind gust is assumed to occur at time instants 22.04 s and 22.44 s, at which the vertical contact force at the windward side of the 3rd wheel of the heavy truck reaches the lowest and highest values when the vehicle is driven around the mid-span of the bridge, respectively, as shown in Fig. 7.12a. The vertical contact force time histories at the windward side of the 3rd wheel for the heavy truck are demonstrated in Fig. 7.14 for the cases with the two initial states as the start of wind excitations.



Figure 7.14 Vertical contact force with different start time of wind excitations

In Fig. 7.14, the vertical contact forces at around 22.5 s when the wind starts at 22.04 s and 22.44 s are 17,040 N and 12770 N, respectively. The results show that the vertical contact force at around the wind start time will be more disadvantageous if wind is applied suddenly at the time when the vehicle wheel reaches the lowest position relative to the bridge and starts to move upward. If the vehicle wheel is at the upward position and starts to move downward when wind is applied abruptly, the increase of the vertical contact force in the upward direction will be less significant. During the following several seconds, the vehicle vibration becomes steady and the difference between the two cases becomes smaller.

After around 35 s, there is hardly any difference for the response in two cases and the influence due to the different start time of wind excitations gradually disappear.

7.7 Conclusions

An integrated dynamic interaction and accident assessment framework was proposed based on the fully coupled bridge-traffic system formulation developed recently. Different from existing studies on vehicle accident analysis, the present study considered realistic stochastic traffic flow passing through a long-span bridge, in which multiple vehicles are on the bridge at the same time and the vehicles can accelerate and decelerate following certain traffic rules. The traffic accident analysis of any vehicle on a long-span bridge was conducted based on the fully coupled bridge-traffic interaction analysis of all vehicles of the stochastic traffic on the bridge simultaneously, providing more realistic insights than existing studies. The proposed approach is able to identify the vehicle lift-up, side-slip and yawing accidents within the same integrated simulation framework for the dynamic interaction analysis between the bridge and vehicles. Taking a prototype long-span cable-stayed bridge and three types of vehicles as example, this study evaluated the safety condition corresponding to each type of accidents for the representative heavy truck and light truck. The case study demonstrates that the proposed approach can be used to efficiently assess the safety of any vehicle of the traffic flow on the long-span cable-stayed bridge subjected to crosswind. Finally the influences from the multi-vehicle presence and dynamic initial states of wind excitations on traffic safety were numerically studied. Some specific observations are summarized in the following.

- The contact force of wheels is influenced by vehicle driving speed, turbulent wind speed, location on the bridge/road and the response on the contact point of bridge.
- The vehicle is more likely to be lifted up at the start time of wind excitations than any other time instants when other conditions remain the same.
- The vehicle is more likely to be lifted up or slip laterally when the vehicle is driven on a longspan bridge than on a road with other conditions remaining the same.

- The 3rd wheel set of the heavy truck is the most disadvantageous one in terms of lift-up or sideslip accidents. The 2nd wheel set is more disadvantageous to be lifted up than the 1st wheel set for the light truck. The side-slip risks of the two wheel sets for the light truck are similar.
- Side-slip accidents will not be likely to happen on dry road surface for both the heavy truck and light truck when the steady-state wind speed is no more than 20 m/s, while it may happen on slippery road surface, such as being covered with snow, water or ice.
- Yawing accident is usually associated with the side-slip of the wheels. Depending on the specific position where the lateral slipping at the wheels may occur, yawing accidents can happen in several possible manners for both the heavy and light trucks.
- The presence of multiple vehicles usually increases the variation of the dynamic response of the vehicle of interest in both vertical and lateral directions. The presence of multiple vehicles on the bridge may significantly influence the accident risks of the vehicles on the bridge in the windy environment.
- The vehicle accident condition on the bridge in the windy environment is closely related to the dynamic initial states of the vehicle when sudden wind excitations are applied.

CHAPTER 8 Fully coupled vehicle comfort analysis from whole-body vibration on long-span bridges subjected to crosswind^{*}

8.1 Introduction

Vehicle ride comfort issues for the drivers are not only related to individual satisfaction of driving experience, but more importantly, driving safety and long-term health of the drivers due to the deteriorated driving environments and performance. This is particularly true for those occupational drivers with extended driving time and also higher responsibilities such as those for public transportation and large cargo trucks (USDOT 2005; NIOSH 2007). It is known that ride comfort is directly related to the vibrations of the vehicle body experienced by the drivers or passengers. Exposure to excessive whole-body vibration of the vehicle may cause short-term body discomfort and long-term physical damage, which are often related to back and neck, such as musculoskeletal pain and back pain (Griffin 1990). Automobile and highway transportation industry have devoted significant efforts on studying ride comfort issues during the past decades and the recent studies on ride comfort analysis primarily focused on the so-called whole-body vehicle vibration measurements. The criteria associated with the whole-body vehicle vibration were also recommended in several existing standards to evaluate vibration severity and assessing vibration exposures (ISO 2631-1, 1985, 1997; BS 6841, 1987).

Although vehicle ride comfort issues are usually evaluated before any new vehicle is approved to be on the roads, these studies are typically conducted considering normal driving conditions, such as on roadways under normal weather. For adverse driving conditions, for example on slender long-span bridges and/or with adverse weather, related studies on ride comfort are still very limited. Long-span bridges regularly support a large amount of vehicles throughout a day, and therefore these critical bridges play an important role in maintaining the safety and efficiency of the entire transportation system. Since long-span bridges are often built in wind-prone regions with bridge decks at heights of over 50 m above

^{*} This chapter is submitted to a journal in a paper that is currently under review (Zhou and Chen 2015e).

the water, wind acting on the bridges and the passing vehicles on the bridges can be pretty significant. These bridges usually exhibit high flexibility and low structural damping due to slender girders and large spans, and therefore are susceptible to considerable wind-induced dynamic vibrations. In addition to wind, other service conditions such as high traffic volume and the presence of multiple heavy vehicles can prevail on long-span bridges. Existing studies show that both the vibration of passing vehicles and the long-span bridges can be significantly affected by the complex dynamic interaction among the bridge, vehicles and wind excitations (Chen and Wu 2010; Zhou and Chen 2015a, b). Consequently, the oscillation on the vehicles may induce ride discomfort and fatigue issues for the drivers, which may further pose increased traffic safety concerns related to the degraded driving behavior and performance.

Most existing studies on vehicle ride comfort evaluation focused on the vehicles that are driven on rigid roads without considering the interaction with supporting structures as well as wind excitations (e.g., Navhi et al. 2009). Besides, the traffic considered in the existing ride comfort studies was usually very approximate. For example, in the study by Xu and Guo (2004), the group of vehicles was assumed to be equally distributed and move along the bridge at a constant speed, which are apparently different from the realistic traffic on the roadway or bridges. In the study by Yin et al. (2011), only one vehicle is present on the bridge when vehicle comfort analysis is conducted. In addition to the oversimplified traffic simulation, the ride comfort analysis in the study by Xu and Guo (2004) was based on the single-axis root-mean-square (RMS) value with respect to one-third octave-band frequency in ISO 2631/1: 1985, which has been replaced with a new method in a later version (ISO 2631-1: 1997). In the new method (ISO 2631-1: 1997), the frequency-weighted RMS values are evaluated for ride comfort criterion based on multi-axis whole-body vibrations. Yin et al. (2011) evaluated the vehicle ride comfort performance based on the criteria in the latest version of ISO 2631-1 (1997), which however only considers the vehicle responses at the translational axes.

This chapter presents a general ride comfort evaluation framework for any vehicle in stochastic traffic flow by considering essential dynamic interactions with supporting structures and environmental loads (e.g., wind). Specifically, such a framework will be applied on long-span bridges by considering the complex dynamic interactions and also possible presence of various crosswind conditions. Built on the analytical framework developed previously by the writers (Zhou and Chen 2015a, b), the fully coupled dynamic analysis is firstly conducted on the bridge-traffic system under crosswind, in which the complex interactions among the bridge, vehicles and wind can be incorporated. The ride comfort is evaluated using the frequency weighting and averaging procedures as currently recommended in Ref (ISO 2631-1: 1997). In order to investigate the significance of the proposed study in terms of incorporating the stochastic traffic flow and wind effects, the study will start with the baseline scenario in which the vehicles move on the rigid road without considering the dynamic interactions with the supporting structure or crosswind. The coupled simulation framework is then applied to evaluate the driving comfort performance of representative vehicles of the simulated stochastic traffic flow on a long-span cable-stayed bridge subjected to crosswind excitations. Finally, the influence of dynamic interactions, presence of other vehicles and wind excitations on the ride comfort is numerically evaluated.

8.2 Fully coupled bridge-traffic-wind interaction analysis

8.2.1 Stochastic traffic flow simulation

Stochastic traffic flow is simulated to represent the realistic vehicle motion on the bridge when cable breakage occurs (Chen and Wu 2010). The instantaneous behavior of vehicles in the traffic flow is simulated by means of the cellular automaton model. It is a computationally efficient microscopic simulation methodology in the sense that time advances in discrete steps and space is discretized into multiple cells, each of which remains empty or occupied with one vehicle. By applying a set of probabilistic traffic transition rules regulating the accelerating, decelerating, lane changing and braking, the discrete variables in each cell are updated based on the vehicle information in the adjacent cells.

8.2.2 Structural idealization and finite element modeling of the bridge

The long-span cable-stayed bridge system is established as a three-dimensional finite element model in this study using two types of finite elements. The bridge superstructure and pylons are modeled with nonlinear three-dimensional spatial beam elements. The three-dimensional beam element is modeled based on the Timoshenko beam theory in which the axial, bending, torsional warping and shear deformation effects are considered at the same time. The stay cables are modeled as two-node catenary cable element based on the analytical explicit solution, which is obtained from the differential equations and boundary conditions for a cable with elastic catenary.

8.2.3 Modeling of road vehicles

Typical vehicles in the traffic flow are categorized into three types, which are heavy truck, light truck and light car (Chen and Wu 2011). The representative vehicle of each category is modeled as several rigid bodies and wheel axles connected by series of springs, dampers and pivots. The upper and lower springs are used to model the suspension system of each axle and elastic tires, respectively. Viscous dampers are used to model the energy dissipation system. The mass of the suspension system is assumed to be concentrated on the secondary rigid body at each wheel axle while the springs and dampers are assumed massless. The displacement vector d_v for the heavy truck model with 19 DOFs includes 8 independent vertical, 8 lateral and 3 rotational DOFs. The displacement vector for light truck and light car includes 5 independent vertical, 5 lateral and 2 torsional DOFs. The response vector of a typical heavy truck is expressed as follows.

$$d_{v} = \{Z_{r1} \ \theta_{r1} \ \beta_{r1} \ Z_{r2} \ \beta_{r2} \ Z_{a1L} \ Z_{a1R} \ Z_{a2L} \ Z_{a2R} \ Z_{a3L} \ Z_{a3R} \ Y_{r1} \ Y_{r2} \ Y_{a1L} \ Y_{a1R} \ Y_{a2L} \ Y_{a2R} \ Y_{a3L} \ Y_{a3R}\}$$
(8.1a)

in which, each degree of freedom in the vector is independent and can be shown in Fig. 8.1a-b.

 θ_{r^2} is dependent on other degrees of freedom which can be defined as:

$$\theta_{r2} = (Z_{r2} - Z_{r1} - L_5 \theta_{r1}) / L_6 \tag{8.1b}$$



(b) Side view

Figure 8.1 Numerical model of the heavy truck

The displacement vector d_{ν} for the light truck and light car is expressed in Eq. (8.2). The elevation view of the light car and light truck is shown in Fig. 8.2. The side view of the light car and truck is similar to that of the heavy truck and is omitted here.

$$d_{v} = \{ Z_{r_{1}} \theta_{r_{1}} \beta_{r_{1}} Z_{a1L} Z_{a1R} Z_{a2L} Z_{a2R} Y_{r_{1}} Y_{a1L} Y_{a1R} Y_{a2L} Y_{a2R} \}$$
(8.2)



Figure 8.2 Elevation view of light car and light truck

8.2.4 Modeling of wind forces on the bridge

The wind forces acting on the bridge are commonly separated into three components: steady-state forces resulting from average wind speed, self-excited forces resulting from wind-bridge interactions and buffeting forces resulting from turbulent wind velocity component. The wind forces are usually discretized as lift force, drag force and torsional moment for each of the three wind force components. The lift force will be given for demonstration for the self-excited and buffeting forces.

The self-excited lift force on a unit span can be expressed by the convolution integral between the arbitrary bridge deck motion and the associated impulse functions, as shown in Eq. (8.3).

$$L_{se}(t) = L_{sep}(t) + L_{seh}(t) + L_{sea}(t) = \int_{-\infty}^{t} f_{Lp}(t-\tau)p(\tau)d\tau + \int_{-\infty}^{t} f_{Lh}(t-\tau)h(\tau)d\tau + \int_{-\infty}^{t} f_{L\alpha}(t-\tau)\alpha(\tau)d\tau$$
(8.3)

in which, f is the response impulse functions; The subscripts "p", "h" and "a" indicate responses in lateral, vertical and torsional directions, respectively.

The buffeting lift force can be formulated using similar method as that for self-excited lift force:

$$L_{b}(t) = L_{bu}(t) + L_{bw}(t) = \int_{-\infty}^{t} f_{Lu}(t-\tau)u(\tau)d\tau + \int_{-\infty}^{t} f_{Lw}(t-\tau)w(\tau)d\tau$$
(8.4)

in which, the subscripts "u" and "w" indicate turbulent wind horizontal and vertical velocities, respectively. Detailed modeling of wind forces on the bridge can be referred to Chapter 7.

8.2.5 Modeling of wind forces on road vehicles

The aerodynamic wind forces acting on the vehicles are determined by means of a quasi-static approach proposed by Baker (1986). Assuming that the steady-state wind velocity is perpendicular to the longitudinal direction of the bridge girder, the relative wind velocity U_R to a vehicle driving on the bridge can be expressed in the following equation.

$$U_{R}(t) = \sqrt{(U_{w} + u(x,t))^{2} + U_{ve}^{2}(t)}$$
(8.5)

in which, u(x,t) is the turbulent wind speed; $U_{ve}(t)$ is the instantaneous driving speed of the vehicle.

The aerodynamic forces and moments on the vehicles have six components, which are drag force, side force, lift force, rolling moment, pitching moment and yawing moment, as expressed in Eqs. (8.6a-8.6f), respectively.

$$F_{vx}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{D}(\Psi)A \quad (8.6a) \qquad F_{vy}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{S}(\Psi)A \quad (8.6b) \qquad F_{vz}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{L}(\Psi)A$$
(8.6c)
(8.6c)

$$M_{vx}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{R}(\Psi)Ah_{v} \quad (8.6d) \qquad M_{vy}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{P}(\Psi)Ah_{v} \quad (8.6e) \qquad M_{vz}^{t} = \frac{1}{2}\rho U_{R}^{2}(t)C_{Y}(\Psi)Ah_{v} \quad (8.6f)$$

$$(8.6f)$$

in which, Ψ is the yaw angle, which is the angle between the direction of relative wind speed and the vehicle driving direction in the range from 0 to π ; $\psi = \arctan[U_w + u(x,t)/U_{ve}(t)]$; h_v is the vehicle reference height; $_A$ is the reference area; $C_D(\Psi)$, $C_S(\Psi)$, $C_L(\Psi)$, $C_R(\Psi)$, $C_P(\Psi)$ and $C_Y(\Psi)$ are the drag force coefficient, side force coefficient, lift force coefficient, rolling moment coefficient, pitching moment coefficient, respectively.

8.2.6 Equations of motion for the fully-coupled bridge-traffic-wind system

The coupled equations of motion of the bridge-traffic system can be built as shown in Eq. (8.7a).

$$\begin{bmatrix} M_{b} & 0 & \cdots & 0 \\ 0 & M_{v_{1}} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & M_{v_{s}} \end{bmatrix} \begin{bmatrix} \ddot{q}_{b} \\ \ddot{q}_{v_{1}} \\ \vdots \\ \ddot{q}_{v_{s}} \end{bmatrix} + \begin{bmatrix} C_{b} & 0 & \cdots & 0 \\ 0 & C_{v_{1}} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & 0 & C_{v_{s}} \end{bmatrix} \begin{bmatrix} \dot{q}_{b} \\ \dot{q}_{v_{1}} \\ \vdots \\ \dot{q}_{v_{s}} \end{bmatrix} + \begin{bmatrix} K_{b} & 0 & \cdots & 0 \\ 0 & K_{v_{1}} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & K_{v_{s}} \end{bmatrix} \begin{bmatrix} q_{b} \\ q_{v_{1}} \\ \vdots \\ q_{v_{s}} \end{bmatrix} = \begin{bmatrix} \sum_{i=1}^{n} F_{v_{i}}^{G} + F_{b}^{R} + F_{b}^{C} + F_{b}^{Se} + F_{b}^{Se} + F_{b}^{Bu} \\ F_{v_{1}}^{R} + F_{v_{1}}^{C} + F_{v_{1}}^{dw} \\ \vdots \\ F_{v_{s}}^{R} + F_{v_{s}}^{C} + F_{v_{s}}^{dw} \end{bmatrix}$$

$$(8.7a)$$

$$F_{b}^{C} = \sum_{i=1}^{n} C_{bci} \{\dot{q}_{b}\} + \sum_{i=1}^{n} K_{bci} \{q_{b}\} + \sum_{i=1}^{n} C_{b,v_{i}} \{\dot{q}_{v_{i}}\} + \sum_{i=1}^{n} K_{b,v_{i}} \{q_{v_{i}}\}$$
(8.7b)

$$F_{v_i}^{\ C} = C_{v_i,b} \{ \dot{q}_{v_i} \} + K_{v_i,b} \{ q_{v_i} \}$$
(8.7c)

in which, *n* is the total number of vehicles; *q* is the displacement vector; *M*, *K*, *C* and *F* are structural mass, stiffness, damping matrices and force vector; subscripts *b* and *v*_i (*i* = 1, 2, ..., *n*) indicate that the parameters are for the bridge and the *i*th vehicle, respectively; K_{bci} and C_{bci} refer to the stiffness and damping contributions to the bridge structure due to the coupling effects between the *i*th vehicle in the traffic flow and the bridge system, respectively; $K_{b,v}$ ($K_{v,b}$) and $C_{b,v}$ ($C_{v,b}$) are the coupled stiffness and damping matrices contributing to the bridge (vehicle) vibration, respectively; the superscripts *G*, *R*, *C*, *W*, *Se* and *Bu* refer to the external loads caused by gravity, road surface roughness, coupling interaction forces, static wind, self-excited and buffeting forces, respectively.

