Multiple-Hazard Fragility and Restoration Models of Highway Bridges for Regional Risk and Resilience Assessment in the United States: State-of-the-Art Review

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Abstract: Highway bridges are one of the most vulnerable constituents of transportation networks when exposed to one or more natural hazards, such as earthquakes, hurricanes, tsunamis, and riverine floods. To facilitate and enhance prehazard and posthazard event mitigation and emergency response strategies of transportation systems and entire communities, probabilistic risk and resilience assessment methodologies have attracted increased attention recently. In this context, fragility and restoration models for highway bridges subjected to a range of hazards are essential tools for efficient and accurate quantification of risk and resilience. This paper provides a comprehensive review of state-of-the-art fragility and restoration models for typical highway bridge classes that are applicable for implementation in multihazard risk and resilience analyses of regional portfolios or transportation networks in the United States. An overview of key gaps in the literature is also presented to guide future research. **DOI: 10.1061/(ASCE)ST.1943-541X.0001672.**

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Introduction

Bridges constitute a component of highway transportation networks with paramount importance since they play a crucial role facilitating an efficient commerce and commuting system between cities and across the country. Moreover, as past notable events have shown, bridges can be very susceptible to damages induced by a multitude of natural hazards such as earthquakes (Housner and Thiel 1995; Basöz and Kiremidjian 1998; Kawashima et al. 2011a, b; Schanack et al. 2012), hurricanes (Douglass et al. 2004; Padgett et al. 2008; Stearns and Padgett 2011), and tsunamis (Ghobarah et al. 2006; Saatcioglu et al. 2006). Bridge damage from such events causes severe disruptions to transportation networks compromising their functionality, which in turn impacts emergency response and ultimately the socioeconomic recovery of extended regions.

This increased awareness of the vulnerability of highway bridges to multiple hazards has led to a growing interest of the research community towards quantification of risk and resilience for bridge structures. Risk is defined as the combination of probabilities and consequences of adverse events generated by specific hazards (Decò and Frangopol 2011). In this context, consequences can be undesirable effects on a structure such as attaining particular damage levels (e.g., collapse) or potential economic losses associated with these damages (e.g., costs due to required repair actions or due to loss of functionality of a transportation network). Resilience is defined as the ability of social units to mitigate hazards, contain the effects of disasters when they occur, and carry out recovery activities that minimize social disruption and mitigate the effects of future hazard events (Bruneau et al. 2003). Therefore, the establishment of comprehensive risk and resilience assessment frameworks for regional bridge portfolios or transportation networks, subjected to various hazards occurring independently to each other or in a concurrent/cascading manner, can facilitate prehazard and posthazard event mitigation and emergency response strategies. Development of such methodologies requires (1) probabilistic quantification of possible damage levels that bridges suffer when exposed to a range of hazard cases through fragility models and (2) mapping the vulnerability of the bridge to appropriate recovery patterns to quantify restoration times and ultimately resilience using restoration models. Therefore, fragility and restoration models constitute necessary analysis tools for the accomplishment of these tasks.

This paper presents an extensive literature review of fragility and restoration models for typical highway bridge classes across the United States, rather than very specific case studies (which are also prevalent in the literature). The models presented are viable for analysis of regional portfolios or transportation networks as

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opposed to individual bridge structures. This state-of-the-art summary identifies and further categorizes existing fragility and restoration models that may be incorporated into regional risk and resilience modeling packages such as NIST-CORE (2015) based on hazard type, geographic location in the United States, bridge typology, and level of sophistication. Additionally, key knowledge gaps are identified that represent areas of needed research. To further facilitate the selection of appropriate models for risk and resilience assessment of bridges under multiple natural hazards, this study addresses important characteristics of various identified models and highlights key aspects regarding the methodologies adopted for their derivation. It is noted that the focus of this study is on earthquake, hurricane-induced surge and wave (i.e., coastal flood), tsunami, and riverine flood hazards, as well as on characteristic cases of concurrent (earthquake and flood-induced scour) and cascading (mainshock-aftershock earthquake sequences) hazards. Other natural hazard types that can potentially cause damages to highway bridges, including strong winds (Jain et al. 1996; Simiu and Scanlan 1996; Gu et al. 1999; Xu et al. 2009; Wu et al. 2011; Seo and Caracoglia 2013), landslides and fire (Naser and Kodur 2015), are omitted due to lack of relevant models. Other non-natural hazard cases can also affect the vulnerability of highway bridges and lead to extensive structural damage or collapse. Characteristic examples are vehicle collision (El-Tawil et al. 2005; Sharma et al. 2012, 2014, 2015), barge collision (Whitney et al. 1996; Davidson 2010; Getter and Consolazio 2011; Davidson et al. 2012), and blast loading (Williamson and Winget 2005; Islam and Yazdani 2008; Fujikura and Bruneau 2010, 2011; Williamson et al. 2011a, b). However, a detailed literature review of these types of hazards is out of the scope of this paper, since the focus is on natural hazards. Furthermore, despite the extensive research on vulnerability of these other hazards, fragility models for bridge portfolios subjected to such hazards are lacking in the relevant literature. Within the existing published literature, a few studies provide comprehensive reviews that specifically focus on seismic fragility models (Tsionis and Fardis 2014; Muntasir Billah and Shahria Alam 2015). However, to the best of the authors' knowledge, there is no other study that provides a state-of-the-art review of fragility and restoration models for multiple hazards with particular emphasis on models for typical highway bridge classes applicable to regional risk and resilience assessment in the United States.

In the next section, the concept of fragility models is introduced for the case of earthquake hazard and an introductory discussion related to fundamental aspects of fragility analysis is provided, whereas the following subsections summarize the existing seismic fragility models for different regions in the United States. The available seismic restoration models are reviewed in the subsequent section. The review then continues with summaries of models for hurricane, tsunami, and riverine flood hazards and the examined concurrent and cascading hazards. The paper concludes with a summary and highlights for future research.

Fragility Models for Earthquake Hazard

In general, a fragility model is a function that quantifies the conditional probability representing the likelihood that a structure will meet or exceed a specified damage state (i.e., level of damage) for a given intensity measure (*IM*) of the seismic hazard. It may also be conditioned on a vector of bridge structural parameters $\mathbf{X}$ and time t, such that the effects of different bridge configurations and deterioration due to aging are taken into account, respectively. The fragility can be expressed as $P[DS|IM, \mathbf{X}, t]$, where DS is the damage state (also known as limit state) of the bridge and $P[A|B]$ denotes the conditional probability of event A given B. Although the definition of fragility discussed earlier is made for the case of seismic hazard, it is stressed that the general form is relevant for any other natural hazard of interest with the selection of appropriate intensity measures and hazard-induced damage states.