The coupling matrices between the bridge and vehicles need to be updated at each time step according to the instantaneous contacting location of each vehicle during the movement on the bridge (Cai and Chen, 2004). The self-excited forces due to turbulent wind speeds and coupling interaction forces between the bridge and vehicles are dependent on bridge unknown motion, therefore they should be calculated iteratively starting with an initial motion vector at each time step until the prescribed convergence criterion is satisfied.

8.3 Ride comfort analysis on road vehicles

There are several standards that have provided procedures for evaluating human exposure to wholebody vibration and repeated shock. The tolerance of human vibration to vehicle ride vibration is known difficult to be evaluated despite extensive research efforts during the past several decades. There is thus no consensus on a criterion that can be unanimously accepted around the world for ride comfort. The regulations issued by the International Standard Organization (ISO) are one of the most popular criteria in current practice for ride comfort evaluations. The latest version (ISO 2631-1 1997) has been updated from the previous 1985 version and is the only international standard for evaluating the whole-body vibration, which can be used for health, comfort, perception and motion sickness. Due to its popularity and applicability, the whole-body vibration measures (ISO 2631-1 1997) are adopted in the present study for vehicle ride comfort analysis and a brief summary is provided in the following sections.

8.3.1 Whole-body vibration measures

The ISO 2631-1 (1997) standard (called "standard" hereafter) is applied to assess the motions transmitted to a human body as a whole through all the supporting surfaces. For example, for a seated person, the three supporting surfaces are the seat, backrest and floor supporting the buttocks, back and feet of the person; for a standing person, the supporting surface is only the floor supporting the feet of the person; for a recumbent person, the supporting surface only includes the side supporting area of the person. This study takes the seated position to investigate the driver comfort by considering all the three supporting surfaces. Apparently, the other two positions are comparatively less complicated by involving only one supporting surface and thus can also be evaluated using the proposed approach.

The standard measures the body vibration through 12 axles of a seated person, which are the vertical, lateral, fore-and-aft, yawing, pitching and rolling axes for the seat surface and the vertical, lateral and fore-and-aft axes for both the backrest and floor surfaces. The axles and locations where vibrations should be measured for comfort analysis based on a seated person are demonstrated in Fig. 8.3.



Figure 8.3 Axles and locations for vibration measurements for a seated person (ISO 2631-1 1997)

From the coupled analysis of the bridge-traffic system, the acceleration response at the centroids of rigid bodies and mass axles of each vehicle can be obtained. The vehicle acceleration response at the seat, backrest and floor of the vehicles can be further determined through the vehicle response from the coupled analysis. It is known that the fore-and-aft acceleration of a vehicle primarily depends on the traction and braking forces exerted by the driver. Since this study focuses on ride comfort issues on long-span bridges under windy conditions, such acceleration has little impact on dynamic interaction and in turn related ride comfort results. Therefore, the fore-and-aft DOF is not included in the vehicle model and the associated acceleration response is not involved in the ride comfort analysis in the present study. In addition, the yawing effects of the vehicles are usually much less significant than the rolling and pitching effects when the bridge-vehicle interaction and wind excitations are considered. In the present study, the yawing acceleration is also ignored to obtain the total vibration effects of the vehicles.

There will be a total of 8 axes that participate in the ride comfort evaluation, including three vertical and three lateral axes at seat, backrest and floor locations, one pitching and one rolling axis at the seat

location. The eight axes are noted as vs, vb, vf, ls, lb, lf, ps and rs in the present study, in which the first letter indicates the response direction and the second letter indicates the axis location. The response direction can be v (vertical), l (lateral), p (pitching) and r (rolling). The axis location can be s (seat), b (backrest) and f (floor). The acceleration response at each participating axis of the vehicle can be expressed in Eqs. (8.8a-8.8d).

$$a_{vs} = a_{vb} = a_{vf} = \ddot{Z}_{r1} \quad (8.8a) \quad a_{ls} = a_{lb} = a_{lf} = \ddot{Y}_{r1} \quad (8.8b)$$
$$a_{ps} = \ddot{\theta}_{r1} \cdot d_s \quad (8.8c) \quad a_{rs} = \ddot{\beta}_{r1} \cdot y_s + \frac{1}{2} \ddot{\beta}_{r1} \cdot h_s \quad (8.8d)$$

where *a* indicates acceleration at different axis; \ddot{z}_{r_1} , \ddot{y}_{r_1} , $\ddot{\theta}_{r_1}$ and $\ddot{\beta}_{r_1}$ are the acceleration at the first rigid body centroid of the vehicle in the vertical, lateral, pitching and rolling directions, respectively; d_s , y_s and h_s are the longitudinal, transverse and vertical distance between the centroid of the first rigid body and the seat, respectively. It is noted that angular acceleration in the pitching and rolling directions are transformed into linear acceleration to be consistent in dimension for calculating the total vibration effects.

8.3.2 *Frequency weighting technique*

According to the standard, the acceleration response of the supporting surface at each axle should be frequency-weighted before calculating the total effects from vibration exposure. The purpose of weighting the original acceleration data is to more realistically model the frequency response of the human body. Since no specific frequency weighting methodology is recommended in the standard, Fast Fourier Transform (FFT) convolution is adopted in the present study. For a response signal x with N discrete values with respect to time, the Discrete Fourier Transform (DFT) is conducted to obtain the signal value X with respect to discrete frequency in the frequency domain, as shown in Eq. (8.9).

$$X(r) = \sum_{k=0}^{N-1} x(k) \omega_N^{rk}$$
(8.9)

in which, $\omega_N = e^{-2\pi r/N}$, r = 0, 1, ..., N-1.
The weighted signal X in the frequency domain can be obtained as the product of the real and imaginary part of X and the corresponding frequency weighting \times , as shown in Eq. (8.10).

$$X'(r) = X(r) \cdot \chi(r) \tag{8.10}$$

The weighted signal X' is then transformed back into the time domain using the inverse DFT to obtain the frequency-weighted signal x' in the time domain.

$$x'(j) = \frac{1}{N} \sum_{r=0}^{N-1} X'(r) \omega_N^{-jr}$$
(8.11)

in which, j = 0, 1, ..., N - 1.

8.3.3 Weighting factors and multiplying factors

The standard (ISO 2631-1 1997) gives the multiplying factors for each of the axes in order to compensate the varying vibrating effects due to different locations and directions. There are four multiplying factors M_k , M_d , M_e and M_c for the participating axes, which are shown in Table 8.1 indicating the application in different locations and directions. The multiplying factors with the subscript "k" are used for vertical axis on the seat and all translational axes on the floor. The multiplying factors with the subscript "d" are used for lateral axes on the seat and floor. Those with the subscript "e" are applied for the rotational axes on the seat. The factor with the subscript "c" is only used in the vertical axis on the backrest.

Value Multiplying factor Location Direction 1.00 Vertical M_k Seat M_d 1.00 Seat Lateral M_{e} 0.40 Seat Pitching Me 0.20 Seat Rolling M_c 0.40 Backrest Vertical M_d 0.50 Backrest Lateral M_k 0.40 Vertical Floor

Table 8.1 Multiplying factors for the participating axes from ISO 2631-1 (1997)

M_k	0.25	Floor	Lateral
ĸ			

The frequency weighting factors are named in a similar way to the multiplying factors. The weighting factors W with the subscripts "k", "d", "e" and "c" are used in the same locations and directions as those of the multiplying factors. The weighting factors are originally quantified in decibel (dB) in the standard and in the present study, the unit of dB is transformed to a dimensionless ratio δ through the following equation.

$$\delta = 10 \exp(\frac{dB}{10}) \tag{8.12}$$

The weighting factors W_k , W_d , W_e , W_c that are used in the frequency domain are shown in Fig. 8.4, which act as several filters to the original response. It is seen that the application of frequency weighting factors reduces the effects of the low and high frequency contents of the response signal. Taking W_k for instance, the frequency contents below 0.63 Hz will account for the total vibrating effects with less than 45 percent of the original record. As the frequency goes lower, the weighting factor will become smaller. The frequency contents in the range between 5 Hz and 8 Hz will be weighted by a weighting factor that is slightly larger than 1.0. After 8 Hz, the weighting factor becomes smaller than 1.0 as frequency increases.



Figure 8.4 Weighting curves of ISO 2631-1 standard for different axes

8.3.4 Determining the overall vibration total value (OVTV)

After applying the frequency weighting factors, the Root-Mean-Square (RMS) values of the weighted response acceleration vector for each axis can be obtained. The weighted RMS values will then be combined for all measurement locations and directions to obtain the overall vibration total value (OVTV). Based on the response at the eight axes involved in the present study, the OVTV can be obtained from the following equation:

$$OVTV = \sqrt{M_k^2 RMS_{vs}^2 + M_d^2 RMS_{ls}^2 + M_e^2 RMS_{ps}^2 + M_e^2 RMS_{rs}^2 + M_d^2 RMS_{vb}^2 + M_d^2 RMS_{lb}^2 + M_k^2 RMS_{vf}^2 + M_k^2 RMS_{lf}^2}$$
(8.13)

in which, *RMS* is the RMS value of frequency-weighted acceleration for each measurement location; the first subscripts "v", "l", "p" and "r" indicate the vertical, lateral, pitching and rolling direction, respectively; the second subscripts "s", "b" and "f" for the acceleration RMS refer to the locations on the seat, backrest and floor, respectively.

8.3.5 Subjective criteria for ride comfort

The criteria of different discomfort levels as suggested by ISO 2631-1 (1997) are shown in Table 8.2. The subjective criteria for ride comfort have overlapping ranges of the OVTV values. The ride comfort of the vehicles is evaluated based on the criteria in Table 8.2 in the present study.

OVTV value (m/s ²)	Subjective indication
< 0.315	Not uncomfortable
0.315-0.63	A little uncomfortable
0.5-1.0	Fairly uncomfortable
0.8-1.6	Uncomfortable
1.25-2.5	Very uncomfortable
>2.0	Extremely uncomfortable

Table 8.2 Subjective criteria for ride comfort based the OVTV

8.4 Numerical demonstration

8.4.1 *The prototype long-span cable-stayed bridge*

The prototype long-span cable-stayed bridge in the present study has a main span, two side spans and two approach spans. The main span length and the total bridge length are 372.5 m and 840 m, respectively, which is shown in Fig. 5.3a. The cable-stayed bridge deck has a steel twin-box cross-section with a width of 28 m and a height of 4.57 m, which is shown in Fig. 5.3b. The two steel pylons have A-shaped cross-sections and are 103.6 m in height. The two cable planes of the bridge are placed in a fan-shaped arrangement with 12 sparsely located cables in each cable plane. The bridge superstructure is partially supported by the reinforced concrete bridge piers with sliding bearing supports at the side spans.

8.4.2 Traffic flow simulation and the prototype vehicles

8.4.2.1 Simulated traffic flow pattern

The vehicles in the stochastic traffic flow are categorized as three types, which are light car, light truck and heavy truck. The percentage in the simulated traffic flow is 20%, 30% and 50% for light car, light truck and heavy truck, respectively. In the driving direction, the entire travel path for the traffic flow includes three sections, which are the bridge section with the length of 840 m and two road sections with the length of 210 m each at the two ends of the bridge section. In the transverse direction, the entire travel path contains a total of four lanes with two lanes in each of the two driving directions. A stochastic traffic flow pattern with a density of 21 vehicles per kilometer per lane is simulated using double-lane cellular automaton model (Chen and Wu 2011). The number of total vehicles remains the same by using periodic boundary conditions. There are a total of 108 vehicles in the simulated traffic flow, including 54 light cars, 32 light trucks and 22 heavy trucks. The initial locations of the vehicles are randomly distributed on the whole travel path. One representative vehicle for each type of vehicles is selected as the ones with the initial locations at the far end in the road section before entering the bridge. The driving paths for the representative light car, light truck and heavy truck are shown in Fig. 8.5. It is seen that the vehicles experience varying driving speeds as reflected by the changing of line slopes of the driving curves. The

decreasing and increasing of the line slopes indicate that the vehicle accelerates and decelerates, respectively. The slope of 90° indicates the vehicle takes a complete brake and remains still. Except for at around 20 second when all the vehicles stop moving by following some realistic traffic rules of congestion, the vehicle speeds generally keep steady in the range between 22.5 m/s and 30 m/s.



Figure 8.5 Vehicle longitudinal location of the representative vehicles

8.4.2.2 <u>Numerical properties of the prototype vehicles</u>

The baseline dynamic parameters for each type of vehicles involved in the present study, including mass, mass moment of inertia, stiffness coefficients and damping coefficients, are listed in Table 5.3 in Chapter 5. The baseline dimension parameters for three types of representative vehicles are listed in Table 8.3.

Table 8.3 Dimensions of the vehicles used in the study

Parameter	Unit	Heavy truck	Light truck	Light car
d_1 between axle 1 and rigid body 1	т	1.83	1.8	1.34
d_2 between axle 2 and rigid body 1	т	1.83	2.0	1.34
d_3 between axle 2 and rigid body 2	т	3.60	-	-
d_4 between axle 3 and rigid body 2	т	2.60	-	-
d_5 between pin and rigid body 1	т	1.83	-	-
d_6 between pin and rigid body 2	т	3.60	-	-

Parameter	Unit	Heavy truck	Light truck	Light car
Y_1 between axle and rigid body	т	1.10	1.0	0.8
Frontal area A	m^2	10.5	6.5	1.96
Reference height h_{ν}	т	2.0	1.65	1.1
d_s between centroid and seat location	т	0.6	1.5	0.9
y_s between centroid and seat location	т	0.5	0.4	0.3
h_s between centroid and seat location	т	0.7	0.4	0.3

8.4.3 Baseline scenario for vehicle driving comfort analysis on roadways

In the baseline scenario, all the three sections of the driving path are assumed to be rigid roadways without considering the interactions between the vehicles and supporting structure. No wind speed is applied on the vehicles, which vibrate on their own under only the external excitations from road surface roughness. The dynamic responses of the representative vehicles for a simulation time period of 50 seconds are investigated for vehicle ride comfort analysis at the driver location.

8.4.3.1 Whole-body acceleration response

The acceleration responses of the representative light car in the vertical, pitching and rolling directions at the driver seat location are depicted in Figs. 8.6a, b and c, respectively. When the vehicle is driven on the rigid road, the lateral vibration is minimal since the only external excitation is road surface roughness. The accelerations from vertical, pitching and rolling movements of the light car have nearly zero mean values and similar magnitudes of extreme values.



(a) Vertical acceleration



(c) Rolling acceleration

Figure 8.6 Acceleration response at the driver seat location for the representative light car

The RMS values of the vertical, pitching and rolling accelerations at the driver seat for the representative light car, light truck and heavy truck are shown in Fig. 8.7. The light truck has the largest RMS values in the vertical, pitching and rolling directions at the driver seat location among all the three representative vehicles. For the light truck and light car, the RMS values in the vertical axis are the largest among all the three axes and the RMS values in the pitching and rolling axis are close to each other. Different from the light car and light truck, the heavy truck has similar values in the vertical and pitching directions, which are larger than the value in the pitching direction.



Figure 8.7 RMS values of the vertical, pitching and rolling acceleration for the representative vehicles 8.4.3.2 *Frequency weighting of the acceleration response*

After the original acceleration response of each axis is obtained, the response signal should be frequency-weighted to obtain the OVTV. Taking the vertical acceleration at the seat location of the light car for instance, the acceleration spectrum for the original response is shown in Fig. 8.8a. By applying the frequency-weighting curve W_k to the original spectrum, the frequency-weighted spectrum of the response can be obtained, which is shown in Fig. 8.8b. Through the comparison between Figs. 8.8a and 8.8b, it is seen that after the acceleration response is frequency-weighted, the acceleration magnitude at the frequency range below 2 Hz is significantly reduced.



(a) Spectrum of the original response



(b) Spectrum of the frequency-weighted response

Figure 8.8 Single-Sided Amplitude Spectrum of the vertical acceleration response of the light car Through the inverse Fast Fourier Transform, the frequency-weighted acceleration response time history is obtained, which is shown in Fig. 8.9 in comparison with the original acceleration response. The frequency-weighted response has smaller extreme response than the original response because the response magnitudes at certain frequency ranges are significantly reduced. The RMS values of the vertical, pitching and rolling acceleration at the seat location of the light car are demonstrated in Fig. 8.10. It is seen that the RMS of the weighted vertical and pitching accelerations only account for 49 % and 39 % of the total RMS of the original response, respectively. Compared to the vertical and pitching accelerations, the frequency-weighting filter has relative smaller influence on the rolling acceleration response. Considering the pitching and rolling accelerations use same frequency-weighting filter W_e , the difference is mainly due to the fact that the frequency components of the rolling acceleration in the lower (< 0.8 Hz) and higher frequency (> 90 Hz) range account for less proportion of the total response than pitching accelerations. Since the frequency-weighting is apparently important in obtaining proper OVTV for the following ride comfort evaluation.



Figure 8.9 Comparison of the original and frequency-weighted vertical acceleration response at seat

Original response Frequency-weighted response 0.22 0.2 Acceleration RMS (m/s²) 0.18 0.16 0.14 0.12 0.1 0.08 0.06 ps rs vs Axle index

location for the representative light car

Figure 8.10 RMS values of the original and frequency-weighted response of the light car at the driver seat 8.4.3.3 <u>Generation of OVTV</u>

After the RMS values of the frequency-weighted acceleration response for each axis are obtained, the multiplying factors are applied to obtain the OVTV of the vehicle in order to evaluate the ride comfort level. The OVTV value for each representative vehicle is listed in Table 8.4. It is shown that when the vehicles move normally on a rigid roadway in moderate traffic flow, the ride comfort of the vehicles satisfies the criteria as specified in ISO 2631-1.

Table	8.4	OV	ΤV	of	the	rep	rese	enta	tive	veł	nicl	es	in	the	base	line	sce	nar	ic
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	Light car	Light truck	Heavy truck
OVTV (m/s ²)	0.1161	0.1337	0.0985

The contributing percentages in OVTV from the acceleration response at each of the eight participating axis are computed for each of the three representative vehicles. The contributing proportion is obtained using the following equation.

$$\chi_{ij} = M_{ind}^2 \cdot a_{ij}^2 / OVT \dot{V}$$
(8.14)

in which, the subscript *i* and *j* indicate the response direction and location, respectively; the subscript *i* can be *v*, *l*, *p* and *s*, representing vertical, lateral, pitching and rolling direction, respectively; the subscript *j* can be *s*, *b* and *f*, indicating seat, backrest and floor location, respectively; the subscript *ij* is the axis index that participates in calculating the OVTV and the subscript *ind* refers to the different multiplying factors that are applied to different axis, which can be referred to Table 1; a_{ij} is the RMS value of the frequency weighted acceleration response at the designated axis *ij*.



Figure 8.11 Proportion of the response at different axes in OVTV for the representative vehicles It is shown in Fig. 8.11 that the vertical acceleration at the seat location takes up the largest portion in the OVTV for the light car and heavy truck. While for the light truck, the proportion of the pitching

acceleration at the seat location is the largest. When the vehicle moves normally on the rigid road, the lateral accelerations are minimal and barely contribute to the OVTV. The responses at the axes in the seat location contribute much more in the OVTV than those in the backrest and floor locations. The proportions of the response in the seat locations for the light car, light truck and heavy truck are 0.817, 0.853 and 0.838, respectively. The largest proportion of OVTV contributed by the response at one single axis for the light car, light truck and heavy truck is 0.537, 0.408 and 0.606, respectively. It is indicated that the whole body vibration response including those in different participating axes is essential for evaluating ride comfort based on the criteria by ISO. To consider only the vibration in one axis like many earlier studies may underestimate the total vibration response and in turn ride comfort evaluation results.

8.4.4 *Effects of interactions between the vehicles and the supporting bridge structure*

The effects of the interaction between the vehicles and the supporting bridge are investigated in this section. The coupled bridge-traffic interaction analysis is conducted to obtain the dynamic response of the bridge and each participating vehicle. The study firstly conducts the coupled analysis with only one single vehicle on the bridge. The effects of the bridge-vehicle interaction due to the presence of multiple vehicles are further investigated by conducting the coupled analysis of the bridge and the entire traffic flow. No wind excitations are considered in this section and only the effects due to bridge-vehicle interactions are considered.

8.4.4.1 Single vehicle vibration analysis

In the present case, only one single representative vehicle is involved in the bridge-vehicle interaction analysis and all other vehicles in the traffic flow are excluded. Each of the representative vehicles is driven through the road-bridge-road path separately. For each simulation, the driving path for each representative vehicle exactly follows the curve in Fig. 8.5 in order to make sure the vehicle moves following the same pattern as in the baseline scenario. The difference as compared to the baseline scenario is that the dynamic interactions between the bridge and vehicle are considered in the present case. By comparing the acceleration response at each axis of the vehicles, it is found that the vehicle responses are not affected significantly by the interactions between the bridge and a single vehicle. The RMS values of the response at each participating axis are listed in Table 8.5 for the three representative vehicles. It can be seen that the RMS value in the case considering the bridge-single-vehicle interaction is very close to the corresponding value in the baseline case with a difference less than 1 percent.

RMS (m/s ²)	Light car	Light truck	Heavy truck
At axis vs (on bridge)	0.1734	0.1317	0.1630
At axis vs (on road)	0.1744	0.1342	0.1623
At axis ls (on bridge)	0.0000	0.0002	0.0001
At axis ls (on road)	0.0000	0.0000	0.0000
At axis ps (on bridge)	0.1647	0.2965	0.0851
At axis ps (on road)	0.1649	0.2976	0.0849
At axis rs (on bridge)	0.2687	0.3436	0.3044
At axis rs (on road)	0.2687	0.3437	0.3044

Table 8.5 RMS values for the case with single vehicle on the bridge and baseline case

After the acceleration response at each axis of the vehicles is frequency-weighted, the OVTV of each vehicle driven on the bridge can be obtained and the results are listed in Table 8.6. For comparison purposes, the OVTVs of the vehicles on rigid road from the baseline scenario are also listed. Very similar results suggest that the influence on the vehicle ride comfort from the interaction between the bridge and a single vehicle is negligible.

Table 8 6 The OVTV	of the representative v	ahiclas when a single	vehicle on the	hridge and a	on the road
	of the representative v	cilicits which a single		, or luge and t	JII LIIC IUau

OVTV (m/s ²)	Light car	Light truck	Heavy truck
Vehicle on bridge	0.1158	0.1337	0.0987
Vehicle on road	0.1161	0.1337	0.0985

8.4.4.2 Effects of the presence of multiple vehicles

In this scenario, the coupled bridge-vehicle interaction analysis is conducted on the bridge and all the vehicles in the traffic flow. The acceleration responses of the three representative vehicles are compared with the corresponding results of the single representative vehicle case on the bridge in the previous section. Figs. 8.12a-8.12c give the vertical, lateral and pitching acceleration results of the light truck for the traffic flow case, respectively, with the results of the single-vehicle case being also plotted for a comparison. No discernible difference of the rolling acceleration of the comparative cases can be observed and the results are therefore not listed for the sake of brevity.



(b) Lateral acceleration



(c) Pitching acceleration

Figure 8.12 Comparison of the acceleration response of the representative light truck in the traffic flow and single vehicle case

As shown in Figs. 8.12a-8.12b, the vertical and lateral accelerations of the light truck with the presence of multiple vehicles are much larger than those with only a single vehicle after the vehicle enters the bridge. The pitching acceleration of the vehicle is notably influenced due to the presence of other vehicles. However, the extreme values of the traffic flow case and single vehicle case don't exhibit as large difference as compared with those from vertical and lateral directions (Fig. 8.12c), e.g., the maximum positive extreme pitching accelerations with the traffic flow and with only the single vehicle are 0.990 and 0.794 m/s², respectively. The RMS values of the vertical, lateral, pitching and rolling accelerations at the seat location are obtained for each representative vehicle in the traffic flow case and single vehicle case, which are shown in Fig. 8.13. It is found that the RMS value of the acceleration in the lateral direction is minimal when compared to those in other directions. The RMS values of rolling acceleration in the comparative cases are very close for all the three types of representative vehicles, indicating that the presence of multiple other vehicles has little effect on the rolling acceleration of the vehicle. In contrast, the presence of multiple vehicles may significantly affect the vertical acceleration of the representative vehicle, as evidenced by the much larger RMS value in the traffic flow case than in the single vehicle case. The ratios of the RMS values of the vertical acceleration in traffic flow case over those in the single vehicle case are 2.67, 3.72 and 1.65 for the light car, light truck and heavy truck,

respectively. The vertical acceleration at the seat location of the light truck and heavy truck are the most and least vulnerable to be affected by the presence of multiple vehicles in the traffic flow, respectively. Such an observation also holds true for the pitching acceleration at the seat location, as it is found that the RMS ratios in the traffic flow case over the single vehicle case in the pitching direction are 1.07, 1.12 and 1.06 for the light car, light truck and heavy truck, respectively. However, it should be noted that the influence of the multiple vehicles on the pitching acceleration is comparatively much smaller than that on the vertical accelerations of the representative vehicles.



Figure 8.13 RMS values of the acceleration response at the seat location for the representative vehicles

The original acceleration response at each axis is frequency weighted and the RMS value of the filtered/weighted acceleration is obtained. The OVTVs in the traffic flow case for each representative vehicle are obtained and listed in Table 8. The OVTVs of each single representative vehicle on the bridge are also listed for comparison. It is shown that the OVTV for ride comfort evaluation is much larger when the vehicle travels on the bridge with the simultaneous presence of multiple other vehicles than that in a single vehicle scenario. The ratios of OVTV in the traffic flow over that with the single vehicle are 2.11, 2.55 and 1.46 for the light car, light truck and heavy truck, respectively. The OVTV of the light truck and heavy truck are the most and least likely to be affected by the presence of other vehicles, respectively. From the OVTV results in Table 8.7, it is found that the drivers of the light car and heavy truck may experience acceptable comfortable levels on the bridge while the driver of the light truck may experience

some discomfort on the bridge according to the ride comfort criteria set by ISO 2631-1. Considering that the presence of multiple vehicles is the most common scenario on the bridge, it can be concluded that the interactions between the bridge and the moving traffic may significantly influence the ride comfort level of a typical vehicle. Therefore realistic simulations of stochastic traffic flow and dynamic interaction should be appropriately made to accommodate the impacts from other vehicles on the ride comfort evaluations on long-span bridges.

OVTV (m/s ²)	Light car	Light truck	Heavy truck
In the traffic flow	0.2457	0.3415	0.1437
Single vehicle	0.1158	0.1337	0.0987

Table 8.7 OVTV of the vehicles in the traffic flow and single vehicle case

Similar to Fig. 8.11, the proportion of the acceleration response in the OVTV when the traffic is on the bridge is also evaluated and shown in Fig. 8.14 for the representative vehicles. Compared to those in the baseline scenario in Fig. 8.11, the results for different types of representative vehicles on the bridge are actually close considering interactions due to multiple other vehicles. When the representative vehicles are driven on the bridge as part of the traffic flow, the vertical accleration at the seat location becomes the dominant contributor to the OVTV for each type of representative vehicles. This is consistent with the previous observation about stronger dynamic interactions in the vertical direction for the bridge/traffic system than other directions (Fig. 8.12). In the mean time, the pitching and rolling accelerations account for much less proportions in the OVTV than those in the baseline scenrio, especially for the light car and light truck. It is indicated that the motions of the supporting bridge structure as well as dynamic interactions with multiple vehicles may practically "even" the difference of the contributing proportions by different axises to the OVTV among different types of representative vehicles.



Figure 8.14 Proportions of the response in each participating axis in OVTV for the representative vehicles 8.4.5 *Effects of wind excitations*

The effects of wind excitations on the ride comfort of the vehicles are investigated when these vehicles move through the bridge. It is assumed that wind excitations at a steady state wind speed of 20 m/s are applied on the bridge and all the vehicles in the traffic flow. Coupled bridge-traffic-wind interaction analysis is conducted to obtain the acceleration response of the vehicles in the traffic flow under wind excitations of 20 m/s. The vehicle acceleration response histories of the vehicles are compared with those without wind excitations. In both of the comparative cases, the interactions between the bridge and multiple vehicles in the traffic flow are considered. Figs. 8.15a, b and c demonstrate the vertical, lateral, pitching and rolling accelerations of the light car for instance, respectively.



(a) Vertical acceleration



(d) Rolling acceleration

Figure 8.15 Comparison of the acceleration response of the representative light car under different wind

speeds

It is found in Fig. 8.15 that the acceleration responses of the vehicle experience an abrupt increase at the start time when wind excitations are applied suddenly. This is especially notable for the acceleration response in the lateral and rolling directions. It is observed that the lateral and rolling accelerations of the vehicle are more likely to be influenced in the wind environment than the vertical and pitching accelerations. The RMS of the accelerations in the vertical, lateral, pitching and rolling directions at the seat locations for the cases with wind excitations are given in Figs. 8.16a, b and c in comparison with the cases without wind excitations for the representative light car, light truck and heavy truck, respectively.



(b) Light truck



(c) Heavy truck

Figure 8.16 RMS values of the acceleration response of the representative vehicles for the cases with or without wind excitations

It is seen from Fig. 8.16 that the RMS values of the pitching acceleration are hardly affected by the wind excitations for the vehicles. The accelerations in the other three directions are affected by the wind excitations to various extents, among which the influence of wind speeds on rolling accelerations is most remarkable. Unlike the response results without wind excitations, the RMS values of rolling and pitching accelerations become the largest and smallest ones among the four axes for each type of the vehicles, respectively.

The RMS values of the frequency-weighted acceleration responses in all different axes are then obtained in the case with wind speed of 20 m/s in order to obtain the OVTV for the vehicles. The proportions of the weighted acceleration response at each participating axis in OVTV are obtained for the three representative vehicles (Fig. 8.17). It is noted that for vertical acceleration at the seat location, applying the frequency-weighting filter W_k causes a significant decrease of the RMS value of the acceleration response, as demonstrated in Fig. 8.10 in the previous sections. However, for lateral acceleration at the seat location, the frequency weighting filter W_d doesn't pose significant influence on the response and the filtered acceleration history in the lateral direction is very close to the original acceleration history. Therefore, although it is shown in Figs. 8.16a-c that RMS values in the vertical direction are notably larger than those in the lateral direction for the vehicles, the proportions of the weighted response in OVTV in the vertical axis are not always larger than those in the lateral axis. By

comparing Fig. 8.17 with Fig. 8.14 in which no wind excitation was considered, it is obvious that the proportion of the lateral acceleration at the seat location increases significantly when wind excitations are considered. For the light truck and heavy truck, the lateral acceleration at the seat location account for the largest portion in the respective OVTV, which are much larger than the corresponding values of the vertical acceleration. For the light car, the proportion of the vertical acceleration is slightly larger than that of the lateral acceleration. As the high-sided vehicles, the light truck and heavy truck are more prone to the lateral excitations from cross wind than the low-sided light car.



Figure 8.17 Proportion of acceleration response at each participating axis in OVTV for the representative vehicles

In addition to the wind excitations at the speed of 20 m/s, the coupled bridge-traffic-wind interactions are also conducted when the wind speeds are 10 m/s and 30 m/s. Although it is assumed in AASHTO specifications (AASHTO 2012) that bridges may be closed when the steady-state wind speed reaches 25 m/s, realistically, long-span bridges are often not closed even such wind conditions are met (Chen and Wu 2010). Therefore, wind excitations at the speed of 30 m/s are also considered in the present study considering that there might be strong wind gusts lasting for a relatively short time period not allowing the drivers or emergency managers to respond in time. The OVTVs are obtained for the representative vehicles as wind speed increases, as shown in Fig. 8.18.



Figure 8.18 The OVTVs for the representative vehicles at different wind speeds

The OVTVs for the driver of the vehicles increase as wind speed increases for all three types of representative vehicles. When the wind speed is not higher than 20 m/s, the increase of the OVTV keeps steady with the increase of wind speeds. When the speed is over 20 m/s, the OVTVs increase more significantly as the wind speed goes higher. Among the three representative vehicles, the largest and smallest OVTVs under the same wind speed occur on the light truck and the heavy truck, respectively. This also holds true in the cases without considering wind excitations as shown earlier. When the wind speed reaches 20 m/s and higher, the OVTV of the light truck increases at a larger rate than that of the light car and heavy truck. By interpolating the curves at the OVTV value of 0.315 m/s² as recommended by ISO in Fig. 8.18, it can be found that the driver will not feel uncomfortable if the wind speed is lower than 8.6 m/s and 23.8 m/s for the light car and heavy truck, respectively (on the bridge with moderate traffic flow). For the light truck, the driver may feel a little uncomfortable even if when the wind speed is zero. As long as the wind speed is not higher than 20.5 m/s, the driver will only feel a little uncomfortable when driving on the bridge in a moderate traffic flow based on the ISO ride comfort criteria. It is however pointed out that when the wind speed increases beyond 20 m/s, the vehicle safety issue may replace the vehicle ride comfort issue to become dominant, which is usually assessed separately in Chapter 7.

8.5 Conclusions

This chapter presents a new methodology of ride comfort analysis for typical vehicles driven on longspan bridges in the windy environment by considering the complex dynamic interactions and adopting advanced techniques for ride comfort evaluation. With the dynamic analytical framework proposed by the writer in previous studies, the long-span bridge, wind and all the vehicles in the traffic flow are directly coupled and more accurate vehicle responses can be obtained. The guidelines recommended in ISO 2631-1 (1997) for vehicle ride comfort evaluation are interpreted specifically in the context of stochastic traffic flow involving multiple vehicles. The essential processes in the vehicle comfort analysis are described in details, which include obtaining the whole-body vibration response, frequency weighting the original response and determining the OVTV (overall vibration total value). Taking a prototype long-span bridge and typical traffic flow as example, this study further evaluated the ride comfort level of three types of representative vehicles numerically. Firstly, the numerical study started with the baseline scenario when the vehicles move on the rigid roadway as a moderate traffic flow. Secondly, the effects of incorporating dynamic interactions between the bridge and the vehicles on the ride comfort of the vehicles were then assessed. Finally, the effects of wind excitations on the ride comfort of the vehicles were evaluated. The main findings of the numerical studies can be summarized in the following aspects:

- The frequency weighting is essential for obtaining proper OVTV for ride comfort evaluation. The ISO standard provides four frequency-weighting filters for different axes and at different locations. The filtering effects from different frequency weighting filters vary notably. It is found that the RMS value of vertical and pitching acceleration at the seat location is significantly reduced through the frequency-weighting filter W_k . However, the frequency-weighting filter W_d has little effect on the lateral acceleration response at the seat location.
- When a single vehicle is driven through the long-span bridge, the interaction between the bridge and the designated vehicle is minimal and barely affects the vibration of the vehicle as well as the associated ride comfort condition. When the interactions between the bridge and all the vehicles in the moderate traffic flow are considered, the vehicle acceleration response can be significantly

affected. It was found that the dynamic interactions with the supporting bridge structure considerably affect the ride comfort level of the vehicles.