There are three main methodologies for constructing seismic fragility models: (1) expert opinion methods (expert-based fragility models) that involve questioning experts to determine estimates of probable damage distribution of bridges when subjected to different earthquake intensities (ATC 1985); (2) empirical methods (empirical fragility models) using damage data from postearthquake field observations (Basöz and Kiremidjian 1999; Basoz et al. 1999; Shinozuka et al. 2000; Yamazaki et al. 2000); and (3) analytical or simulation-based methods (often referred to as analytical fragility models) that typically rely on numerical structural models to simulate the seismic response of bridges. The focus here is primarily on the latter type of fragility models for which the mathematical description facilitates their incorporation in computational environments for regional risk and resilience assessment of transportation networks such as *MAEviz* (Mid-America Earthquake Center 2006), Hazus (Hazus-MH 2011), REDARS (Werner et al. 2006), or NIST-CORE (NIST-CORE 2015). These analytical fragility models have received increased attention in the literature in the past decade given their ability to overcome limitations of subjective expert-based fragilities or empirical ones that are constrained by the lack of adequate data.

Due to lack of appropriate data and/or models to characterize the time-progressive deteriorating nature of structural components, most of the available fragility models in the literature are time-independent. Furthermore, computational constraints have imposed some limitations during the development of fragilities, such as not accounting for the variation of bridge parameters that lead to different structural configurations (e.g., number of spans), and ignoring (Zhang et al. 2008; Agrawal et al. 2011) the effect from variations of other geometrical parameters (e.g., span length, deck width, column height, etc.) or only incorporating this effect aggregated with the earthquake intensity measure during the estimation of seismic structural demand (Nielson and DesRoches 2007a; Padgett and DesRoches 2009; Ramanathan et al. 2012). As a result of these limitations, the general expression $P[DS|IM, \mathbf{X}, t]$ is usually simplified to $P[DS|IM]$ and the most commonly used functional form adopted to mathematically express fragilities is the lognormal cumulative distribution function (CDF)

$$P[DS_d|IM] = \Phi\left[\frac{\ln(IM) - \ln(m_d)}{\zeta_d}\right] \tag{1}$$

where $d \in \{1, \ldots, N_d\} = N_d$ different damages states considered; m_d and ζ_d = median and logarithmic standard deviation for each one of the damage states, respectively; and $\Phi(\bullet)$ = standard normal CDF. However, it is now evident that bridges with different structural configurations will have different fragilities (Zhang et al. 2008) and that deterioration effects of aging can considerably change the fragility models of pristine bridges (Choe et al. 2008, 2009, 2010; Ghosh and Padgett 2010; Gardoni and Rosowsky 2011; Zhong et al. 2012). Therefore, more recent research efforts in the bridge engineering community focus on the development of parameterized (Mackie and Stojadinović 2007; Dukes 2013; Ghosh et al. 2013; Kameshwar and Padgett 2014) and time-dependent (Choe et al. 2009; Ghosh and Padgett 2010; Gardoni and Rosowsky 2011; Ghosh 2013) bridge fragility models.

An essential process in the development of fragility models is the definition of damage states that describe different levels of damage for the various bridge components (e.g., columns, bearings, abutments, etc.) in addition to the bridge as a system comprised by these components. The majority of the fragility models reported and discussed in this study use four damage states defined as slight, moderate, extensive, and complete. These damage states originate from the early versions of Hazus (Hazus-MH 2011), which provided qualitative definitions for each one of them. However, derivation of analytical fragility models requires a quantitative description of the considered damage states. Therefore, the onset of the damage states for the different bridge components is determined through thresholds of response quantities usually denoted as engineering demand parameters (EDPs) that are deemed appropriate to adequately capture different levels of damage. Different EDPs are used for different components of a bridge and various researchers have adopted different EDPs to determine damage states for the same component. For example, curvature ductility (Nielson and DesRoches 2007b; Pan et al. 2007, 2010; Padgett and DesRoches 2008; Zhang et al. 2008; Ghosh and Padgett 2010; Ramanathan et al. 2012, 2015; Dukes 2013; Ghosh et al. 2013; Zakeri et al. 2013a; AmiriHormozaki et al. 2015; Kameshwar and Padgett 2014; Pahlavan et al. 2016; Yang et al. 2015), drift displacement (Mackie and Stojadinović 2007; Choe et al. 2008), and shear force (Choe et al. 2008) among others have been used to describe damages to bridge columns. Ultimately, derivation of fragilities at the bridge-system level requires combination of the failure probabilities for the different components such that limit states for the entire bridge are defined. This task is usually performed assuming that the bridge components are connected in series, whereas alternative system-level definitions have been implemented such as the combination of series and parallel components (Lupoi et al. 2006). Some efforts have been made to justify this assumption based upon postevent functionality definitions for each damage level and the fact that inspector closure decisions are often dictated by the condition of the worst component. However, alternative perspectives have been presented in the literature as well, for example, through the use of weighting functions (Zhang et al. 2008; Zhang and Huo 2009), and methods have been proposed for efficient exploration of system failure events (Song and Kang 2009; Dueñas-Osorio and Padgett 2011).

A synopsis of the more recent analytical seismic fragility models for bridge portfolios discussed here is presented in Table 1. The various fragility models are primarily categorized based on their applicability with respect to geographic location in the United States, since one of the objectives of this paper is to facilitate the selection of models viable for risk assessment of regional bridge portfolios through a detailed region-specific inventory of fragilities. Moreover, these models are further characterized based on bridge type, design (e.g., conventional or seismic) and retrofit conditions, time dependency, and parameterization. Also, special notes have been made on the available fragility models incorporated in widely used software tools for regional risk assessment and loss estimation such as Hazus (Hazus-MH 2011) and *MAEviz* (Mid-America Earthquake Center 2006).

Central and Southeastern United States

A considerable number of studies of regional bridge fragility assessment for the Central and Southeastern United States (CSUS) exist in the published literature. These efforts covered a wide range of typical as-built (Nielson and DesRoches 2007a) or retrofitted (Padgett 2007; Padgett and DesRoches 2009) bridge classes, investigated the effect of the preseismic and postseismic design considerations (Ramanathan et al. 2012), incorporated the influence of aging and corrosion (Ghosh and Padgett 2010; Ghosh et al. 2013) or bridge skewness (Yang et al. 2015), and introduced parameterization (Ghosh 2013; Kameshwar and Padgett 2014; Rokneddin et al. 2014) in the development of fragility models.

In particular, Nielson and DesRoches (2007a) developed seismic fragility curves for nine classes of bridges typical to the CSUS (common three-span, zero-skewed bridges with nonintegral abutments) through nonlinear time-history analysis. A probabilistic seismic demand model (PSDM) was adopted that utilizes regression analysis to relate the engineering demand parameters of interest with the chosen earthquake *IM* (Nielson and DesRoches 2007b). Following a series system assumption for the different components of the bridge that takes into account the potential correlation among the component damage measures (Nielson and DesRoches 2007b), exceedance of a particular system damage state was defined and ultimately bridge system fragility curves were proposed. The proposed fragility model was described through the commonly used lognormal form in Eq. (1) for the four common damage states discussed in the previous section. By using a similar approach and series system assumption, Padgett and DesRoches (2008) developed a fragility assessment methodology for common classes of retrofitted bridges in the CSUS. Fragility models were developed for as-built and seismically upgraded bridges considering five different retrofit measures (steel jackets, elastomeric isolation bearings, restrainer cables, seat extenders, and shear keys) and combinations of them. In particular, the four most common and most vulnerable bridge classes in the CSUS were examined (Padgett and DesRoches 2009), including multispan continuous (MSC) steel girder, multispan simply supported (MSSS) steel girder, MSC concrete girder, and MSSS concrete girder bridges. The proposed fragility functions were described through Eq. (1) with m_d and ζ_d depending on the retrofit measure in addition to the damage state. In an attempt to approximately capture the impact of the different retrofit measures on the fragility models of the other common bridge classes in the CSUS, modification factors for m_d of as-built bridges were proposed in Padgett (2007).

Ramanathan et al. (2012) explored the differences in seismic performance of bridge classes built with and without seismic detailing. Fragility models described through Eq. (1) were developed for four common CSUS bridge classes (Table 1) constructed with (post-1990) and without (pre-1990) seismic design detailing considerations, following the approach for bridge fragility development in Nielson and DesRoches (2007a). Yang et al. (2015) studied the influence of the degree of skew angle on the fragility of six bridge classes (Table 1). Fragility models were proposed for nonseismically designed, seismically designed, and seismically upgraded with the five retrofit measures considered in Padgett and DesRoches (2008). Using the conventional lognormal functional form [Eq. (1)] to quantify the proposed fragilities, the effect of the bridge's skewness was taken into account by expressing the fragility parameters m_d and ζ_d as a function of the degree of skew angle. Following a similar approach as Sullivan (2010), fragilities were developed for nonseismically designed skewed MSSS steel girder bridges.

Fig. 1 presents different proposed fragility curves for the four commonly used damage states of MSSS concrete bridges in the CSUS. In particular, fragilities for as-built conventionally designed bridges developed by Nielson and DesRoches (2007a), as-built and retrofitted with steel jackets bridges (Padgett and DesRoches 2009), as-built conventionally and seismically designed bridges (Ramanathan et al. 2012), and as-built conventionally designed bridges for 45° skew and without skew (Yang et al. 2015) are shown in Figs. 1(a–d), respectively. By observing the various fragilities, the advantageous effects of seismic upgrading [Fig. 1(b)] and of higher level of seismic detailing [Fig. 1(c)] on bridge vulnerability are evident, whereas Fig. 1(d) illustrates the increased vulnerability exhibited for skewed bridges.

Table 1. Synopsis of Seismic Fragility Models for Bridge Portfolios

Location	Reference	Bridge typology	Design/retrofit conditions	*IM*	Characteristics
CSUS	Nielson and DesRoches (2007a)	MSSS steel, MSSS concrete, MSSS box, MSSS slab, MSC steel, MSC concrete, MSC slab, SS steel, SS concrete	As-built bridges conventionally designed	PGA	Fragility models dependent on *IM*
	Padgett and DesRoches (2009) and Padgett (2007)	MSSS steel, MSSS concrete, MSSS box, MSSS slab, MSC steel, MSC concrete, MSC slab, SS steel, SS concrete	As-built conventionally designed and retrofitted bridges	PGA	Fragility models dependent on *IM*
	Ramanathan et al. (2012)	MSSS steel, MSSS concrete, MSC steel, MSC concrete	As-built conventionally and seismically designed bridges	PGA	Fragility models dependent on *IM*
	Yang et al. (2015) and Sullivan (2010)	MSSS steel, MSSS concrete, MSC steel, MSC concrete, SS steel, SS concrete	As-built conventionally and seismically designed and retrofitted skewed bridges	PGA	Fragility models dependent on *IM* and the skew angle of the bridge
	Ghosh (2013) and Ghosh and Padgett (2010)	MSSS concrete, MSC steel	As-built bridges conventionally designed	PGA	Time-dependent fragility models due to aging and deterioration
	Ghosh et al. (2013) and Kameshwar and Padgett (2014)	MSSS concrete	As-built bridges conventionally designed	PGA, $S_{a,g}$	Parameterized fragility models dependent on *IM* and bridge material and geometrical parameters
WUS	Zhang et al. (2008)	Six multispan bridges with single column piers (continuous with monolithic abutments, with seat-type abutment, with expansion joint, pier isolation and simply supported connection)	As-built conventionally designed bridges	PGA, PGD	Fragility models dependent on *IM* considering both ground shaking and liquefaction
	Ramanathan (2012) and Ramanathan et al. (2015)	MSC concrete (T and I girder), MSC slab, MSC box	As-built conventionally and seismically designed bridges	$S_{a,T=1}$	Fragility models dependent on *IM* and level of seismic detailing
	Abdel-Mohti and Pekcan (2013)	Posttensioned concrete box girder	As-built conventionally designed skewed bridges	PGA	Fragility models dependent on *IM* and the skew angle of the bridge
	Zakeri et al. (2013a, b)	Single-frame concrete box girder	As-built conventionally and seismically designed and retrofitted skewed bridges	PGA	Fragility models dependent on *IM* and the skew angle of the bridge
	Pahlavan et al. (2016)	Horizontally curved multicolumn concrete box	As-built seismically designed bridges	PGA	Fragility models dependent on *IM* and the bridge deck radius
	Choe et al. (2009) and Gardoni and Rosowsky (2011)	Concrete highway overpass bridges	As-built seismically designed bridges	S_a	Time-dependent fragility models due to corrosion
	Mackie and Stojadinovic (2007)	Concrete highway overpass bridges	As-built seismically designed bridges	$S_{a,T=1}$, CAD	Fragility models dependent on *IM* and the force reduction factor
	Dukes (2013)	MSC box	As-built bridges	PGA, $S_{a,T=1}$	Fragility models dependent on *IM* and bridge material and geometrical parameters
NEUS	Agrawal et al. (2011) and Pan et al. (2010)	MSSS steel, MSC steel	As-built conventionally designed and retrofitted bridges	PGA	Fragility models dependent on *IM*
	Seo and Linzell (2012)	Horizontally curved steel I-girder	As-built conventionally designed bridges	PGA	Fragility models dependent on *IM*
Entire U.S.	AmiriHormozaki et al. (2015)	Horizontally curved steel I-girder	As-built conventionally and seismically designed bridges	PGA, $S_{a,T=1}$	Fragility models dependent on *IM* and the central angle of the bridge

Table 1. (*Continued.*)

Location	Reference	Bridge typology	Design/retrofit conditions	IM	Characteristics
Regional risk assessment platforms	Hazus-MH (2011)	28 classes of bridges	As-built conventionally and seismically designed bridges	PGA, PGD	Fragility models dependent on IM
	MAEviz (Mid-America Earthquake Center 2006)	MSSS steel, MSSS concrete, MSC steel, MSC concrete, SS steel, SS concrete	As-built bridges conventionally designed	PGA	Fragility models dependent on IM

Note: CAD = cumulative absolute displacement; CSUS = Central and Southeastern United States; MSC = multispan continuous; MSSS = multispan simply supported; NEUS = Northeastern United States; PGA = peak ground acceleration; PGD = permanent ground deformation; S_a = spectral acceleration at the natural period of the system; $S_{a,g}$ = spectral acceleration at the geometric mean of periods in the longitudinal and transverse directions; $S_{a,T=1}$ = spectral acceleration at a period equal to 1 s; SS = single-span; WUS = Western United States.