- The lateral and rolling accelerations of the vehicle are more likely to be influenced in the wind environment than the vertical and pitching accelerations. The acceleration response of the vehicle is significantly influenced by the wind excitations.
- The response at the axes of the seat location account for the majority part of the OVTV for the vehicles. Unless wind excitations are applied, the accelerations in the vertical direction usually remain the larger portion in OVTV than those in other directions. The lateral acceleration only becomes notable in OVTV when wind excitations are applied. Under strong wind excitations, the acceleration in the lateral direction at the seat location may become dominant among all the participating axes.
- When the vehicles move normally on the rigid road in a moderate traffic flow, the ride comfort of the vehicles satisfies the criteria specified in ISO 2631-1. When the vehicles move through the long-span bridge as a part of the traffic flow, the OVTV for the light car and heavy truck are below 0.315 m/s², which is the threshold of ride discomfort as defined in the ISO standard. The OVTV of the light truck is slightly over the discomfort threshold value.
- The light truck is more prone to the ride discomfort issue compared with the other two types of vehicles. This becomes more remarkable when the vehicle travels through the oscillating bridge under wind excitations. Although the ride comfort issues on this particular prototype bridge is not critical, other long-span bridges may exhibit more severe ride comfort issues. With the proposed framework, extensive studies on various long-span bridges from different regions need to be carried out in order to comprehensively investigate the ride comfort issues on long-span bridges.

CHAPTER 9 Reliability Assessment Framework of Long-Span Cable-Stayed Bridge and Traffic System Subjected to Cable Breakage Events^{*}

9.1 Introduction

As an important type of transportation structure to cross major rivers and straits, long-span cablestayed bridges are built around the world due to the engineering advantages, structural efficiency and aesthetic attraction. Stay cables are important load bearing members of a cable-stayed bridge and may experience corrosion, fatigue, accidental and intentional actions, which may cause a reduction of sectional resistance capacity and even lead to possible breakage failure. Due to the critical role of cables among all major components, cable-stayed bridges are required to be designed against single-cable breakage incident (PTI 2007). Most existing cable breakage studies were limited to some scenario-based studies with simplified structural models, loads and simulation methodologies (e.g. Ruiz-Teran and Aparicio 2009; Wolff and Starossek 2010). In recent years, some efforts have been made to develop more realistic load models and advanced analytical tools to study cable breakage. For example, Zhou and Chen (2014a) studied long-span bridges subjected to cable loss under stochastic traffic loads with SAP2000. Recently, a finite element-based advanced simulation strategy was proposed to simulate cable breakage events considering dynamic excitations from both traffic and wind (Zhou and Chen 2015c), which are particularly critical for long-span cable-stayed bridges with slender sections. In the newly proposed simulation framework (Zhou and Chen 2015c), the dynamic coupling effects among the bridge, each vehicle in the stochastic traffic flow and wind excitations are incorporated. Moreover, in addition to the improved characterization of cable breakage process, various sources of geometric and material nonlinearities are considered in the bridge model and the dynamic responses of the bridge and vehicles are obtained through nonlinear iteration (Zhou and Chen 2015c, d).

^{*} This chapter is submitted to a journal in a paper that is currently under review (Zhou and Chen 2015f).

Existing deterministic analyses on the cable breakage events as summarized above indicate that the dynamic response of the bridge and moving vehicles may be significantly affected by several parameters such as wind speed, traffic flow composition, cable breakage instant, duration and process. Considering that uncertainties exist in nature on structures and environmental loads, it is necessary to evaluate the risk posed on the bridge structure and moving vehicles by taking into account various sources of uncertainties associated with cable breakage events. During the past decades, reliability theory has been widely applied on investigating structural performance under various natural and man-made hazards, such as earthquakes and hurricanes etc. (Mackie and Stojadinovic 2001; Li and Ellingwood 2006; Nielson and DesRoches 2007; Kwon and Elnashai 2010). In contrast, there is little, if any, reliability-based work that has been reported on cable breakage incidents. Different from traditional natural or man-made hazards with relatively straightforward characterization of hazard functions, cable breakage is essentially a type of structural member failure incidents due to several possible reasons, such as fire, impact, lighting or structural deterioration, etc. Such unique feature poses some difficulties on investigating this type of important yet complex phenomena by incorporating reliability theory (Zhou and Chen 2015c, d).

Vehicles are often prone to various single-vehicle accidents under some adverse driving conditions, such as crosswind (Chen and Cai 2004; Zhou and Chen 2015b), iced-covered road or complex terrain (Chen and Chen 2010). When a long-span bridge suffers from concurrent single-vehicle accidents of multiple vehicles during an incident, such as cable breakage, serious congestion, traffic disruption and even shutdown of the bridge may occur. During emergency conditions, how much traffic movement can still be accommodated and how soon a bridge can regain the normal transportation functionality directly affect whether an evacuation, if warranted, can be successful, and emergency responders can quickly reach those in need. This is particularly critical for long-span bridges since most of long-span bridges are vital links of evacuation routes and post-hazard emergency responding paths for the whole transportation system in a region. Therefore, the serviceability condition of a long-span bridge under incidents like cable breakage needs to be quantitatively evaluated in order to improve the bridge resilience to the incidents.

It is the goal of this study to develop a holistic reliability assessment framework for cable breakage incidents based on recent advances on deterministic methodology. In addition to bridge structural ultimate limit state, a new bridge serviceability limit state is also proposed by focusing on the overall traffic safety of all vehicles of the traffic flow on the bridge subjected to cable breakage incidents. Uncertainties of those variables that are critical to the bridge and vehicle performance, including cable breakage parameters, dynamic loads from stochastic traffic and wind, are considered. A restrained sampling technique is adopted to obtain a series of structural models with reasonable computational efforts for nonlinear time history analyses. Different from those hazards with straightforward hazard intensity functions, the fragility of the bridge subjected to cable breakage incidents are assessed against some key parameters without specifying the particular hazard event leading to the cable breakage. The influences on the bridge failure probability due to different parameters including the cable breakage duration, breakage process and dynamic loads from stochastic traffic and wind are discussed. Finally, fragility study of the bridge in terms of the proposed bridge serviceability limit state focusing on the overall traffic safety of all vehicles of the traffic flow is made.

9.2 Nonlinear dynamic simulation methodology for cable breakage

9.2.1 Fully-coupled bridge-traffic-wind interaction analysis

The fully-coupled bridge-traffic-wind interaction model is developed using finite element method. The methodology of the fully-coupled bridge-traffic-wind interaction analysis is described in Sec.5.2.

9.2.2 Modeling of geometric nonlinearity on the bridge

The beam-column effect on the bridge girders and pylons due to the presence of large axial force is primarily responsible for the geometric nonlinearity of the beam elements. The geometric nonlinearity of the beam element is considered in this study by updating the geometric stiffness matrix of the element. The sources of geometric nonlinearity for cable elements include cable sag effect, tension-stiffening effect and large displacement effect. Various types of geometric nonlinearity for cable elements are taken into account in the analytical formulation of elastic catenary.

9.2.3 Modeling of material nonlinearity on the bridge

The concentrated plastic hinge approach (McGuire et al. 2000) is used to model the material nonlinearity of beam and cable elements on the bridge. The plastic hinges are applied at the two ends of each element with a hinge length equal to the half-length of the element. Isotropic hardening is introduced to evaluate the post-yield behavior at the hinge location of the element. For the beam elements, the yielding state of the beam element is determined according to the prescribed interacting yielding surface of axial force and bi-axial bending moments at the hinge location.

Different from the interacting plastic hinge for beam elements based on axial force and bi-axial bending moments, the plastic hinge model for cable elements can only bear pure axial force. The cable element is deemed to enter the yielding state if the axial force of cable is greater than the yield force of cable. After plastic hinge forms at one end or both ends of the cable element, the yield force is applied and the elastic modulus of the cable element will become zero at the yielding end of the cable.

9.2.4 *Nonlinear dynamic simulation strategy*

Considering a cable breakage event starting at t_1 second in the total simulation duration of (t_1+t_2) seconds, the cable-loss simulation is conducted following several steps. Firstly, nonlinear static analysis of the bridge is conducted under the bridge self-weight and static wind load to obtain the initial deformed shape of the bridge, which will be used as the static initial state for further dynamic analysis. Secondly, nonlinear dynamic analysis of the bridge under stochastic traffic and wind is conducted for t_1 seconds in order to obtain the nodal axial forces of the breaking cable just before breakage occurs. Thirdly, in each time step during cable breakage process with a total duration of t_{tot} seconds, the relative breakage time is calculated and relative cable loss will be determined based on the cable loss function. The impact nodal force on the cable is determined and applied to the ends of the breaking cable to simulate the force condition during the breakage duration. In the meantime, the cable area is gradually lost over the elapsed breakage time to mimic the change of structural formulation during cable breakage process. Finally,

nonlinear dynamic analysis is conducted for another (t_2-t_{tot}) s to simulate the post-breakage behavior of the bridge. The simulation procedure is demonstrated in the flowchart in Fig. 9.1.

In each step during the simulation process, every bridge member of the model will be evaluated at each iteration step to determine whether it may enter the inelastic range or fail. For the members that yield without failing, the element tangent stiffness matrix will be reduced to the plastic stiffness matrix and the fixed end forces will also be updated. The members will be eliminated from the bridge FE model if they meet the prescribed failure criterion.



Figure 9.1 Flowchart of the simulation steps using nonlinear dynamic simulation methodology for cable breakage events

9.3 Structural reliability assessment framework

9.3.1 Modeling of random variables

It is known that uncertainties associated with structures and environmental loads exist in nature due to various sources, and modeling uncertainties is the initial step for structural reliability analysis. The sources of uncertainties of a long-span cable-stayed bridge subjected to the breakage of stay cables are categorized as five types, which are structural material properties, sectional properties, traffic condition, wind load condition and cable breakage parameters. The structural material properties that are treated as random variables include elastic modulus, mass density and yield strength. The random variables related to sectional properties, such as sectional area and moment of inertia, are considered to be dependent on the size parameters of the section. Therefore as an independent random variable quantified for each section of the bridge girders, pylons and cables, the size factor is defined as the ratio of the actual dimension over the nominal (mean) dimension of the section in the model. The traffic flow involved in the analysis is simulated using the CA model as introduced previously, in which the vehicle instantaneous information in both time and space can be obtained, such as occupied lane, longitudinal location and driving speed. In the process of traffic flow simulation, traffic density and the proportion of each type of vehicles are treated as random variables. For the three types of vehicles, the dynamic properties, such as the mass of vehicle body, mass of wheel axles, stiffness coefficients of upper and lower springs, are also treated as random variables considering variations among different vehicles and associated errors on parameter estimations. The steady-state wind speed is the only random variable that is considered in the present study for wind and the turbulent parameters are treated as deterministic parameters. The previous study by the authors (Zhou and Chen 2015d) has suggested that the dynamic response of the bridge may be significantly influenced by the cable breakage duration, breakage processes and initial states at breakage. In the present study, several parameters related to cable breakage events, including breakage duration, process and initial states, are also treated as random variables.

9.3.2 Nonlinear time history analysis with Latin hypercube sampling

The present study deals with nonlinear dynamic simulation of cable-breakage events on long-span cable-stayed bridges under service load excitations from stochastic traffic and wind. The simulation process involves complex coupled interactions between the bridge and vehicles as well as between the bridge and wind. In addition, various sources of nonlinearity are incorporated into the simulation and nonlinear iteration is involved in the analysis. Owing to the involvement of multiple dynamic loadings from stochastic traffic, wind and cable breakage events, there are inherent uncertainties that are associated with various types of dynamic loads in addition to the structural properties as commonly considered in most related structural reliability analyses. A previous study on deterministic analysis of cable-breakage events (Zhou and Chen 2015d) has suggested that various dynamic loading conditions from stochastic traffic, wind and cable breakage parameters are important for defining a cable breakage event and should be taken into account in the evaluation of bridge dynamic response. Since the dynamic response of the bridge requires nonlinear dynamic analysis with a finite element bridge model, it is impossible to obtain an accurate explicit expression of limit state functions. Furthermore, the nonlinear dynamic analysis of the cable breakage events on the bridge under service loads from traffic and wind requires relatively long time for a single trial and therefore, Monte Carlo simulation will be too time-demanding to become a feasible option for the proposed cable breakage investigation. In the present study, the Latin Hypercube Sampling (LHS) technique, which has been widely adopted for earthquake risk assessment involving complex nonlinear analyses, is employed for the proposed reliability evaluation. With the LHS technique, 100 models of the bridge are generated, each of which involves a combination of the samples for each random variable from the bridge structure, traffic, wind loads and cable breakage parameters.

9.3.3 Defining limit state functions

The breakage of stay cables may induce large internal forces which may further trigger yielding, buckling of bridge members and global instability of the bridge structure. The bridge safety subjected to cable breakage incidents is apparently of utmost importance. In addition, depending on causes, cable breakage may occur at some time instants when considerable traffic may still remain on the bridge. When the bridge undergoes amplified response and rebalance process following cable breakage, the moving vehicles on the bridge may experience increased safety risks (Zhou and Chen 2015c), which should be considered in order to evaluate the possible consequences of cable breakage. Accordingly, two limit states are defined for the bridge: ultimate limit state and serviceability limit state focusing on the overall vehicle safety condition of the traffic flow on the bridge, which will be discussed in the following in details.

9.3.3.1 <u>Ultimate limit state</u>

Since the explicit limit state function cannot be obtained, the limit state function can only be implicitly expressed and determined from nonlinear dynamic analysis. In terms of all random variables, the ultimate limit state function can be expressed in Eq. (9.1).

$$f(X_1, X_2, ..., X_n) = 0$$
(9.1)

in which, f is the implicit function of the structural performance; $X_1, X_2, ..., X_n$ are the random variables defined in the analysis. When the value of limit state function is smaller than 0, i.e., $f(X_1, X_2, ..., X_n) < 0$, the failure event of the bridge exceeding the ultimate limit state is considered to occur. There are three initial failure modes corresponding to the ultimate limit state of the bridge-traffic system, which are summarized in the following:

(1) Failure mode I: Initial failure due to exceedance of material ultimate strength. Plastic hinges are formed on a major bridge component, for which the material yields and exhibits inelastic behavior. When the material ultimate strength is reached, structural failure is deemed to occur.

(2) Failure mode II: Initial failure due to structural instability. Local buckling may first occur on the compression-bearing members of the bridge girders and pylons, which may further incur the global instability of the bridge structure. From the time-history response perspective, the instability failure mode is featured by the rapidly divergent dynamic response.

(3) Failure mode III: Initial failure due to aerodynamic instability. This is a possible failure pattern for slender long-span bridges, especially for those with relatively bluff cross sections associated with relatively poor aerodynamic performance. Different from structural instability, in which local buckling typically occurs at the compression-bearing structural members, the aerodynamic instability usually exhibits divergent wind-induced bridge response when wind speeds reach a certain critical value (i.e. flutter critical speed).

Because of possible risks of progressive failures, the actual failure behavior of the bridge may exhibit as mixed or sequential processes of different failure patterns as time progresses. For instance, the instability in Failure Modes II and III may lead to the divergence of the bridge response and also induce the formation of plastic hinges on structural members. However, since the initial failure is due to structural or aerodynamic instability, the actual failure mode should belong to Type II or III. For most long-span bridges with streamlined cross sections, the failure mode III is unlikely to occur unless the wind speed is very high at the time when the cable breakage occurs at the same time. When wind speed is high, the increase of wind speed may not induce large increase of internal forces on the structural members, rather it may significantly increase the chance of instability caused by unsymmetrical aerodynamic stiffness and damping matrices. When the wind speed is approaching the critical value leading to aerodynamic instability, the breakage of stay cables may act as additional impact force on the bridge which may directly trigger aerodynamic instability, i.e., flutter. The breakage of stay cables may induce large unbalance force on the bridge pylon columns and cause local buckling at the bridge pylons, followed by global instability of the bridge structure. Although wind-induced or wind-rain-induced cable failure is possible, there is no strong tie between cable failure and high wind speed to justify the simultaneous occurrence of cable loss and very high wind speed. Therefore, the study considers the cable breakage events independent of the specific traffic and wind situations. As compared to aerodynamic instability in Failure Mode III, the initial failure due to structural instability in Failure Mode II is a more common type of failure mode for long-span cable-stayed bridges subjected to cable breakage.

9.3.3.2 Serviceability limit state

For cable breakage events, the dynamic impact from breakage of stay cables may not only significantly influence the dynamic behavior of the bridge but also cause amplification on the vehicle response, which may pose risk on the vehicle riding safety and comfort (Zhou and Chen 2015c, d). In a cable breakage event, vehicle comfort issue may not become significant because of the short duration of uncomforting driving experience following cable breakage. Comparatively, vehicle safety issue is more critical for cable breakage events on long-span bridges since it could mean life or death for drivers involved into accidents as well as people stranded on a failing bridge because of serious congestion following accidents. Chapter 7 (Zhou and Chen 2015b) has identified three common vehicle accident types, which are lift-up, side-slip and yawing, among which lift-up accident is more common in the windy

environment. Side-slip and yawing accidents are usually secondary following lift-up accidents after the wheels lose contact with the road surface. For a specific vehicle in the present study, a lift-up accident is considered to occur if at least one wheel set is lifted up at one side (Zhou and Chen 2015b), which is expressed in Eq. (9.2).

$$V_{il(R)}^{t}(t) > 0, i = 1, 2 \text{ or } 3$$
 (9.2)

in which, $V_{iL(R)}^{t}(t)$ is the total vertical contact force at the left (right) side of the *i*th wheel set, and is expressed as the summation of the static vertical contact force $V_{iL(R)}^{s}$ and dynamic vertical contact force $V_{iL(R)}^{d}(t)$, as expressed in Eq. (9.3a).

$$V_{iL(R)}^{t}(t) = V_{iL(R)}^{s} + V_{iL(R)}^{d}(t)$$
(9.3a)

$$V_{iL(R)}^{s} = K_{lzL(R)}^{i} \dot{h}_{L(R)}^{i} + C_{lzL(R)}^{i} \dot{h}_{L(R)}^{i}$$
(9.3b)

$$V_{iL(R)}^{d}(t) = K_{izL(R)}^{i}\overline{h}_{L(R)}^{i}(t) + C_{izL(R)}^{i}\overline{h}_{L(R)}^{i}(t)$$
(9.3c)

where $h_{L(R)}^{i}$ and $\dot{h}_{L(R)}^{i}$ are the physical vertical displacement and velocity of the mass block of the *i*th wheel set under gravity and static wind loads on the left (right) side, respectively; $\bar{h}_{L(R)}^{i}(t)$ and $\bar{h}_{L(R)}^{i}(t)$ are the vertical displacement and velocity of the mass block relative to the bridge displacement and velocity response at bridge contact point for the *i*th wheel set on the left (right) side under dynamic loads at time *t*, respectively; $K_{lzL(R)}^{i}$ and $C_{lzL(R)}^{i}$ are the spring and damping coefficients of the lower spring and damper at the *i*th wheel on the left (right) side, respectively. The contact force on the bridge due to the presence of vehicles is usually downwards with a negative value in the study. Based on the point-contact assumption between the bridge and vehicles, the positive vertical contact force only occurs when the vehicle wheel loses contact from the bridge surface.