The models discussed so far were derived for pristine bridges by making the implicit assumption that fragility remains unchanged during the service life of a bridge. However, as discussed earlier, the time-dependent deteriorating effects of aging and corrosion can significantly affect the vulnerability of a bridge. Despite the fact that incorporation of these effects in fragility modeling has started attracting more attention during the last decade (Choe et al. 2008, 2009; Sung and Su 2011; Zhong et al. 2012), the relevant research efforts focusing on bridge classes common in the CSUS region are rather limited (Ghosh and Padgett 2010; Ghosh 2013). Ghosh and Padgett (2010) generated time-dependent fragility functions for MSC steel girder bridges considering the deteriorating effect of corrosion due to chloride-laden deicing salts on the steel reinforcement of the columns as well as on the steel bearings. Similar to Ghosh and Padgett (2010), Ghosh (2013) developed and proposed time-dependent fragility models due to aging and deterioration for MSSS concrete girder bridges. The time-dependent effect of corrosion to various susceptible components such as column piers, bearing dowels, and pads was investigated, and exposure conditions such as deicing salt, marine splash zone, and atmospheric zone exposures were taken into account. Both of these studies adopted the lognormal model of Eq. (1) with fragility parameters being dependent on time t along the bridge's service life. Fig. 2(a) illustrates the deteriorating impact of corrosion on the fragility of a MSC steel girder bridge for different times over the service life of the structure (Ghosh and Padgett 2010).

Ghosh et al. (2013) used surrogate modeling techniques, series systems assumption, and component limit states from Nielson and DesRoches (2007a) to determine system failure, and proposed parameterized fragility models for the extensive damage state for non-seismically designed MSSS concrete girder bridges, which is a representative class in the CSUS. The parameterization of the fragility involved characteristic material, modeling, and geometrical parameters that affect the seismic performance of the bridge and the deterioration-affected bridge structural parameters. The latter feature is important since it facilitates the application of the proposed fragility model to aging bridges. The adoption of surrogate modeling methods to parameterize the fragility functions led to a different functional form than Eq. (1) for the proposed models derived using logistic regression (Rokneddin et al. 2014)

$$P(DS_d|IM,\mathbf{X}) = \frac{e^{g(IM,\mathbf{X})}}{1+e^{g(IM,\mathbf{X})}} \tag{2}$$

where $g(IM,\mathbf{X})$ = regression function consisting of a linear combination of the predictor variables IM and $\mathbf{X}$, whereas d in this study corresponds to the extensive damage state. Following a similar approach, Kameshwar and Padgett (2014) proposed a parameterized fragility model quantifying the probability that MSSS concrete girder bridges in South Carolina exceed the complete damage state. The same functional form as in Eq. (2) was adopted with the difference that a nonlinear regression function $g(IM,\mathbf{X})$ was used consisting of higher-order terms of the predictor variables and their cross-terms to account for interaction and higher-order effects. Figs. 2(b and c) show the parameterized fragilities developed by Kameshwar and Padgett (2014) as a function of span length and PGA for MSSS concrete bridges with 3 and 4 spans, respectively. It is clearly observed that variation of bridge structural parameters can significantly alter the fragility characteristics.

Western United States

Various studies are available in the relevant literature regarding fragility modeling in the Western United States (WUS). These include models investigating the effect of the temporal evolution of seismic

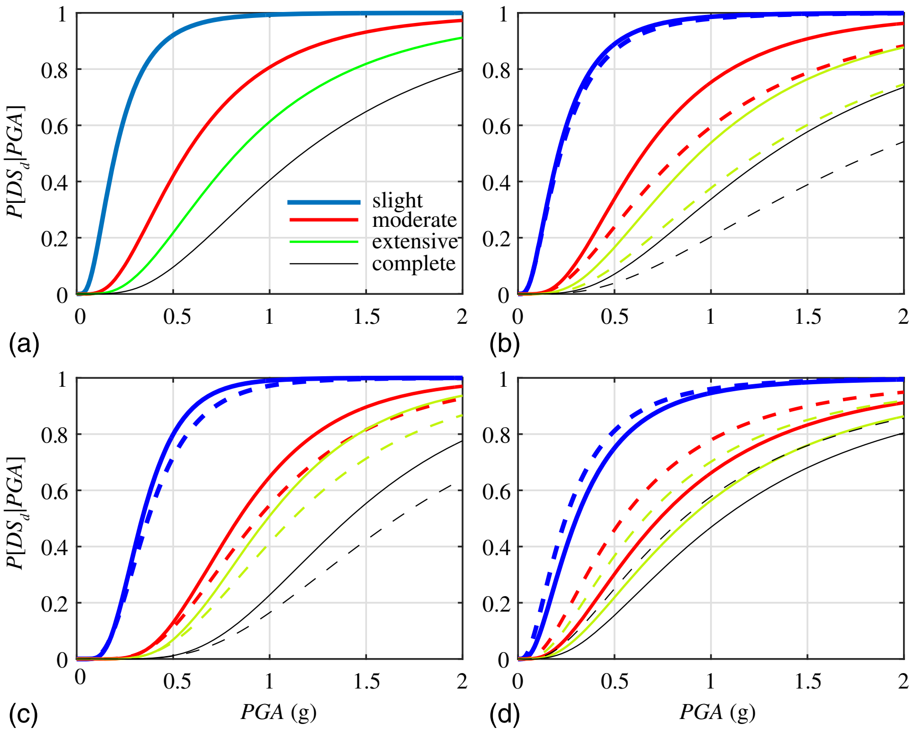

Fig. 1. Fragility curves for (a) as-built bridges (data from Nielson and DesRoches 2007a); (b) as-built (solid lines) and retrofitted with steel jackets (dashed lines) bridges (data from Padgett and DesRoches 2009); (c) as-built conventionally (solid lines) and seismically designed (dashed lines) bridges (data from Ramanathan et al. 2012); (d) as-built conventionally designed bridges for 45° skew angle (dashed lines) and without skew (solid lines) (data from Yang et al. 2015); all cases correspond to MSSS concrete bridges in CSUS

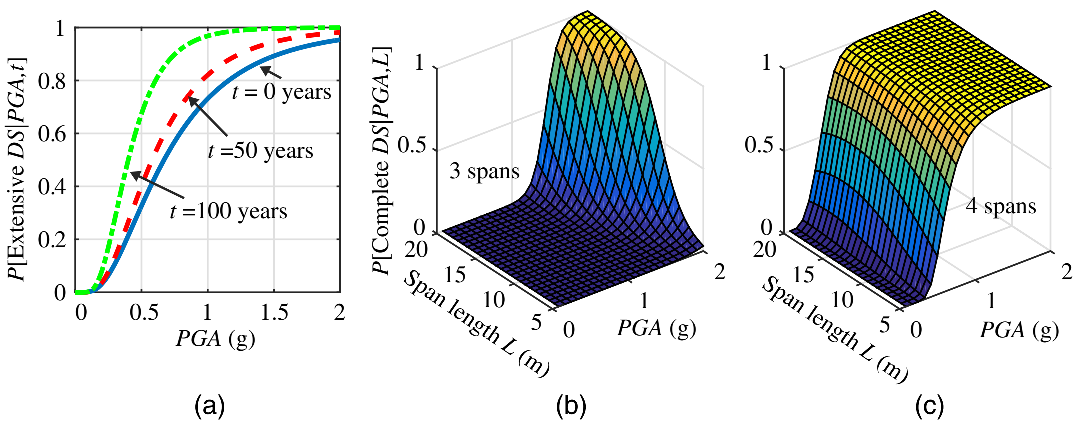

Fig. 2. (a) Time-dependent fragilities for MSC steel girder bridges susceptible to corrosion (data from Ghosh and Padgett 2010) in CSUS; parameterized fragilities as a function of span length and PGA for MSSS concrete bridges in South Carolina with (b) 3 spans; (c) 4 spans (data from Kameshwar and Padgett 2014)

design provisions and detailing standards in typical bridge classes (Ramanathan 2012; Ramanathan et al. 2015); the impacts of bridge skewness (Zakeri et al. 2013a, b), liquefaction (Zhang et al. 2008), and bridge curvature (AmiriHormozaki et al. 2015; Pahlavan et al. 2016); as well as the influence of corrosion (Choe et al. 2009; Gardoni and Rosowsky 2011) and the parameterization of the proposed fragilities with respect to structural bridge parameters (Mackie and Stojadinović 2007; Dukes 2013). It is stressed here that although there have been a significant number of structure specific fragility studies on WUS bridges (Kim and Feng 2003; Kim and Shinozuka 2004; Banerjee and Shinozuka 2007, 2008, 2011; Torbol and Shinozuka 2012; Billah et al. 2012; Bhatnagar and Banerjee 2015), the review is limited to fragility models for bridge portfolios.

Zhang et al. (2008) derived fragility curves for six classes of older (i.e., designed pre-1971) multispan straight bridges in California (Table 1) using incremental dynamic analysis (IDA) (Vamvatsikos and Cornell 2002) that does not require any a priori assumption to be made in terms of the probabilistic distribution for the seismic demand as an alternative to the commonly used PSDM involving regression (Nielson and DesRoches 2007b). Definition of damage states at the bridge system level was performed through

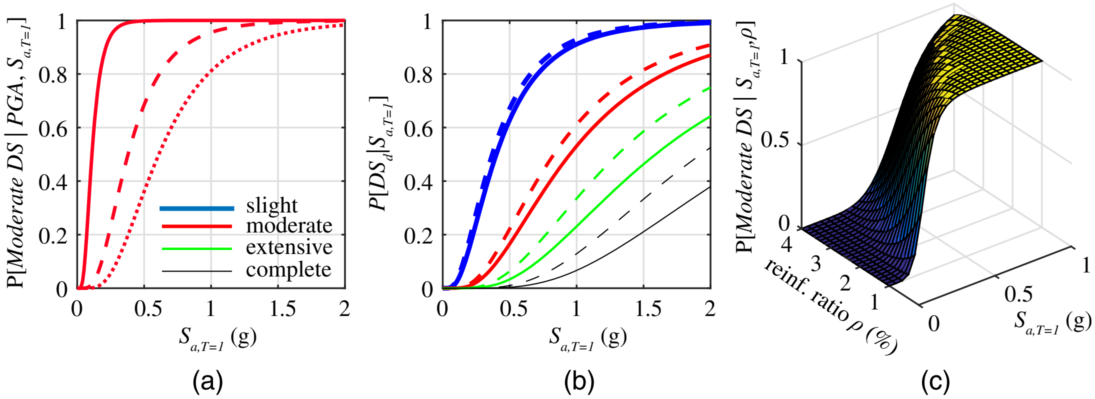

Fig. 3. Fragility curves: (a) for multicolumn bent MSC box girder bridges designed pre-1971 (solid line), 1971–1990 (dashed line) and post-1990 (dotted line) in California (data from Ramanathan et al. 2015); (b) for as-built seismically designed straight (solid lines) and curved with 60° central angle steel I-girder bridges (dashed lines); (c) parameterized fragility as a function of longitudinal column reinforcement ratio ρ and $S_{a,T=1}$ for single-column bent MSC box girder bridges in California (data from Dukes 2013)

a weighted summation of the component damage states using weight factors reflecting their relative importance. The adoption of IDA allowed expressing fragilities either through a lognormal [Eq. (1)] or a normal CDF. Additionally, in the same study, fragility curves considering liquefaction-induced lateral spreading were developed using equivalent static analysis and the first-order second-moment (FOSM) method (Ang and Tang 1984).

Ramanathan (2012) and Ramanathan et al. (2015) investigated the influence of the evolution of seismic design principles and details on the seismic performance of four typical California bridge classes (Table 1) over three significant seismic design eras (pre-1971, 1971–1990, and post-1990). Bridge system fragilities were developed using the series system assumption for the four common damage states in which the individual components are combined in a way that aligns with *Caltrans* details and operational experience. For capturing the effect of the temporal evolution of design and detailing standards on the fragilities of the examined bridge types, several subclasses were identified for each class based on significant bridge structural features such as bent type, abutment seat width, and gap size between the girder and the abutment backwall. The conventional form of Eq. (1) was adopted with fragility parameters that depend on the bridge classes, subclasses, and design era. Fig. 3(a) highlights the effect of the level of seismic design on the fragility of multicolumn bent MSC box girder bridges over the three seismic design eras in California (Ramanathan et al. 2015). It is noted that the fragilities derived in these studies (Ramanathan 2012; Ramanathan et al. 2015) were a part of a pilot project and are the subject of ongoing refinement prior to deployment for regional risk assessment.

Abdel-Mohti and Pekcan (2013) developed fragility curves for the four standard damage states for posttensioned reinforced concrete box-girder highway bridges with moderate-to-large skew angles in California using IDA (Vamvatsikos and Cornell 2002). The proposed fragilities were expressed mathematically through the lognormal [Eq. (2)] or normal CDF. Zakeri et al. (2013a) investigated the effect of skew angle on the seismic fragility for various subclasses of seismically and nonseismically designed single-frame concrete box-girder bridges, which are common in California. The proposed fragilities corresponding to the four conventional damage states adopted the lognormal functional form of Eq. (1) and were constructed for different distinct skew angles following the methodology in Nielson and DesRoches (2007b). Another study conducted by the same authors (Zakeri et al. 2013b) extended their fragility models to bridges of the same type retrofitted with 10 different strategies.