In the present study, there are a number of vehicles on the bridge at any time in the simulated stochastic traffic. In order to evaluate the overall bridge serviceability affected by possible vehicle

accidents, the proportion of the vehicles exceeding the vehicle safety criteria on the bridge at any time instant during the simulated period is quantified. For each experiment, the maximum proportion value of the vehicles exceeding vehicle safety criterion during the time period after cable breakage occurs will be determined and used to evaluate the serviceability limit state of the bridge. When the maximum unsafe vehicle proportion exceeds the threshold of the prescribed limit, the serviceability limit state of the bridge is deemed being exceeded, as shown in Eq. (9.4).

$$x_{max} - x_{lim} = 0$$
 (9.4)

in which, x_{max} and x_{lim} are respectively the maximum and limit state values of the vehicle proportion exceeding vehicle safety criterion.

9.4 Uncertainty modeling of prototype long-span cable-stayed bridge and vehicles

9.4.1 Prototype long-span cable-stayed bridge

The prototype cable-stayed bridge in the present study has a total length of 836.7 m, including a main span, two side spans and two approach spans as shown in Fig. 9.2. The cable-stayed bridge has a bridge deck with a steel twin-box cross-section, which has a width of 28 m and a height of 4.57 m. The two steel pylons have A-shaped cross-sections with a height of 103.6 m. The bridge superstructure is supported by the cross beam of bridge pylons at the pylon locations and the bridge piers at the side spans with longitudinally sliding bearing supports. The windward stay cables connected to the west pylon are labeled as Cable A to F as indicated in Fig. 9.2.


Figure 9.2 3-D sketch of the prototype long-span cable-stayed bridge

9.4.2 Prototype vehicles in traffic flows

The travelling vehicles in this study are classified into three types: heavy truck, light truck and light sedan car. The dynamic parameters for each type of vehicles involved in the present study include mass, mass moment of inertia, stiffness coefficients and damping coefficients. Important dynamic properties will be defined in the following section as random variables. Other dynamic parameters and the dimension parameters for each type of vehicles can be referred to Sec. 5.4.2.

9.4.3 Modeling of random variables

9.4.3.1 Statistical variation of structural parameters

Uncertainties associated with structural properties and capacities arise from variations of material strength, section properties, measurements and manufacturing processes. For a cable breakage event, structural properties, such as mass density and yield strength, are important to the post-breakage behavior of the bridge, especially when the bridge experiences redistribution of the loads among members after cable breakage occurs. During the dynamic breakage process, the ability of the bridge structure to withstand large unbalanced loads and breakage impact loads mainly depends on load bearing capacities of the structural components, which are closely related to the material strength and sectional properties of the component. In the present study, elastic modulus and mass density of the steel material are assumed to follow a normal distribution. The yield strength is assumed to follow a lognormal distribution with

different parameters for the steel member in bridge girders and pylons and the tendons in bridge stay cables. The section size factor is assumed to follow a normal distribution in terms of different random variables for the bridge girder, pylon and cables. The size, area, plastic modulus and moment of inertia of the section are obtained by multiplying the corresponding mean value with the section size factor to the first, second, third and fourth order, respectively. The parameters of the random variables (X_1 - X_7) for the bridge structure as well as the probability distribution types are listed in Table 9.1. For the random variables following a normal distribution, Parameter¹ and Parameter² are the mean value and coefficient of variance, respectively. For the random variables with lognormal distribution, Parameter¹ and Parameter² are the logarithmic mean value and logarithmic coefficient of variance, respectively.

Variable	Structure	Distribution	Parameter ¹	Parameter ²
Elastic modulus, X_l	Girder, pylon, cable	Normal	$2{\times}10^{11}\text{N/m}^2$	0.15
Mass density, X_2	Girder, pylon, cable	Normal	7627 kg/ m ³	0.05
Yield strength, X_3 - X_4	Girder, pylon	Lognormal	$3.5 \times 10^8 \text{N/m}^2$	0.15
	Cable	Lognormal	$1.5 \times 10^9 \text{N/m}^2$	0.05
	Girder	Normal	1.0	0.05
Size factor, X_5 - X_7	Pylon	Normal	1.0	0.05
	Cable	Normal	1.0	0.05

Table 9.1 Probability distributions and associated parameters for the bridge structure

9.4.3.2 <u>Random variables in the stochastic traffic flow</u>

Three types of vehicles are involved in the simulation of stochastic traffic flow, which are heavy truck, light truck and light car. The random variables in the stochastic traffic flow are those related to vehicle composition of the traffic flow, i.e., proportion of each type of vehicles, and the dynamic parameters for each type of vehicle. The vehicle proportions are treated as random variables fluctuating around nominal mean values since the traffic flow has different vehicle composition during different time periods. Considering that the summation of the proportion for each type of vehicles should be unity, only proportions for two types of vehicles are treated as independent random variables. In the present study,

the proportion for the heavy trucks and light trucks are assumed to be the independent random variables following a normal distribution. The mean values of the proportion of the heavy truck and light truck to the total vehicle number in the traffic flow are selected as 20% and 30%, respectively, following an existing study (Chen and Wu 2010). The coefficients of variance for the vehicle proportion of the heavy truck and light truck are assumed to be 0.3. The light car proportion in each experiment is determined by subtracting the proportions of the heavy truck and light truck from 1.0.

In addition to the traffic flow composition from different types of vehicles, major vehicle dynamic parameters are also treated as random variables, including vehicle body mass, wheel set mass, stiffness coefficients and damping coefficients. These dynamic parameters are assumed to follow a uniform distribution within ranges around respective nominal values. Uniform distribution has maximum uncertainty and can give conservative results when maximum and minimum values of the random variable are known. The probability distributions and associated parameters for the traffic flow composition and vehicle dynamic parameters are summarized in Table 9.2. Parameter¹ and Parameter² for the random variables with a uniform distribution refer to the lower and upper bounds of the varying range, respectively.

Variable	Vehicle type	Distribution	Parameter ¹	Parameter ²
Vehicle proportion, X_8	Heavy truck	Normal	20 %	0.30
Vehicle proportion, X ₉	Light truck	Normal	30 %	0.30
Mass of rigid body 1, X_{10}	Heavy truck	Uniform	3000 kg	5000 kg
Mass of rigid body 2, X_{II}	Heavy truck	Uniform	9375 kg	15625 kg
Axle mass 1, X_{12}	Heavy truck	Uniform	333 kg	407 kg
Axle mass 2, X_{I3}	Heavy truck	Uniform	1125 kg	1375 kg
Axle mass 3, X_{14}	Heavy truck	Uniform	990	1210
Upper stiffness of axle 1, X_{15}	Heavy truck	Uniform	29.75 kN/m	40.25 kN/m
Upper stiffness of axle 2, X_{16}	Heavy truck	Uniform	213 kN/m	288 kN/m
Upper stiffness of axle 3, X_{17}	Heavy truck	Uniform	340 kN/m	460 kN/m

Table 9.2 Probability distributions and associated parameters for the traffic flow parameters

Lower stiffness of axle 1, X_{18}	Heavy truck	Uniform	183 kN/m	247 kN/m
Lower stiffness of axle 2, X_{19}	Heavy truck	Uniform	334 kN/m	452 kN/m
Lower stiffness of axle 2, X_{20}	Heavy truck	Uniform	417 kN/m	564 kN/m
Mass of rigid body, X_{21}	Light truck	Uniform	4875 kg	8125 kg
Axle mass, X_{22}	Light truck	Uniform	720 kg	820 kg
Upper and lower stiffness, X_{23}	Light truck	Uniform	205 kN/m	295 kN/m
Mass of rigid body, X_{24}	Light car	Uniform	1200 kg	2000 kg
Axle mass, X_{25}	Light car	Uniform	142 kg	174 kg
Upper and lower stiffness, X_{26}	Light car	Uniform	51 kN/m	69 kN/m

9.4.3.3 <u>Random variables in the turbulent wind simulation</u>

The turbulent wind speed is simulated as a multi-variate stationary random process on the bridge. Kaimal's spectrum is chosen as the target wind spectrum in both along-wind and vertical directions. Two parameters in the Kaimal's spectrum are treated as random variables following a uniform distribution, which are listed in Table 9.3.

Table 9.3 Probability distributions and associated parameters for turbulent wind simulation

Variable	Load	Distribution	Parameter ¹	Parameter ²
Height above ground, X_{27}	Turbulent wind	Uniform	45 m	55 m
Ground roughness, X ₂₈	Turbulent wind	Uniform	0.025 m	0.035 m

9.4.3.4 Random variables regarding cable breakage events

In Chapter 4 and 6, the authors have concluded that the bridge extreme response during a cable breakage event may be very different when the breakage starts at different time instants. The studies also showed that the bridge extreme response is closely related to the instantaneous position of a bridge in the motion cycle at the breakage instant. Therefore, the breakage start time is chosen as a random variable, which is uniformly distributed in a bridge motion cycle. The motion cycle is selected around 6.0 s so that the bridge vibrates for about 6.0 seconds and 9.0 seconds before and after breakage occurs, respectively.

The total simulation time is chosen as 15 seconds in order to balance the simulation needs to generate sufficiently long time-history response and also the required computational costs. In addition to the breakage start time, there are also two parameters that have been proven to significantly influence the dynamic response of the bridge subjected to a cable breakage event, which are the breakage duration and breakage process. There is currently lack of experimental or field data to exactly define the breakage duration and process as well as their distributions for the cable breakage events due to different causes. Without defining distribution functions, the present study chooses to investigate structural fragility corresponding to several selected values of breakage duration and process.

9.5 Structural fragility analysis due to cable breakage events

Fragility curves or surfaces are conditional probabilities of the bridge exceeding particular limit states under different levels of defined intensity measures. Bridge seismic fragility analysis has been popularly applied in the field of seismic risk assessment. In bridge seismic fragility analysis, the intensity measures are usually chosen as peak ground acceleration, spectral acceleration or arias intensity, etc. The structural fragility concept is adopted in the present study to investigate the conditional probability of the bridge exceeding different limit states during cable breakage events characterized by different levels of intensity measures. As discussed earlier, due to the uniqueness of cable breakage events, the intensity measures in the present study are chosen as the breakage duration, breakage process, steady-state wind speed and vehicle density in the traffic flow based on the deterministic analysis results in Chapters 4 and 6.

A preliminary reliability analysis is conducted on the bridge considering the breakage for each of the single cable from A to F in order to identify the most unfavorable breakage event among all the single cables. Through a Latin Hypercube sampling technique, a series of 20 experiments involving sample values of all random variables are conducted to evaluate the structural reliability of the bridge subjected to sudden loss of a single cable. The failure probability is estimated as the ratio of the number of failure events to the number of total experiments, as shown in Eq. (9.5).

$$P_f = \frac{\text{No. of failure events}}{\text{No. of total experiments}}$$
(9.5)

In the preliminary reliability analysis, the abrupt breakage is assumed and the breakage of the stay cable takes one time step to complete. The steady-state wind speed is assumed to be 10 m/s. The traffic flow is chosen to be free flow with a vehicle density of 12 vehicles/km/lane. It is found that the failure probabilities corresponding to the ultimate limit state of the bridge are around 0.05, 0.10, 0.05, 0.05, 0.10, and 0.25 for breakage of Cable A, B, C, D, E and F, respectively. The breakage of stay cable F may pose much larger risk of the bridge to reach ultimate limit state than the breakage of other stay cables. In the following bridge fragility analysis, the breakage event of Cable F is conducted in order to show clearer trends of bridge fragility subjected to several different intensity measures of the cable breakage events.

The structural fragility is firstly evaluated in terms of the bridge ultimate limit state. The bridge serviceability limit state is also evaluated if vehicles are also involved in the analysis, which is determined based on the vehicle performance exceeding vehicle safety criterion. The whole reliability assessment methodology for the present study includes three stages, which are (1) defining random variables, (2) Latin hypercube sampling, and (3) structural fragility analysis. The flowchart for each of the stages is demonstrated in Fig. 9.3.



Figure 9.3 Flowchart of the methodology for the structural fragility analysis of the bridge subjected to cable breakage events

9.5.1 Structural fragility analysis with respect to cable breakage parameters

The structural fragility of the bridge subjected to the breakage of stay cables is investigated with respect to cable breakage parameters, including breakage duration and breakage process. Several discrete breakage durations are selected in the range between 0 and 3.0 s with an interval of 0.5 s. The upper bound of the breakage duration range is selected as a value over 2.5 s, which is the fundamental period of the bridge. During the trial analysis, the bridge extreme response varies significantly when the breakage duration is close to zero. Therefore, in addition to the discrete values at the interval of 0.5 s, several more

durations in the range between 0 and 0.2 s are also investigated in the study, which are 0.05 s, 0.10 s, 0.15 s and 0.20 s at the interval of 0.05 s.

The cable area loss function f_{rel} is time-dependent and defined as the ratio of lost cable cross-section area to total cable cross-section area. Based on some existing studies (Ruiz-Teran and Aparicio 2009; Zhou and Chen 2015b), the present study assumes the cable area loss function to be an exponential function of the relative breakage time t_{rel} , which is defined as:

$$f_{rel}(t_{rel}) = t_{rel}^{a} \tag{9.6}$$

where t_{rel} is defined as the ratio of the elapsed time t_{elap} after breakage starts over the total breakage time duration t_{total} . *a* is the exponential factor. By changing the exponential factor *a*, the breakage may occur quickly at one end of the time period during the breakage. When *a* is set to be 0, the breakage process indicates an abrupt breakage during which the cable breaks completely at the beginning of breakage duration. When *a* is set to be 1, the function is a linear breakage process indicating that the breakage occurs uniformly during the breakage duration. In the current study, the exponential factor is chosen as several representative values, which are 0.005, 0.05, 0.2, 0.5 and 1.0. The cable area loss curves corresponding to the five representative exponential factors are demonstrated in Fig. 9.4.



Figure 9.4 Cable area loss curves corresponding to the five representative exponential factors

For each combination of different breakage durations and processes, 100 combinations of random variables are generated using Latin Hypercube sampling and 100 experiments of nonlinear dynamic analysis are conducted considering the breakage of Cable F. It is assumed that no stochastic traffic and wind loads are present during the simulation period and breakage of Cable F starts at static initial state in each analysis. Since no vehicle is involved in the analysis, only failure probabilities corresponding to the ultimate limit state of the bridge are obtained. The failure probabilities of the bridge corresponding to ultimate limit state are given in Fig. 9.5 considering the breakage of Cable F with different breakage durations and processes.

It is shown in Fig. 9.5 that the failure probability has the largest value irrespective of the breakage processes when the breakage of stay cable completes abruptly within 0.04 s, which is also the integration time step in the analysis. When the breakage time is at a certain value, the failure probability of the bridge generally increases nonlinearly as the breakage process approaches the abrupt breakage process. The increase is more notable when the breakage durations are smaller than 1.50 s and the breakage process is closer to the abrupt breakage process. For the cases with breakage durations equal to and over 1.50 s, the failure probability is zero for the breakage process still exists for longer breakage durations, although the results are not displayed because no failure events actually occur when the breakage processes are closer to the linear breakage process, i.e., with an exponential factor of 0.2, 0.5 and 1.0. When the breakage duration is equal to or over the fundamental period of the bridge, i.e. 2.5 s, the failure probability of the bridge will not change with breakage durations, irrespective of breakage processes.

For certain cable breakage processes, the failure probability of the bridge decreases as the breakage duration increases. However, the failure probability will become stable after the breakage duration is larger than a certain value, which varies with different breakage processes. The failure probability drops drastically with respect to breakage duration when the breakage duration is shorter than 0.2 s. This trend becomes more and less remarkable when the breakage process approaches the linear breakage process and

abrupt breakage process, respectively. It is concluded that both the breakage duration and breakage process may influence the bridge failure probability significantly under certain premises.



Figure 9.5 Failure probabilities of the bridge with respect to different breakage durations and processes

9.5.2 Structural fragility analysis with respect to wind and traffic loads

In addition to the breakage parameters, stochastic traffic and wind loads may also influence the dynamic response of the bridge in the cable breakage events. In this section, the cable breakage is assumed to be an abrupt breakage, i.e., occur within 0.04 s, considering that this is the most disadvantageous breakage duration as indicated in the previous section. The structural fragility analysis of the bridge subjected to the breakage of stay cables is conducted with respect to the traffic flow density and steady-state wind speed. Four wind conditions are considered with steady-state wind speeds of 0, 10, 20 and 30 m/s, respectively. Traffic flow densities are considered to be 0, 11, 22 and 32 vehicles/km/lane, representing no, light, moderate and busy traffic flow condition, respectively. Turbulent wind speed corresponding to each steady-state wind speed is simulated as multi-variate random processes and the simulation is repeated for 100 times to take into account uncertainties. For stochastic traffic flow, 100 patterns of traffic flow are simulated for the light, moderate and busy traffic density, respectively. 100 experiments are conducted to evaluate the structural fragility for each of breakage scenario involving different combinations of traffic flow and wind.

9.5.2.1 <u>Typical dynamic response for the bridge</u>

The vertical time history responses at the mid-span of the windward girder are given in Fig. 9.6, in which the samples of experiments haven't led to structural failure. The four lines in Fig. 9.6 are corresponding to the scenarios under busy traffic flow when the steady-state wind speeds are 0, 10, 20 and 30 m/s, respectively. For each of the comparative cases, the breakage starts at 6.96 s and the same pattern of busy traffic flow is applied.



Figure 9.6 Vertical displacement at the mid-span windward girder under different wind conditions

Before the cable breakage starts, the bridge vibrates normally under the excitations from wind and traffic. After the breakage starts at 6.96 s, the bridge response experiences an abrupt amplification and decays gradually. It is seen that the extreme response of the bridge increases nonlinearly as the wind speed increases. It is mainly because the wind forces on the bridge, including steady-state wind force, self-excited force and buffeting force, are formulated based on the square of steady-state wind speed. It is inferred that a higher wind speed may pose higher risk for the bridge to exceed the ultimate limit state during a cable breakage event.

In the experiments with different combinations of wind speed and traffic flow density, the breakage of Cable F in most cases will not induce the structural failure by exceeding the bridge ultimate limit state. For some experiments with certain combinations of random variables from the structure and dynamic loadings, the breakage of the stay cable may lead to the divergent response of the bridge which definitely exceeds the bridge ultimate limit state. A typical structural failure case is given and the divergent vertical

displacement at the mid-span of the windward girder is shown in Fig. 9.7 for demonstration. After the abrupt failure of the stay cable, plastic hinges are found to firstly develop at the top and bottom of leeward pylon column. Soon after that, several elements on the windward pylon column enter the yielding state successively. As the simulation analysis proceeds, plastic hinges also form at the girder elements. Later, the divergent displacement response is found to occur throughout the bridge structure and the structural instability is incurred as evidenced by rapidly divergent response. Nonlinear dynamic analysis continues for another several steps and cannot proceed further due to the large residual force vector after divergence occurs.

It is seen that for the prototype long-span cable-stayed bridge, due to the presence of the steel pylons, the local yielding on the pylons is the initial trigger event for the successive structural instability and global collapse of the whole bridge structure. It is concluded that the dominant failure modes for the prototype bridge subjected to cable breakage are essentially Mode I and II following the initial failure of the pylon elements. For the girder section of the prototype bridge, the failure mode III with aerodynamic instability is not likely to occur after the breakage of stay cables especially when the wind speed didn't reach the critical flutter speed of the bridge. If structural instability were indeed caused by large unsymmetrical aerodynamic matrices, the plastic hinges usually would have formed after rather than before the divergence of response.



Figure 9.7 Divergent vertical displacement at the mid-span of the windward girder

9.5.2.2 <u>Typical dynamic response for the vehicles</u>

The change of vehicle response after the cable breakage occurs largely depends on the vehicle location on the bridge and vehicle driving motion at breakage. Therefore, different vehicles in the traffic flow exhibit different response patterns during a cable breakage event even for the same vehicle type. When a vehicle is driven at a high speed at around the mid-span of the bridge main span or side span at the breakage instant, the vehicle response is more likely to be influenced by the breakage event. A representative light truck is selected for the vehicle response demonstration. The vertical displacement of the vehicle body of the representative vehicle is given in Fig. 9.8 under the same pattern of busy traffic when the steady-state wind speed is 0, 10, 20 and 30 m/s, respectively. It is seen that the vehicle response is greatly excited after the cable breakage occurs, which is more notable when the wind speed is relatively low. When the wind speed gets very high, i.e., 30 m/s, the vehicle response may already be very large before the cable breakage and therefore comparatively, the excitation due to the cable breakage impact may become less significant. The vehicle response increases nonlinearly with the increase of the wind speed, which holds true both before and after cable breakage occurs. However, the response difference among different comparative cases after cable breakage occurs is much less significant than that before cable breakage occurs.



Figure 9.8 Vertical displacement of the vehicle body for the representative vehicle

The vertical contact force time histories at one wheel side of the same representative vehicle are obtained and demonstrated in Fig. 9.9 for the comparative cases. When the vehicle vertical contact force

on the ground becomes positive, the vehicle is lifted up at one wheel side and the safety limit state of the vehicle is exceeded. It is seen in Fig. 9.9 that the vehicle vertical contact force may be greatly amplified due to the cable breakage event in each of the four cases. The vehicle vertical contact force may also be excited when the wind loads are applied suddenly, i.e., at the start time of the simulation period. When the wind speed is not higher than 10 m/s, the lift-up accident may only appear after the cable breakage occurs. When the wind speed is equal to and higher than 20 m/s, the lift-up accident may occur at around the instant when the wind loads are applied suddenly as well as the instant after the cable breakage occurs. However, due to the superimposing nature of the cable breakage impact force, the vertical contact force is still more unfavorable after cable breakage occurs under higher wind speeds than that when wind loads are initially applied.



Figure 9.9 Vertical contact force at the one side of wheel for the representative vehicle

9.5.2.3 <u>Structural fragility of the bridge corresponding to ultimate limit state</u>

For each combination of the traffic flow density and steady-state wind speed, the samples of random variables are generated and grouped using Latin Hypercube Sampling technique. Failure events corresponding to the bridge ultimate limit state are identified and the failure probabilities for each combination of traffic density and wind speed are calculated. The structural fragility surface with respect to traffic density and wind speed is obtained and demonstrated in Fig. 9.10.

It is seen in Fig. 9.10 that structural failure probability doesn't have much variation for the same traffic flow density when the wind speed is not larger than 20 m/s. When the wind speed is over 20 m/s,

the failure probability is significantly increased as the wind speed increases for the same traffic flow density. Compared to the steady-state wind speed, traffic flow density has larger influence on the bridge failure probability. As shown in Fig. 9.10, the bridge failure probability increases significantly as the traffic density increases for the same wind speed. The values of the failure probability with respect to the breakage scenarios with different traffic flow densities and steady-state wind speed are listed in Table 9.4. Compared with the failure probability in the breakage case with no traffic and wind, the application of only wind excitations at wind speed of 30 m/s increases the bridge failure probability by 18.8 %. When the traffic flow density is not larger than 22 vehicles/km/lane, the increase of wind speed will not increase the bridge failure probability as long as the wind speed is not larger than 20 m/s. For the busy traffic flow with a density of 33 vehicles/km/lane, the failure probability of the bridge increase by 5.0 % when the wind speed increases from 10 m/s to 20 m/s. Compared with the bridge failure probability without considering traffic and wind excitations, the sole excitation from traffic at a density of 11, 22 and 33 vehicles/km/lane increases the failure probability by 12.5 %, 18.7 % and 25 %, respectively. The most disadvantageous scenario occurs when the traffic flow density and wind speed are both the largest among all scenarios. Specifically, the failure probability of the bridge in the cable breakage event under the traffic flow at a density of 33 vehicles/km/lane and wind speed of 30 m/s increases by 81.2 % compared with the base case without traffic and wind excitations. It is concluded that traffic and wind excitations may significantly increase the bridge failure probability during the cable breakage event. The dynamic excitations from traffic and wind play an important role in the overall risk of the bridge subjected to cable breakage events.



Figure 9.10 Structural fragility surface for the ultimate limit state of the bridge with respect to traffic density and wind speed

Table 9.4 Failure probability of the bridge subjected to cable breakage with respect to different traffic and

Failure probability		Steady-state wind speed			
		None	10 m/s	20 m/s	30 m/s
	None	0.16	0.16	0.16	0.19
Traffic density11 vehicle/km/lane22 vehicle/km/lane33 vehicle/km/lane	0.18	0.18	0.18	0.23	
	22 vehicle/km/lane	0.19	0.19	0.19	0.24
	33 vehicle/km/lane	0.20	0.20	0.21	0.29

wind conditions

9.5.2.4 Structural fragility of the bridge corresponding to serviceability limit state

In addition to the bridge ultimate limit state, the bridge fragility corresponding to serviceability limit state is also investigated. As discussed earlier, a vehicle is considered to exceed the vehicle safety criterion when the maximum vertical contact force after cable breakage occurs becomes positive at certain time instants. The vehicles in which vehicle safety criterion is exceeded are identified in the experiments for each combination of different traffic flow densities and steady-state wind speeds. The cable breakage experiments incurring bridge failure corresponding to ultimate limit state are excluded from serviceability evaluation considering that all the vehicles on the bridge will be unsafe if the bridge exceeds ultimate limit state. The proportion of those vehicles exceeding vehicle safety criterion out of the total vehicles on the bridge is calculated in each time step after cable breakage occurs for each participating experiment. The maximum proportion of unsafe vehicles is further obtained for each participating experiment and the cumulative distribution functions are obtained from the maximum proportion values for different breakage scenarios. The probabilities of exceedance are evaluated corresponding to the prescribed serviceability limit states in the breakage scenarios under different traffic and wind loads. The fragility curves can be obtained as the exceedance probabilities conditioned on traffic density and wind speed corresponding to different serviceability limit states. The assessment methodology for bridge fragility corresponding to serviceability in the cable breakage events is demonstrated in Fig. 9.11.



Figure 9.11 Methodology flowchart for structural fragility analysis corresponding to bridge serviceability The cumulative distribution function curves for the proportion of unsafe vehicles in traffic flows with different densities are demonstrated in Figs. 9.12a, b, c and d under wind speed of 0, 10, 20 and 30 m/s, respectively. When wind speed is zero, the unsafe vehicle proportion in the cable breakage event is at most 0.21, 0.18 and 0.22 for free, moderate and busy traffic flow, respectively. The unsafe vehicle proportion is at least 0.058, 0.065 and 0.081 for free, moderate and busy traffic flow, respectively. The curves don't show significant variations among different traffic flow conditions when wind load is not involved, with the scenario under busy traffic flow slightly disadvantageous than that under moderate and light traffic flow conditions.



(a) No wind speed



(b) Wind speed of 10 m/s



(d) Wind speed of 30 m/s

Figure 9.12 Cumulative distribution function and probability of exceedance of unsafe vehicle portion

When wind excitations are applied in addition to traffic flow in the cable breakage events, the proportion of unsafe vehicles increases nonlinearly with wind speed compared with that without wind excitations. This is especially remarkable for the case with light traffic flow and becomes even more remarkable as wind speed gets higher. The nonlinear increase pattern of the unsafe vehicle proportion with respect to wind speed is mainly due to the square of wind speed in the expression of wind forces on vehicles. Taking the light traffic flow case for instance (Figs. 9.12a-d), the maximum values of unsafe vehicle proportion are 0.21, 0.25, 0.44 and 0.90 for wind speed of 0, 10, 20 and 30, respectively. Unlike

the breakage cases without wind excitations, the proportion of unsafe vehicles under same wind excitations is notably larger for the case with light traffic flow than the case with moderate and busy traffic flow. As wind speed increases, the difference of the proportion of unsafe vehicles in the case with light traffic flow becomes more remarkable than those in the case with moderate and busy traffic flow. When wind speed is low, e.g., at around 10 m/s, the bounds of the range of the unsafe vehicle proportion for the busy traffic flow are slightly larger than the corresponding values for the moderate traffic flow. As wind speed goes higher, the vehicles become less safe in the moderate traffic flow than in the busy traffic flow, as evidenced by the gradually increased proportion of unsafe vehicles. Such phenomenon is understandable from the perspective of traffic safety under windy conditions. Due to the relatively lower number of total vehicles for lighter traffic flow, the vehicles are generally driven at higher speeds in the lighter traffic flow than that in the heavier traffic flow. The higher driving speed will lead to larger resultant wind forces on the vehicles which are more likely to cause lift-up and other vehicle accidents.

To obtain the general trend of the unsafe vehicle proportion with respect to traffic density and wind speed, the average value of unsafe vehicle proportions is obtained from all experiments for the different scenarios, which is listed in Table 9.5 and shown in Fig. 9.13. It is seen that the vehicle safety performance for different traffic densities remain similar when there is no or very mild wind. The vehicles become less safe in the cable breakage events as wind speed gets higher and the traffic flow is lighter. For the light traffic flow case, the unsafe vehicle proportions at the wind speed at 10, 20 and 30 m/s increase by 26%, 82% and 425% as compared to that of the case without wind excitations. Compared with the traffic density, wind speed plays a more important role in the vehicle safety risk in the cable breakage events.



Figure 9.13 Average proportion of vehicles exceeding vehicle safety criterion in different breakage

scenarios

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Vehicle proportion		Steady-state wind speed			
		No wind	10 m/s	20 m/s	30 m/s
	11 vehicle/km/lane	0.137	0.172	0.250	0.762
Traffic density	22 vehicle/km/lane	0.123	0.150	0.206	0.592
	33 vehicle/km/lane	0.137	0.150	0.179	0.400

In addition to the bridge fragility corresponding to bridge ultimate limit state, the bridge fragility corresponding to the prescribed serviceability limit state is also investigated. The limit states are often subjectively defined from simulation results and field studies based on a consensus of stakeholders. There is currently little research on the definition on serviceability limit states of bridges in terms of transportation capacity under incidents. As an explorative study to investigate the bridge serviceability under extreme events from cable breakage, three serviceability limit states I, II and III are defined for demonstration corresponding to light, moderate and severe damage states of bridge serviceability. In this

study, the limit values x_{lim} are selected as 0.20, 0.40 and 0.60 for serviceability limit state I, II and III, respectively. It is noted that these x_{lim} values are selected for demonstration purposes when certain threshold values are not yet available.

The probability of exceedance for serviceability limit states I, II and III of the bridge in the cable breakage event is demonstrated with respect to different wind speeds in Figs. 9.14 a, b and c for light, moderate and busy traffic flow condition, respectively. It is seen that the probability of exceedance of the bridge exceeding serviceability limit state increases nonlinearly with the increase of wind speed for certain traffic flow condition. As wind speed gets higher, a larger slope of the fragility curve is usually observed, indicating a larger rate of increase for the probability of exceedance to serviceability limit state. This is especially remarkable for the curves corresponding to serviceability limit state II and III when the wind speed is equal to or higher than 20 m/s. For serviceability limit state I, the probability of exceedance remains below 6.25 % for the three traffic conditions when no wind loads are applied. When wind speed reaches 30 m/s, the probability of exceedance for serviceability limit state I will become 1.0. For serviceability limit state II, the probability of exceedance is zero under the three traffic flow conditions when wind speed is not larger than 10 m/s. The probability of exceedance for serviceability limit state II remains zero or a small value when the wind speed reaches 20 m/s while it significantly increases when the wind speed is higher than 20 m/s. Similar findings are observed for serviceability limit state III but the probability of exceedance for serviceability limit state III is lower than that to serviceability limit state II, especially for the moderate and busy traffic flow condition when the wind speed is over 20 m/s. For the prescribed serviceability limit states in the present study, the cable breakage event will usually induce the exceedance of serviceability limit state I corresponding to light damage condition. However, unless wind speed is higher than 20 m/s, the serviceability limit states II and III are usually not exceeded or exceeded at a low probability.



(a) Light traffic flow condition



(b) Moderate traffic flow condition



(c) Busy traffic flow condition

Figure 9.14 Bridge fragility corresponding to serviceability limit states

9.6 Conclusions

This study presents a structural reliability assessment framework for long-span cable-stayed bridges subjected to cable breakage events. The cable breakage events are simulated using a recently developed nonlinear dynamic simulation methodology, in which the cable breakage can start at a dynamic initial state and dynamic loads from stochastic traffic and wind can be considered. Various sources of geometric and material nonlinearities for the beam and cable elements are incorporated in the nonlinear dynamic simulation process. Based on the preliminary sensitivity analysis, the random variables are defined in the study from five sources, which are structural material properties, sectional properties, traffic condition, wind load condition and cable breakage parameters. The Latin hypercube sampling technique is adopted to sample the random variables and form combinations of different simulation models. Structural fragility analysis of a prototype long-span cable-stayed bridge subjected to breakage of stay cables is conducted for bridge ultimate limit state with respect to cable parameters including breakage duration and process. In addition, fragility analysis of the bridge and traffic conditioned on traffic density and steady-state wind speed is conducted corresponding to both bridge ultimate and serviceability limit states. For bridge ultimate limit state, several types of possible failure modes are analyzed and the dominant failure mode for the prototype bridge is determined. For the breakage cases involved with vehicles, bridge serviceability is also investigated based on the vehicle proportion exceeding vehicle safety criterion in addition to bridge performance corresponding to ultimate limit state. The main conclusions based on the investigated prototype long-span cable-stayed bridge can be summarized in the following several points:

- Both the breakage duration and breakage process may influence the bridge failure probability significantly under certain conditions.
- Bridge failure probability has the largest value irrespective of the breakage processes when the breakage of stay cable occurs abruptly. The failure probability of the bridge decreases as the breakage duration increases. However, the failure probability will be stable after the breakage duration is larger than a certain value, which varies with different breakage processes.
- For a specific breakage time duration, the failure probability of the bridge generally increases nonlinearly as the breakage process approaches the abrupt breakage process.
- The dominant failure mode for the prototype bridge subjected to breakage of stay cables is attributed to structural instability with the initial failure in the form of plastic hinges on the pylon elements.
- Traffic and wind excitations may significantly increase the bridge failure probability during the cable breakage event.
- For the same traffic flow density, failure probability corresponding to bridge ultimate limit state doesn't have much variation over wind speeds when they are no more than 20 m/s. However the probability significantly increases as the wind speed increases over 20 m/s for the same traffic flow density. For the same wind speed, the bridge failure probability gradually increases as the traffic density increases.
- Vehicle safety conditions are similar in traffic flows with different traffic densities when wind is very mild. Following the cable breakage events, vehicles become less safe as wind speed gets higher and the traffic flow is lighter.

- For the prescribed serviceability limit states in the present study, the cable breakage event will usually induce the exceedance of serviceability limit state I. However, the serviceability limit states II and III are usually not exceeded or exceeded at a low probability when wind speed is not higher than 20 m/s. When wind speed is higher than 20 m/s, all the three defined serviceability limit states will be inevitably exceeded, with the largest and smallest probability of exceedance for serviceability limit states I and III, respectively.
- The most disadvantageous scenario for both bridge ultimate and serviceability limit state occurs when the traffic flow density and wind speed are both the largest among all cable breakage scenarios. Between the two types of dynamic loads from traffic flow and wind in a cable breakage event, traffic flow density has larger influence on the bridge failure probability corresponding to bridge ultimate limit state, while bridge serviceability is more prone to be influenced by wind excitations.