The considerable number of horizontally curved bridges in the bridge inventory of the United States has recently led researchers to study their seismic vulnerability. Pahlavan et al. (2016) investigated the effect of bridge deck radius on the vulnerability of a subclass of horizontally curved multicolumn concrete box girder bridges in California and developed fragility curves for different values of the deck radius adopting the commonly used methodology in Nielson and DesRoches (2007b), whereas using the same bridge fragility methodology AmiriHormozaki et al. (2015) proposed fragility models for horizontally curved I-girder highway bridges across the entire United States. In the latter study, both nonseismically and seismically designed bridges were considered and the lognormal fragility parameters of Eq. (1) were expressed as a function of the central angle that was used as a parameter describing the degree of curvature of the bridges. Fig. 3(b) presents fragility curves for straight and horizontally curved steel I-girder bridges (AmiriHormozaki et al. 2015). It is evident that horizontally curved bridges exhibit increased vulnerability compared with straight ones.

Moving now to fragility models that consider aging effects such as corrosion, Choe et al. (2009) using a Bayesian methodology (Gardoni et al. 2003) developed probabilistic models to predict flexural deformation and shear column demand of typical seismically designed highway overpass bridges in California. The combination of these models with probabilistic capacity models for reinforced concrete columns that capture the effect of reduced area of steel reinforcement due to corrosion generated by the same authors (Choe et al. 2008), facilitated the development of time-dependent bridge fragility estimates considering flexural deformation and shear failure modes, as well as their combination. Gardoni and Rosowsky (2011) developed probabilistic models for the ratio between the fragility of a corroded typical highway overpass bridge in California at a specified time of its service life and the fragility of the pristine bridge for column deformation and shear failure modes. These ratios defined as fragility increment functions, accounted for the loss of diameter of steel bars due to corrosion, as well as the effects of increasing uncertainty over time. Also, these ratios facilitated a computationally efficient way to estimate the fragility of deteriorating bridges at any time of interest without requiring any extra reliability analysis once the fragility of the pristine bridge is known.

In one of the earliest efforts to parameterize fragilities for bridges in the United States, Mackie and Stojadinović (2007) developed fragility models based on time-history analysis for typical new overpass highway bridges in California complying with the *Caltrans* seismic design criteria. The influence of the variation of span length, pier height, material properties, amount of longitudinal and transverse reinforcement, and soil stiffness was incorporated in the fragility models through expressing the fragility parameters [i.e., median and logarithmic standard deviation in Eq. (1)] as a function of the force-reduction factor of the bridge for three damage states related only to pier damages using two different *IM*s (Table 1). In a more recent study, Dukes (2013) developed parameterized fragility models of two typical bridge classes in California for the four common damage states. Two different *IM*s were chosen to describe the earthquake ground motion, whereas the bridge parameters comprising the vector **X** that parameterizes the model consisted of material and geometrical variables such as reinforcement ratios or span length. The logistic regression based functional form in Eq. (2) was adopted using a linear $g(IM, \mathbf{X})$ function. Fig. 3(c) shows the parameterized fragility as a function of longitudinal column reinforcement ratio and $S_{a,T=1}$ for single-column bent MSC box girder bridges in California (Dukes 2013). Similar to Fig. 2(c), the influence of bridge structural parameters on the fragility is apparent.

Northeastern United States

In contrast to the WUS and CSUS, there are limited research efforts regarding fragility modeling of bridges in the Northeastern United States (NEUS). The available studies have focused on developing fragility curves for as-built and retrofitted typical bridge classes in the region (Pan et al. 2010; Agrawal et al. 2011), and horizontally curved steel I-girder bridges (Seo and Linzell 2012). In particular, fragilities were proposed for as-built and retrofitted typical MSC (Agrawal et al. 2011) and MSSS steel girder bridges (Pan et al. 2010) in New York and the NEUS for four damage states. The functional form of the developed fragilities is the same as in Eq. (1) with lognormal parameters that depend on the retrofit condition (as-built or retrofitted), retrofit measure, and damage state. Seo and Linzell (2012) developed fragility curves of horizontally steel I-girder bridges located in Pennsylvania, New York, and Maryland for four damage states using response surface metamodels (Franchin et al. 2003) in conjunction with Monte Carlo simulation.

Seismic Fragility Models Currently Adopted in Regional Risk Assessment Packages

The Hazus technical manual (Hazus-MH 2011) provided fragility models for 28 classes of bridges that differentiate between the unique bridge characteristics found in the National Bridge Inventory (FHWA 1995). The classification was performed based on the following criteria: seismic design, number of spans, structure type, pier type, abutment type and bearing type, and span continuity. It should be noted that not all of these 28 classes correspond to different fragility models. For example, even though continuous concrete and steel bridges are categorized in different classes, their corresponding fragilities are the same. Two types of fragility models, originally developed by Basöz and Mander (1999), for the four damage states with the form of Eq. (1) were provided corresponding to ground motion shaking and ground failure. In this approach, the capacity spectrum method (Freeman 1998) was adopted for estimation of the median values m_d, whereas a constant value $\beta_d = 0.6$ for the logarithmic standard deviation was used based on observed data (Basöz and Kiremidjian 1998). The open-source platform for regional risk assessment *MAEviz* (Mid-America Earthquake Center 2006) has incorporated fragilities for six typical bridge classes (Table 1) and the same four aforementioned damage states. These fragility curves have been originally developed in DesRoches et al. (2003), Choi et al. (2004), and Padgett (2007). The functional form of the developed fragilities is the same as in Eq. (1) with median and logarithmic standard deviation values that depend on the bridge class.

Restoration Models for Earthquake Hazard

Although the literature on seismic bridge restoration models is not as rich as for the case of fragility models, primarily due to the inherent difficulties (subjectivity, human factor, high uncertainty related to postevent available resources, etc.) related to the quantification of restoration processes, there have been a few notable studies addressing this issue (Shinozuka et al. 2003; Padgett and DesRoches 2007; Hazus-MH 2011; Bocchini et al. 2012). Furthermore, since the concept of resilience of structures and infrastructures has become popular in the civil engineering community recently (Bocchini and Frangopol 2010, 2011; Cimellaro et al. 2010a), there is an increased attention oriented towards development of new and more versatile restoration/functionality models (Bocchini et al. 2012). In general, two types of seismic restoration models for bridges in the United States have been proposed, one describing the probability that the bridge will be completely repaired (i.e., it will gain 100% functionality) given the bridge's damage state and the time after the seismic event (Shinozuka et al. 2003), whereas the other quantifies the percentage of the bridge's functionality conditional on the damage state and the time after the earthquake occurrence (Padgett and DesRoches 2007; Hazus-MH 2011; Bocchini et al. 2012).