CHAPTER 10 Summary of the dissertation and future studies

This dissertation aims to develop simulation frameworks and assess the performance for long-span bridges under multiple threats. By focusing on investigation of long-span cable-stayed bridges, the specific dynamic excitations from stochastic traffic, wind speed, earthquake ground motions and breakage of stay cables are incorporated in the simulation frameworks. The dynamic performance of the bridge as well as traveling vehicles are obtained and evaluated under multiple combinations of dynamic excitations. Although demonstrated on long-span cable-stayed bridges, the same simulation frameworks can be readily applied to long-span suspension bridges as well as short- or medium-span highway bridges. The main achievements of the dissertation and the possible improvements in future studies are summarized in the following sections.

10.1 Achievements of the dissertation

10.1.1 Fully-coupled bridge-traffic interaction analysis using mode superposition method

Two sets of systematical time-domain simulation frameworks are developed for bridge-traffic interaction system under multiple excitations. Different from previous studies, the developed simulation frameworks directly couple the bridge and each individual vehicle, and therefore the dynamic response of the bridge as well as each vehicle can be obtained from the analysis. The first set of program is established based on the linear random vibration theory in the time domain, in which the bridge response is obtained from linear superposition of the response from each participating mode (Chapter 2). The mode-based fully-coupled bridge-traffic interaction model is demonstrated with external dynamic excitations from wind and earthquake ground motions in Chapter 2. For this method, the bridge structure is developed based on the modal coordinates and the vehicles in the traffic flow are modeled using physical coordinates. The proposed strategy provides a general simulation platform for the dynamic analysis of the bridge-traffic system subjected to various types of service and extreme loads, including road roughness, turbulent wind and earthquake ground motions. Since the simulation platform is

developed in the multi-threat context, it can also be used in various traditional single-threat (hazard) analyses, such as aerodynamic flutter, buffeting analysis, or seismic analysis using the time-domain vibration theory.

10.1.2 Fully-coupled bridge-traffic interaction analysis using finite element method

In addition to mode superposition method as summarized in 10.1.1, the fully-coupled bridge-traffic interaction model is also developed using nonlinear structural dynamics on the finite element basis (Chapter 5). Different from the mode-based model, the bridge is modeled using two types of elements, which are the spatial beam element based on Timoshenko beam theory and the spatial catenary cable element formulated on the analytical equilibrium equation of elastic catenary. The framework underwent a systematic and long-term developing process to realize the function of nonlinear dynamic analysis under multiple hazards from wind, traffic and cable breakage events. The formulation of the program is validated by comparing the results from similar analysis in SAP2000 program, specifically, nonlinear static analysis, modal analysis and dynamic analysis. In the finite element simulation platform, comprehensive considerations of both geometric and material nonlinearities originated from structure and dynamic excitations are incorporated. The aerodynamic forces of the bridge structure are modeled in the time domain using convolution integrals, in which the frequency dependent aerodynamic flutter derivatives and admittance functions in the classical formulation are expressed using the rational function approximation approach. In addition to the functions of the mode-based simulation framework, the finite element based simulation framework can also be used in general nonlinear static analysis, aerostatic stability analysis, static, dynamic stability analysis and progressive failure analysis.

10.1.3 The equivalent moving traffic load (EMTL) and hybrid earthquake analysis approach

The equivalent moving traffic loads (EMTL) with respect to time are obtained from either the modebased or the finite-element based bridge-traffic interaction analysis. EMTL are explicitly expressed in the form of force time histories on the bridge to take into account of the static and dynamic effects from the moving vehicles. EMTL can be adopted with reasonable accuracy for engineers who are not familiar with coupled bridge-traffic interaction analysis. In addition, considering that the fully-coupled bridge-traffic interaction analysis cannot be easily implemented in commercial software, the obtained EMTL can be combined with commercial software for complex nonlinear analysis by taking advantage of the strength on nonlinear assessment from well-established and calibrated commercial software. In Chapter 3, a hybrid time-domain simulation methodology of long-span bridges as well as moving vehicles subjected to seismic excitations is developed by using the modal-based bridge-traffic interaction model and commercial FE software SAP2000 through EMTL. Although it was demonstrated with SAP2000, it is noted that the same hybrid analysis concept and procedure can be applied to other commercial software when some other extreme or hazardous loads are considered.

10.1.4 The simulation methodology for cable breakage events using SAP2000

A time-progressive dynamic simulation strategy of cable breakage is developed based on the commercial finite element program SAP2000. The simulation methodology considers the bridge-traffic interaction during the cable breakage process through the EMTL obtained from bridge-traffic interaction analysis. Dynamic initial states of abrupt cable breakage events are incorporated for the first time, in which the cable breakage occurs at a random time instant with non-zero velocity and acceleration. Various sources of geometric and material nonlinearity are considered in the cable breakage process through the nonlinear modules of the commercial program. The simulation methodology is presented in Chapter 4 along with numerical evaluations based on the cable breakage simulation on a prototype long-span cable-stayed bridge. The response envelope analysis is conducted to evaluate the safety of using quasi-dynamic analysis with dynamic factors as suggested by PTI for cable breakage design.

10.1.5 The direct simulation methodology for cable breakage events using finite element method

The methodology for cable breakage simulation in Chapter 4 is essentially a type of counteracting force method to mimic the breakage process. In Chapter 5, after the finite element based dynamic coupling analysis framework is established, the application of the new framework on investigating cable-loss incidents for long-span cable-stayed bridges was carried out. In the new FEM-based cable-loss

simulation framework, cable-loss incidents are simulated through nonlinear iteration in the time domain and the cable-breakage process is realized by both applying the counteracting forces and also physically reducing the area of the ruptured cable. In this way, both the elemental configuration and force condition during a general cable loss incident can be characterized more realistically. The full dynamic interactions of the bridge-traffic-wind system are incorporated in the simulation framework instead of using EMTL. The various sources of geometric and material nonlinearities originated from structure, aerodynamic loads and cable-loss incidents are incorporated in the same simulation framework.

Based on the direct nonlinear dynamic simulation framework for cable breakage events developed in Chapter 5, a comprehensive parametric investigation on a prototype bridge is carried out to study the impact on the post-breakage bridge response from cable-breakage parameters including cable-breakage process, duration and initial state in Chapter 6. The influences of dynamic excitations from stochastic traffic and wind as well as the coupling effects on the dynamic response of the bridge subjected to cable breakage events are also investigated. Response envelopes on the bridge girders, pylons and stay cables due to breakage of a single stay cable are obtained using the nonlinear dynamic simulation methodology and then compared to those obtained with the equivalent static analysis approach as introduced by the Post-Tensioning Institute (PTI) in order to reveal the applicability of the codified analytical approach on long-span bridges.

10.1.6 Methodologies for driving safety and comfort analysis of moving traffic

An integrated dynamic interaction and accident assessment framework is proposed in Chapter 7 based on the fully coupled bridge-traffic-wind interaction model developed in Chapter 5. Different from existing studies on vehicle accident analysis, realistic stochastic traffic flow is considered in the vehicle driving safety analysis, in which multiple vehicles are on the bridge at the same time and the vehicles can accelerate and decelerate following certain traffic rules. The proposed vehicle safety assessment methodology is able to identify the vehicle lift-up, side-slip and yawing accidents within the same integrated simulation framework for the dynamic interaction analysis between the bridge and vehicles. Taking a prototype long-span cable-stayed bridge and three types of vehicles as example, the safety condition corresponding to each type of accidents for the representative vehicles is evaluated. The influences from the multi-vehicle presence and dynamic initial states of wind excitations on traffic safety are numerically studied.

A methodology of ride comfort analysis is proposed in Chapter 8 for typical vehicles driving on the long-span bridge in wind environment. The vehicle responses are obtained from the fully-coupled bridge-traffic-wind interaction analysis. Therefore, the long-span bridge and all the vehicles in the traffic flow are directly coupled under wind excitations. The guidelines that are recommended in ISO 2631-1 for vehicle ride comfort evaluation are interpreted within the present vehicle model context. The essential processes in the vehicle comfort analysis are described in details, which includes obtaining the whole-body vibration response, frequency weighting the original response and determining the OVTV (overall vibration total value). Numerical studies are conducted on a prototype long-span bridge and typical vehicles. The effects of interactions between bridge and vehicles and the effects of wind excitations on the ride comfort of the vehicles are investigated and discussed.

10.1.7 The structural reliability assessment framework for the bridge-traffic system

A reliability assessment framework for cable breakage events is developed in Chapter 9 based on the nonlinear dynamic simulation methodology established in Chapter 5. Important cable breakage parameters, dynamic loads from stochastic traffic and wind, are considered as random variables. A restrained sampling technique is adopted to obtain a series of structural models with reasonable computational efforts for nonlinear time history analyses. Different from traditional reliability analysis for bridge structures, the study defines both bridge ultimate and serviceability limit states based on the bridge and vehicle behavior, respectively. Due to the lack of straightforward hazard intensity functions for cable breakage events, the fragility of the bridge subjected to cable breakage incidents are assessed conditioned on some key parameters without specifying the particular hazard event. The structural failure probability corresponding to bridge ultimate limit state is evaluated in the cable breakage scenarios with different

cable breakage parameters. The bridge fragility with respect to dynamic loads from stochastic traffic and wind are also discussed in terms of bridge ultimate and serviceability performance.

10.2 Possible improvements of the dissertation and future studies

The dissertation has made some promising progress on the research of long-span bridges subjected to multiple threats such as developing some analytical frameworks for the bridge, traffic and hazards. As a relatively new field to be explored, the work in this dissertation on the long-span bridge and traffic system is explorative. Some possible improvements and extensions from the research work in the dissertation will be discussed in the following.

10.2.1 More refined modeling of the bridge structure

The finite element model of the bridge structure in the dissertation uses two types of elements, which are the spatial beam element and spatial catenary cable element. The spatial beam element can be replaced by plate elements in order to obtain more detailed response information on the integrated bridge girder. However, this may be computationally prohibitive due to the large element number of the whole bridge. If more detailed bridge model using plate elements is adopted, some related technical challenges, such as modeling aerodynamic forces and more advanced modeling of damage propagation involving material nonlinearity, need to be taken care of.

10.2.2 Dynamic parameter validation of road vehicle models

The road vehicles are idealized as a combination of rigid bodies and mass axles connected by a series of springs and dampers. It is known that vehicles from different makes and models have different axle mass, stiffness and damping coefficient. In future studies, more accurate vehicle data may be needed in order to develop the vehicle dynamic models and improve the associated accuracy of analyses. In addition, the vehicles could be modeled as detailed finite element models instead of the idealized dynamic model to obtain more accurate vehicle response.

10.2.3 Aerodynamic interference effects in the bridge-traffic system

For bridges, the static wind coefficients and flutter derivatives may be changed due to the presence of multiple vehicles. However, considering that the vehicle positions on the bridge change from time to time, the whole cross section of the bridge-traffic system also change over time. For the vehicles, the aerodynamic interference effects are more complicated due to the temporary shielding effects of bridge towers and the adjacent vehicles. The time-dependent wind coefficients of the bridge and vehicles may be needed in the future studies through developing some innovative wind tunnel testing strategy.

10.2.4 Cable breakage simulation due to specific causes

The study established methodologies for simulating cable breakage events for cable-stayed bridges by assuming breakage of certain stay cables occurs regardless of causes. There is currently little data or research on the accurate breakage parameters for stay cables for different specific causes. Future experimental and analytical studies should be carried out to investigate the cable breakage parameters and also dynamic performance of the bridge subjected to cable breakage events with specific causes.

10.2.5 Dynamic simulation of long-span bridges subjected to more types of natural and manmade hazards

In this dissertation research, earthquake and cable-loss are two major hazardous events being considered. The analytical frameworks developed in the present study can actually be applied to many other natural and man-made hazards. Based on the established simulation frameworks developed in this dissertation, more types of natural or manmade hazards should be incorporated into the comprehensive dynamic assessment of long-span bridges, which may include but not be limited to hurricane, fire, flooding, wave, blast, vehicle or ship collision, etc. Each type of new hazards requires the modeling of the specific hazard characterizations and the respective interaction effects with the structure. With more studies on various hazards, more insights about the vulnerability and resilience of long-span bridge system performance subjected to typical hazards can be disclosed.

REFERENCES

AASHTO (2012). LRFD bridge design specifications, *American Association of State Highway and Transportation Officials*, Washington, D.C.

ASCE. (2003). "Assessment of performance of vital long-span bridges in the United States." *ASCE* subcommittee on performance of bridges, R. J. Kratky, ed., ASCE, Reston, VA.

ASCE (2010). "Minimum Design Loads for Buildings and Other Structures", ASCE/SEI 7-10, American Society of Civil Engineers, Reston, Virginia.

Baker, C. J. (1986). "A simplified analysis of various types of wind-induced road vehicle accidents", *Journal of Wind Engineering and Industrial Aerodynamics*, 22 (1986), 69-85.

Baker, C. J. (1987). "Measures to control vehicle movement at exposed sites during windy periods." *J. Wind Eng. Ind. Aerodyn.*, 25 (2), 151–161.

Baker, C. J. (1991). "Ground vehicles in high cross winds. 3: The interaction of aerodynamic forces and the vehicle system." *J. Fluids Struct.*, 5, 221–241.

Baker, C. J. (1994). "Quantification of accident risk for road vehicles in cross winds." *J. Wind Eng. Ind. Aerodyn.*, 52(1–3), 93–107.

Barlovic, R., Santen, L., Schadschneider, A. and Schreckenberg, M. (1998). "Metastable states in cellular automata for traffic flow", *European Physical Journal B*, 5 (3), 793-800.

Berrah, M. and Kausel, E. (1992). "Response spectrum analysis of structures subjected to spatially varying motions", *Earthquake Engineering and Structural Dynamics*, 21, 461-470.

Blejwas, T. E., Feng, C. C. and Ayre, R. S. (1979). "Dynamic interaction of moving vehicles and structures", J. Sound & Vibration, 67 (4), 513-521.

British Standards Institution 1987, British Standard Guide to Measurement and evaluation of human exposure to whole-body mechanical vibration and repeated shock, BS 6841:1987 edn.

Bucher, C. G. (1988). "Adaptive sampling-an iterative fast Monte Carlo procedure", *Structural Safety*, 5 (1988), 119-126.

Cai, C. S. and Chen, S. R. (2004). "Framework of Vehicle-Bridge-Wind Dynamic Analysis", *Journal* of Wind Engineering and Industrial Aerodynamics, 92 (7-8), 579-607.

Cai, J. G., Xu, Y. X., Zhuang, L. P., Feng, J. and Zhang, J. (2012). "Comparison of various procedures for progressive collapse analysis of cable-stayed bridges". *Journal of Zhejiang University-science A (Applied physics and Engineering)* 13(5), p. 323-334.

Catbas, F. N., Susoy, M. et al. (2008). "Structural Health Monitoring and Reliability Estimation: Long Span Truss Bridge Application with Environmental Data", *Engineering Structures*, 30 (9), 2347–2359.

Chen, S. R. and Cai, C. S. (2003). "Evolution of long-span bridge response to wind-numerical simulation and discussion", *Computers & Structures*, 81, 2055-2066.

Chen, S. R. and Cai, C. S. (2004). "Accident assessment of vehicles on long-span bridges in windy environments", *Journal of Wind Engineering and Industrial Aerodynamics*, 92 (2004), 991-1024.

Chen, S. R. and Cai, C. S. (2007). "Equivalent wheel load approach for slender cable-stayed bridge fatigue assessment under traffic and wind: Feasibility study", *Journal of Bridge Engineering*, ASCE, 12(6), p. 755-764.

Chen, S. R., Cai, C.S. and Wolshon, B. (2009). "From normal operation to evacuation: single-vehicle safety under adverse weather, topographic and operational conditions." *Natural Hazards Review*, ASCE, 10 (2), 68-76.

Chen, S. R. and Chen, F. (2010). "Simulation-based assessment of vehicle safety behavior under hazardous driving conditions", *Journal of Transportation Engineering*, ASCE, 136 (4), 304-315.

Chen, S. R., Nelson, R., Chen, F. and Chowdhury, A. (2013). "Impact of Stochastic Traffic on Modified Cross-section Profiles of Slender Long-span Bridge - Wind Tunnel Experimental Investigation", *Journal of Engineering Mechanics, ASCE*, 139(3), 347-358.

Chen, S. R. and Wu, J. (2010). "Dynamic performance simulation of long-span bridge under combined loads of stochastic traffic and wind", *Journal of Bridge Engineering*, 15(3), p. 219-230.
Chen, S. R. and Wu, J. (2011). "Modeling stochastic live load for long-span bridge based on microscopic traffic flow simulation", *Computer & Structures*, 89, 813-824.

Chen, W.F. and Duan, L. (2015). *Bridge Engineering Handbook: seismic design*, 2nd edition, CRC press.

Chen, X. Z., Matsumoto, M. and Kareem, A. (2000). "Time domain flutter and buffeting response analysis of bridges", *Journal of Engineering Mechanics*, ASCE, 126(1), 7-16.

Chen, X. Z., Kareem, A. (2001). "Nonlinear response analysis of long-span bridges under turbulent winds", *Journal of Wind Engineering and Industrial Aerodynamics*, 89 (2001), 1335-1350.

Clough, R. W. and Penzien, J. (2003). *Dynamics of structures*, 3rd Edition, McGraw Hill, New York. Coleman, S. A., and Baker, C. J. (1990). "High sided road vehicles in cross wind." *J. Wind Eng. Ind.*

Aerodyn., 36 (1990), 1383-1392.

CSI Analysis Reference Manual (2008). Computers & Structures INC. Berkeley, California, USA.

Daubechies, I. (1992). "Ten lectures on wavelets", CBMS-NSF Regional Conference Series in Applied Mathematics, Philadelphia.

Davenport, A. G. (1962). "Buffeting of a suspension bridge by storm winds", *Journal of Structural Engineering*, ASCE, 88 (3), 233-270.

Der Kiureghian, A. and Neuenhofer, Y. (1992). "Response spectrum method for multi-support seismic excitation", *Earthquake Engineering and Structural Dynamics*, 22, 943-956.