The latter one, being more versatile since it tries to capture the evolution of the bridge's functionality, can be expressed through the following general form (Bocchini et al. 2012):

$$Q(t > t_0) = Q_r + H(t - t_0 - \delta_i) R\left(\frac{t - t_0 - \delta_i}{\delta_r}\right)(Q_t - Q_r) \quad (3)$$

where $Q(t)$ = functionality; t_0 = time of occurrence of the seismic event; and Q_r = residual functionality after the event occurrence. $H(\bullet)$ = Heaviside step function; Q_t = functionality reached at the end of the recovery process; δ_r = duration of the recovery; δ_i = idle time between the occurrence of the seismic event and the beginning of the recovery process; and $R(\bullet)$ = restoration function describing the profile of the recovery process that depends on the actual model used. A characteristic illustration of the variation of functionality with respect to time is shown in Fig. 4(a). Appropriate selection of the recovery function $R(\bullet)$ and calibration of the parameters defining it can support modeling of different restoration profiles. It is noted that the general form of Eq. (3) is not constrained to modeling recovery processes after occurrence of earthquakes and is applicable to other hazards.

Based on expert opinion survey data (ATC 1985), Hazus (Hazus-MH 2011) developed restoration curves for highway bridges that have the functional form of a normal cumulative distribution function

$$R(t) = \Phi\left(\frac{t - m_{t,d}}{\sigma_{t,d}}\right) \quad (4)$$

where $m_{t,d}$ and $\sigma_{t,d}$ = mean and standard deviation of the restoration functions (in units of time) for each one of the considered damage states, respectively. It is noted that the proposed restoration functions do not differentiate among the various bridge classes identified in Hazus (Hazus-MH 2011), rather the same models are used for all highway bridges. Fig. 4(b) shows the restoration curves for highway bridges proposed in Hazus-MH (2011). Padgett and DesRoches (2007), using data collected from an expert opinion survey of CSUS Department of Transportation bridge inspectors and officials through an expert opinion survey, developed functionality probability matrices that yield the probability that the bridge will achieve specific values of functionality at certain times after earthquake occurrence conditional on the extent of damage to different bridge components. Ultimately, stepwise bridge functionality restoration profiles were constructed, which can be mathematically expressed as (Padgett and DesRoches 2007)

$$Q(t) = \begin{cases} Q_{r,} & t < t_1 \\ Q_{1,} & t_1 \leq t < t_2 \\ Q_{2,} & t_2 \leq t < t_3 \\ Q_{3,} & t_3 \leq t < t_4 \\ Q_{t,} & t \geq t_4 \end{cases} \tag{5}$$

where Q_1 to Q_3 are values of the functionality between Q_r and Q_t achieved at time instants t_1, t_2, t_3, and t_4 corresponding to 1, 3, 7, and 30 days, respectively; whereas Q_i, Q_r, and Q_t depend on the bridge's damage state. In a recent study, Bocchini et al. (2012) proposed a versatile six-parameter sinusoidal recovery process model $R(\bullet)$ that allows representation of very different recovery shapes as a function of the investigated system and type of damage, such as linear, trigonometric (Cimellaro et al. 2010b), and exponential (Kafali and Grigoriu 2005) types of restoration patterns. Fig. 4(c) presents the restoration curves obtained through implementation of the multiparameter sinusoidal model. Comparing these curves with the ones prescribed in Hazus [Fig. 4(b)], the capability of the multiparameter sinusoidal model to describe different and more complex recovery patterns is evident. Although this model exhibits high versatility, the calibration of its parameters can be a challenging task since they depend on the preparedness of the affected community, effort put in recovery, financial and logistic resources, and possible prioritization of critical transportation infrastructure components. Therefore, proper and reliable calibration requires empirical data from past hazard events, expert opinion surveys, and engineering judgment. A detailed description of the mathematical formulation for this model can be found in Bocchini et al. (2012).

Using the identified restoration models, various studies related to quantification and assessment of the seismic resilience of bridges have been conducted. In particular, Decò et al. (2013) using the six-parameter sinusoidal-based recovery model (Bocchini et al. 2012), proposed a probabilistic approach for the pre-event assessment of seismic resilience of bridges including uncertainties associated with expected damage, restoration process, and retrofit costs. Numerous seismic resilience studies have adopted the normal cumulative distribution function model [Eq. (4)] proposed in Hazus (Hazus-MH 2011). More specifically, Venkittaraman and Banerjee (2014) evaluated the effectiveness of seismic retrofit techniques to enhance seismic resilience of highway bridges in California; Karamlou and Bocchini (2015) performed probabilistic resilience analysis for a typical MSSS steel bridge; Dong and Frangopol (2015) presented a probabilistic framework for risk and resilience assessment of bridges subjected to mainshock and aftershock earthquake sequences; Alipour and Shafei (2016) proposed a framework for the analysis of seismic resilience of highway bridge networks with deteriorating components due to aging; and Dong and Frangopol (2016) presented a framework for time-variant loss and resilience assessment of bridges under time-dependent multiple hazards. Using a simple linear recovery model, which can be considered as a special case of the six-parameter sinusoidal model (Bocchini et al. 2012), Bocchini and Frangopol (2010) proposed a methodology for the optimal resilience-based and cost-based prioritization of interventions on bridges distributed along a highway segment; Bocchini and Frangopol (2012) presented a multicriteria intervention optimization procedure for the restoration activities associated with a complex existing bridge transportation network in California; and Chandrasekaran and Banerjee (2015) performed a multiobjective retrofit optimization with the goal to maximize resilience and minimize retrofit cost of a typical bridge in California under the multihazard effect of earthquake and flood-induced scour. Finally, Zhou et al. (2010), using the model describing the probability for complete repair of a bridge (Shinozuka et al. 2003) and the normal cumulative distribution model (Hazus-MH 2011), evaluated the socioeconomic effects of the seismic retrofit of bridges on the Los Angeles area highway transportation network.

Fragility Models for Hurricane-Induced Surge and Wave Hazard

The extent and severity of damage to bridges during recent hurricanes [e.g., Katrina 2004 (Padgett et al. 2008), Ike 2008

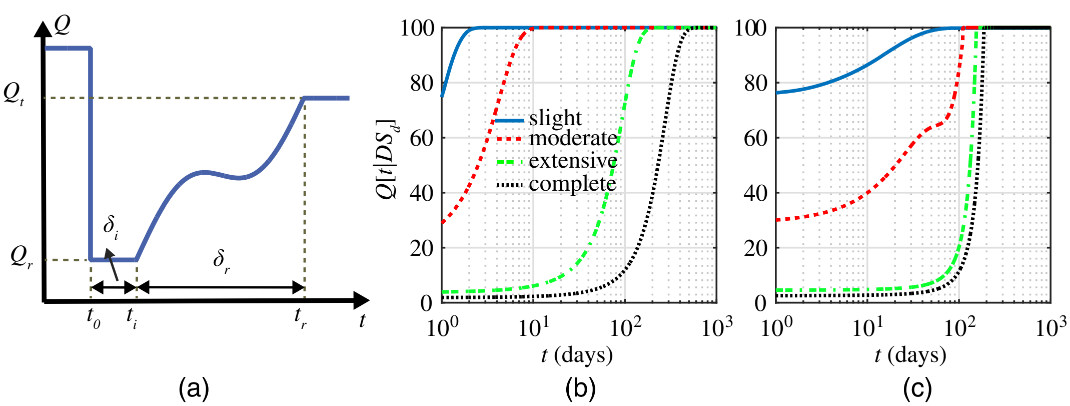

Fig. 4. (a) Illustration of functionality recovery process; restoration curves for highway bridges obtained using; (b) Hazus (2011); (c) the multiparameter sinusoidal model (data from Bocchini et al. 2012)