Der Kiureghian, A. (1996). "A coherency model for spatially varying ground motions", *Earthquake Engineering and Structural Dynamics*, 25 (1), 99-111.

Deodatis, G. (1996a). "Simulation of ergodic multivariate stochastic processes", *Journal of Engineering Mechanics*, ASCE, 122 (8), 778-787.

Deodatis, G. (1996b). "Non-stationary stochastic vector processes: seismic ground motion applications", *Probabilistic Engineering Mechanics*, 11 (1996), 149-168.

Ding, Q. S., Chen, A. R. and Xiang, H. F. (2002). "Coupled flutter estimate of a suspension bridge". *Journal of Wind Engineering and industrial Aerodynamics*, 86 (2000), 1981-1993. Dodds, C. J. (1972). "BSI proposals for generalized terrain dynamic inputs to vehicles". *International Organization for Standardization ISO/TC/108/WG9*, Document No. 5, 1972.

Edward, L. W. (2002). Three-dimensional static and dynamic analysis of structures (3rd Edition), *Computer and Structures*, Inc., CA, USA.

Engelund, S. and Rackwitz, R. (1993). "A benchmark study on importance sampling techniques in structural reliability", *Structural Safety*, 12 (1993), 255-276.

Federal Emergency Management Agency, 2000 (FEMA-356). Prestandard and commentary for the seismic rehabilitation of buildings. *Building Seismic Safety Council*, Washington, DC.

Flanagan, J. L. (1972), "Speech analysis: Synthesis and perception", 2nd Edition, Springer, New York.
Florian, A. (1992). "An efficient sampling scheme: Updated Latin Hypercube Sampling", *Probabilistic Engineering Mechanics*, 7 (1992), 123-130.

Ge, Y. J. and Tanaka, T. (2000). "Aerodynamic analysis of cable-supported bridges by multi-mode and full-mode approach". *Journal of Wind Engineering and industrial Aerodynamics*, 86 (2000), 123-153.

Griffin, M.J. (1990). Handbook of human vibration, Academic Press, London.

Guo, W. H. and Xu, Y. L. (2001). "Fully computerized approach to study cable-stayed bridge-vehicle interaction", *Journal of Sound and Vibration*, 248(4), 745-761.

Guo, W. H. and Xu, Y. L. (2006). "Safety analysis of moving road vehicles on a long bridge under crosswind", *Journal of Engineering Mechanics*, ASCE, 132 (4), 438-446.

Harichandran, R.S. and Vanmarcke, E.H. (1986). "Stochastic variation of earthquake ground motion in space and time", *Earthquake Engineering and Structural Dynamics*, 112 (1986), 154-174.

Huang, D.Z. and Wang, T.L. (1992). "Impact analysis of cable-stayed bridges", *Computers and Structures*, 31, 175-183.

ISO 2631/1 (1985), Evaluation of human exposure to whole-body vibration: part 1 -general requirements, International Organization for Standardization, Geneva.

ISO 2631-1 (1997), *Mechanical vibration and shock: evaluation of human exposure to whole-body vibration - part 1: general requirements,* International Organization for Standardization, Geneva. Jain, A., Jones, N.P. and Scanlan, R.H. (1996). "Coupled flutter and buffeting analysis of long-span bridges", *Journal of Structural Engineering*, ASCE, 122(7), 716-725.

Karmakar, D., Ray-Chaudhuri, S. and Shinozuka, M. (2012). "Seismic response evaluation of retrofitted Vincent Thomas bridge under spatially variable ground motions", *Soil Dynamics and Earthquake Engineering*, 42 (2012), 119-127.

Kwon, O., Mwafy, A., and Elnashai, A.S. (2010). "Analytical Assessment of Seismic Performance Evaluation Procedures for Bridges," *ACI Special Publication*. Vol. 271:45-62.

Léger, P., Idé, I.M. and Paultre, P. (1990). "Multiple-support seismic analysis of large structures", Computers & Structures, 36 (6), 1153-1158.

Li, Y., and Ellingwood, B. R. (2006). "Hurricane damage to residential construction in the U.S.: Importance of uncertainty modeling in risk assessment", *Engineering Structure*, 28 (7), 1009-1018.

Li, Y. L., Chen, N., Zhao, K. and Liao, H. L. (2012). "Seismic response analysis of road vehiclebridge system for continuous rigid frame bridges with high piers", *Earthquake Engineering and Engineering Vibration*, 11 (2012), 593-602.

Liang, J. W., Chaudhuri, S. R. and Shinozuka, M. (2007). "Simulation of nonstationary stochastic processes by spectral representation", *Journal of Engineering Mechanics*, ASCE, 133 (6), 616-627.

Lin, Y. K. and Yang, J. N. (1983). "Multimode bridge response to wind excitations", Journal of Engineering Mechanics, ASCE, 109 (2), 586-603.

Liu, M. F., Chang, T. P. and Zeng, D. Y. (2011). "The interactive vibration behavior in a suspension bridge system under moving vehicle loads and vertical seismic excitations", *Applied Mathematical Modeling*, 35, 398-411.

Li, Y. L., Chen, N., Zhao, K. and Liao, H. L. (2012). "Seismic response analysis of road vehiclebridge system for continuous rigid frame bridges with high piers", *Earthquake Engineering & Engineering Vibrations*, 11, 593-602.

Mackie, K. and B. Stojadinovic (2001). Probabilistic Seismic Demand Model for California Highway Bridges. *J. Bridge Engineering*, ASCE, 6(6): 468-481. MATLAB version R2013b [Computing language]. MathWorks, Natick, MA.

McGuire, W., Gallagher, R. H. and Ziemain, R. D. (2000). "Matrix Structural Analysis", Second edition, Wiley, Hoboken, NJ, USA.

Mozos, M. and Aparicio A.C. (2010a). "Parametric study on the dynamic response of cable stayed bridges to the sudden failure of a stay, part I: bending moment acting on the deck." *Engineering Structures*, 32(2010), 3328-3300.

Mozos, C.M. and Aparicio, A.C. (2010b). "Parametric study on the dynamic response of cable stayed bridges to the sudden failure of a stay, part II: bending moment acting on the pylons and stress on the stays." *Engineering Structures*, 32(2010), 3301-3312.

Mozos, C.M. and Aparicio, A.C. (2011). "Numerical and experimental study on the interaction cable structure during the failure of a stay in a cable stayed bridge", Engineering Structures, 33(8), p. 2330-2341.

Nagel, K. and Schreckenberg, M. (1992). "A cellular automaton model for freeway traffic." *Journal* de Physique I France, 2(12), p. 2221-2229.

Nahvi, H., Fouladi, M. H. and Nor, M. J. M. (2009). "Evaluation of Whole-Body Vibration and Ride Comfort in a Passenger Car", *International Journal of Acoustics and Vibration*, 14(3), 143-149.

National Institute for Occupational Safety and Health (NIOSH) (2007). "Truck driver occupational safety and health, 2003 conference report and selective literature review", *Centers for Disease Control and prevention*, Department of Health and Human Services.

Nazmy, A.S. and Abdel-Ghaffar, A.M. (1990). "Three-dimensional nonlinear static analysis of cablestayed bridges". *Computer & Structures*, 34(2), p. 257-271.

Neilson, B. G. and DesRoches, R. (2007). "Seismic fragility methodology for highway bridges using a component level approach", *Earthquake Engng Struct. Dyn.*, 36 (2007), 823–839.

Olsson, M. (1985). "Finite element modal coordinate analysis of structures subjected to moving loads", J. Sound & Vibration, 99 (1), 1-12.

Olsson, A. Sandberg, G. and Dahlblom, O. (2003). "On Latin Hypercube Sampling for structural reliability analysis", *Structural Safety*, 25 (2003), 47-68.

Pourzeynali, S. and Datta, T. K. (2002). "Reliability analysis of suspension bridges against flutter", *Journal of Sound and Vibration*, 254 (1), 143-162.

Post-Tensioning Institute (PTI) (2007). *Recommendations for Stay Cable Design, Testing and Installation*. Fifth edition. Cable-Stayed Bridges Committee, Phoenix, USA.

Reddy, J. N. (2006). "An Introduction to the Finite Element Method", Third edition, McGraw-Hill, New York, NY, USA.

Ruiz-Teran, A. and Aparicio, A. (2009). "Response of under-deck cable-stayed bridges to the accidental breakage of stay cables", *Engineering Structure*, 31 (2009), 1425-1434.

Sadiku, S. and Leipholz, H. H. E. (1987). "On the Dynamics of Elastic Systems with Moving Concentrated Masses," *Archive of Applied Mechanics*, 57 (3), 223-242.

SAP2000 version 15.0.0 [Computer software]. Computers and Structures, Berkeley, CA.

Sears, W. R. (1941). "Some aspects of non-stationary airfoil theory its practical application", *Journal* of the Aeronautical Science, 8 (3), 104-108.

Scanlan, R. H. (1984). "Role of indicial functions in buffeting analysis of bridges", *Journal of Structural Engineering*, ASCE, 110(7), 1433-1446.

Shinozuka, M. and Jan. (1972). "Digital simulation of random processes and its applications", *Journal of Sound and Vibration*, 25 (1), 111-128.

Simiu, E. and Scanlan, R. H. (1978). Wind Effects on Structures: An Introduction to Wind Engineering, 2nd Edition, John Wiley & Sons Inc, Hoboken, NJ.

Soyluk, K. and Dumanoglu, A. A. (2004). "Spatial variability effects of ground motions on cablestayed bridges", *Soil Dynamics and Earthquake Engineering*, 24 (2004), 241-250.

Starossek, U. (2007). "Typology of progressive collapse". *Engineering Structures*, 29 (9), p.2302-2307.

Torres, M. A. and Ruiz, S. E. (2007). "Structural reliability evaluation considering capacity degradation over time", *Engineering Structures*, 29, 2183-2192.

U. S. Department of Transportation (USDOT) (2005), "Report to Congress on the Large Truck Crash Causation Study", Federal Motor Carrier Safety Administration, U.S. Department of Transportation.

Wilson, E. L. (2002). Three-dimensional static and dynamic analysis of structures: A physical approach with emphasis on earthquake engineering, *Computers and Structures, Inc.*, California, 2002.

Wolff, M. and Starossek, U. (2010). "Cable-loss analyses and collapse behavior of cable-stayed bridges". IABMAS 2010. *The Fifth International Conference on Bridge Maintenance, Safety and Management*, July 2010, Philadelphia, USA.

Xia, H. et al. (2006). "Dynamic analysis of train-bridge system subjected to non-uniform seismic excitations", Earthquake Engineering & Structural Dynamics, 35, 1563-1579.

Xu, Y. L. and Guo, W. H. (2003). "Dynamic analysis of coupled road vehicle and cable-stayed bridge systems under turbulent wind", Engineering Structures, 25 (2003), 473-486.

Xu, Y. L., and Guo, W. H. (2004). "Effects of bridge motion and crosswind on ride comfort of road vehicles." *J. Wind Eng. Ind. Aerodyn.*, 92 (2004), 641-662.

Yau, J. D. (2009). "Dynamic response analysis of suspended beams subjected to moving vehicles and multiple support excitations", 325, 907-922.

Yin, X. F., Fang, Z. and Cai, C. S. (2011). "Lateral Vibration of High-Pier Bridges under Moving Vehicular Loads", *J. Bridge Eng.*, ASCE, 16(3), 400-412.

Zerva, A. (1990). "Response of multi-span beams to spatially incoherent seismic ground motions". *Earthquake Engineering and Structural Dynamics*, 19 (1990), 819-832.

Zhang, Y. H., Li, Q. S., Lin, J. H. and Williams, F. W. (2009). "Random vibration analysis of longspan structures subjected to spatially varying ground motions. *Soil Dynamics and Earthquake Engineering*, 29 (2009), 620-629.

Zhang, W. M., Ge, Y. J. and Levitan, M. L. (2011). "Aerodynamic flutter analysis of a new suspension bridge with double main spans", *Wind and structures*, Vol.14, No.3, p.187-208.

Zhou, Y. F. and Chen, S. R. (2013). "Dynamic simulation of long-span cable-stayed bridges subjected to multiple service and hazardous loads". Proceedings of *the 12th Americas Conference on Wind Engineering, ACWE 2013: Wind Effects on Structures, Communities, and Energy Generation*, v2, 1161-1180, June 16-20, Seattle, WA.

Zhou, Y. F. and Chen, S. R. (2014a). "Time-progressive dynamic assessment of abrupt cablebreakage events on cable-stayed bridges", *Journal of Bridge Engineering*, ASCE, 19 (2), 159-171.

Zhou, Y. F. and Chen, S. R. (2014b). "A hybrid simulation strategy for the dynamic assessment of long-span bridges and moving traffic subjected to seismic excitations". Proceedings of *the 13th International Symposium on Structural Engineering, ISSE-13*, Oct. 24-27, 2014, Hefei, China.

Zhou, Y. F. and Chen, S. R. (2015a). "Dynamic simulation of long-span bridge-traffic system subjected to combined service and extreme loads", *Journal of Structural Engineering*, ASCE, 141 (9), 04014215, 1-18.

Zhou, Y. F. and Chen, S. R. (2015b). "Fully coupled driving safety analysis of moving traffic on long-span bridge subjected to crosswind". *Journal of Wind Engineering and Industrial Aerodynamics*, 143 (2015), 1-18.

Zhou, Y. F. and Chen, S. R. (2015c). "Framework of fully-coupled nonlinear dynamic simulation of long-span cable-stayed bridges subjected to cable-loss incidents", *Journal of Structural Engineering*, ASCE, 10.1061/(ASCE)ST.1943-541X.0001440, 04015160.

Zhou, Y. F. and Chen, S. R. (2015d). "Numerical investigation of cable breakage events on long-span cable-stayed bridges under stochastic traffic and wind", *Engineering Structures*, 105 (2015), 299-315.

Zhou, Y. F. and Chen, S. R. (2015e). "Vehicle ride comfort analysis from whole-body vibration on long-span bridges subjected to crosswind", *Journal of Wind Engineering and Industrial Aerodynamics*, ASCE, under review.

Zhou, Y. F. and Chen, S. R. (2015f). "Reliability assessment framework of the long-span cable-stayed bridge and traffic system subjected to cable breakage events", *Journal of Bridge Engineering*, ASCE, under review.

Zoli, T. and Woodward, R. (2005). "Design of Long Span Bridges for Cable Loss." International Association for Bridge and Structural Engineering (IABSE) Symposium Report, Lisbon 2005: Structure and Extreme Events, pp 17-25 (9).

CURRICULUM VITAE AND PUBLICATIONS DURING PH.D.

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Ph.D.	Structural Engineering Colorado State University, Fort Collins, CO Advisor: Dr. Suren Chen Dissertation Title: "Dynamic assessment of the long-span cable-supported bridge and traffic system subjected to multiple hazards" Cumulative GPA: 4.00/4.00	Spring 2016
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Publications

Referred Journal Papers

J1. **Zhou, Yufen** and Chen, Suren (2014). "Time-progressive dynamic assessment of abrupt cable-breakage events on cable-stayed bridges." *Journal of Bridge Engineering*, ASCE, 19 (2), 159-171.

J2. **Zhou, Yufen** and Chen, Suren (2015). "Dynamic simulation of long-span bridge-traffic system subjected to combined service and extreme loads". *Journal of Structural Engineering*, ASCE, 141 (9), 04014215, 1-18.

J3. **Zhou, Yufen** and Chen, Suren (2015). "Fully coupled driving safety analysis of moving traffic on long-span bridge subjected to crosswind". *Journal of Wind Engineering and Industrial Aerodynamics*, 143 (2015), 1-18.

J4. Zhou, Yufen and Chen, Suren (2015). "Framework of fully-coupled nonlinear dynamic simulation of long-span cable-stayed bridges subjected to cable-loss incidents", *Journal of Structural Engineering*, ASCE, 10.1061/(ASCE)ST.1943-541X.0001440, 04015160.

J5. **Zhou, Yufen** and Chen, Suren (2015). "Numerical investigation of cable breakage events on long-span cable-stayed bridges under stochastic traffic and wind", *Engineering Structures*, 105 (2015), 299-315.

J6. **Zhou, Yufen** and Chen, Suren (2015). "Vehicle ride comfort analysis from whole-body vibration on long-span bridges subjected to crosswind", *Journal of Wind Engineering and Industrial Aerodynamics*, ASCE, in review.

J7. Zhou, Yufen and Chen, Suren (2015). "Reliability Assessment Framework of the Long-Span Cable-Stayed Bridge and Traffic System Subjected to Cable Breakage Events", *Journal of Bridge Engineering*, ASCE, under review.

J8. Wu, Jun, **Zhou, Yufen** and Chen, Suren (2012). "Wind-induced Performance of Long-span Bridge with Modified Cross-section Profiles by Stochastic Traffic". *Engineering Structures*, 41, 464-476.

Referred Conference Papers

C1. **Zhou, Yufen** and Chen, Suren (2015). "Structural reliability assessment of long-span cable-stayed bridges subjected to cable loss". *Proceedings of Structural Congress 2015*, Apr. 23-25, 2015, Portland, OR.

C2. **Zhou, Yufen** and Chen, Suren (2014). "A hybrid simulation strategy for the dynamic assessment of long-span bridges and moving traffic subjected to seismic excitations". Proceedings of *the 13th International Symposium on Structural Engineering, ISSE-13*, Oct. 24-27, 2014, Hefei, China.

C3. **Zhou, Yufen** and Chen, Suren (2014). "Toward the design load characterization: Applicability of the AASHTO design live load on long-span cable-stayed bridges". Proceedings of *the 13th International Symposium on Structural Engineering, ISSE-13*, Oct. 24-27, 2014, Hefei, China.

C4. **Zhou, Yufen** and Chen, Suren (2013). "Dynamic simulation of long-span cable-stayed bridges subjected to multiple service and hazardous loads". Proceedings of *the 12th Americas Conference on Wind Engineering, ACWE 2013: Wind Effects on Structures, Communities, and Energy Generation*, v2, 1161-1180, June 16-20, Seattle, WA.