(Stearns and Padgett 2011), Ivan 2004 (Douglass et al. 2004)] that made landfall in different coastal regions of the United States has stimulated the interest of researchers (Ataei and Padgett 2012; Padgett et al. 2012; Kameshwar and Padgett 2014) to develop fragility models for coastal bridges subjected to hurricane-induced surge and wave hazard. While related efforts have focused more specifically on reconnaissance reporting failure modes (Mosqueda Porter et al. 2007; Robertson et al. 2007) or surge and wave load modeling (Douglass et al. 2006; Schumacher et al. 2008; Bradner et al. 2010), these probabilistic fragility modeling studies offer some of the first models to facilitate hurricane risk assessment of bridges and transportation networks. These models are categorized and discussed in the following subsections based on regional criteria. It is noted that no coastal bridge restoration function specific for hurricane events has been proposed in the literature to date.

U.S. Gulf Coast

Using several data sets regarding observed damages to bridges along the U.S. Gulf Coast due to the 2005 Hurricane Katrina, Padgett et al. (2012) conducted statistical analysis to develop empirical fragilities representing the probability of being in or exceeding a specified damage state given an appropriate hurricane hazard *IM*. In particular, this study developed empirical fragilities for MSSS concrete water crossing bridges in the Mississippi and Louisiana Katrina exposed region in the form of an exceedance probability matrix describing the empirical probability $P(DS|\text{Surge elevation})$ of being in or exceeding a damage state given hurricane surge elevation. The latter parameter serves as the *IM* describing the hurricane hazard. The four considered damage states were defined through appropriate modification of the seismic damage states in Hazus-MH (2011) such that additional failure modes related to hurricane events were included.

Houston/Galveston Bay Area

The recent work by Ataei and Padgett (2012) constitutes the first research endeavor that provided a probabilistic framework for developing analytical fragility models for bridges vulnerable to hurricane-related hazards. The methodology focused on a single failure mode (i.e., damage state) corresponding to bridge deck unseating due to the hurricane-induced wave forces. This particular type of limit state is a very common severe failure mode for simply supported bridges lacking supplemental restraints such as restrainers or shear keys; hence, the fragility models developed are particularly relevant and more applicable to bridges exhibiting this characteristic. Statistical analysis was employed and the failure of each span was determined through a comparison of the vertical resistance (capacity) of the bridge's superstructure with the maximum wave force demand. Ultimately, the failure probability of the bridge as a system was estimated through a series system assumption for the bridge decks. Implementation of this reliability assessment approach to the majority of the bridge inventory of the Houston and Galveston Bay area of Texas resulted in the development of fragility surfaces quantifying the probability of failure of a bridge due to deck unseating conditional on two hurricane intensity measures, wave height H and relative surge elevation Z_c. Furthermore, the developed fragility model was also parameterized with respect to the span mass per unit length, which is a metric that was deemed appropriate for classification of bridges with different structural characteristics. The proposed model was expressed through linear regression as $P(\text{Failure}|H, Z_c) = a + bH + cZ_c$, where a, b, and c are regression coefficients that depend on the span mass per unit length.

South Carolina

Kameshwar and Padgett (2014) proposed a parameterized multihazard risk assessment framework to a portfolio of highway bridges. This framework was implemented to MSSS concrete girder bridges in South Carolina for earthquakes and hurricanes; hence, parameterized fragilities were developed for both hazards. Similar to the hurricane fragility models considered in Ataei and Padgett (2012), a single failure mode corresponding to bridge deck unseating/uplift due to wave forces and the system probability of failure was calculated through a series systems assumption. However, this study identified failure (i.e., deck uplift) through a more rigorous nonlinear dynamic analysis in which the bridge models were subjected to sinusoidal wave force time-histories rather than relying on a simplified static analysis approach. The proposed fragility model quantified the probability of bridge failure due to deck uplift $P(\text{Failure}|S, H, H_B)$ conditional on surge height S and wave height H (i.e., the intensity measures of the hurricane hazard), as well as the height of the bridge H_B measured as the sum of the column and bent height. It is noted that parameterization of the proposed fragility model with respect to the bridge characteristic (i.e., H_B) affecting the performance of the structure subjected to hurricane-induced loads enhances its applicability to a broader range of bridges of this particular type. The proposed parameterized fragility model was expressed using the functional form in Eq. (2) with the following regression function (Kameshwar and Padgett 2014):

$$g(S, H, H_B) = -2.71 - 3.47(H_B - S) + 1.59H + 0.17(H_B - S)^2 + 0.05H^2 \qquad (6)$$

Fig. 5(a) shows the parameterized fragility as a function of surge and wave height for MSSS concrete bridges in South Carolina (Kameshwar and Padgett 2014). Finally, it is worth mentioning that although the aforementioned hurricane-induced coastal flood models are categorized in the individual hazard fragilities, they are conditional on the joint occurrence of two hazard load effects (i.e., surge and wave), reflecting in this way a multihazard feature.

Fragility and Restoration Models for Tsunami Hazard

Existing studies of fragility modeling for bridges subjected to tsunami hazard are in general limited. The majority of the developed fragility models correspond to bridges in Japan and Southeast Asia subjected to the devastating 2011 Tohoku and 2004 Indian Ocean tsunamis. In particular, Akiyama et al. (2013) developed analytical tsunami hazard fragilities based on Monte Carlo simulation for a concrete bridge that was damaged during the 2011 Tohoku earthquake, whereas Shoji and Moriyama (2007) presented empirical fragility curves based on damage data for bridges in Sri Lanka and Sumatra during the 2004 tsunami in the Indian Ocean. The Hazus tsunami methodology (FEMA 2013) developed fragility models through an expert opinion survey approach for highway bridges in the United States subjected to tsunami hazard for the four standard Hazus damage states. These models adopted the lognormal functional form of Eq. (1) with the median parameter corresponding to inundation depth, which was used as the primary tsunami *IM*. However, since the tsunami hazard cannot be adequately described only through inundation depth, the Hazus methodology incorporated the effects of tsunami flow rate, quantified based on the flow velocity, and presence of large debris using modification factors applied to the median parameter of the fragility curves such that the damage state thresholds are reduced as the flow rate increases or large debris is present. It is noted that the proposed fragilities do not differentiate between different bridge classes.

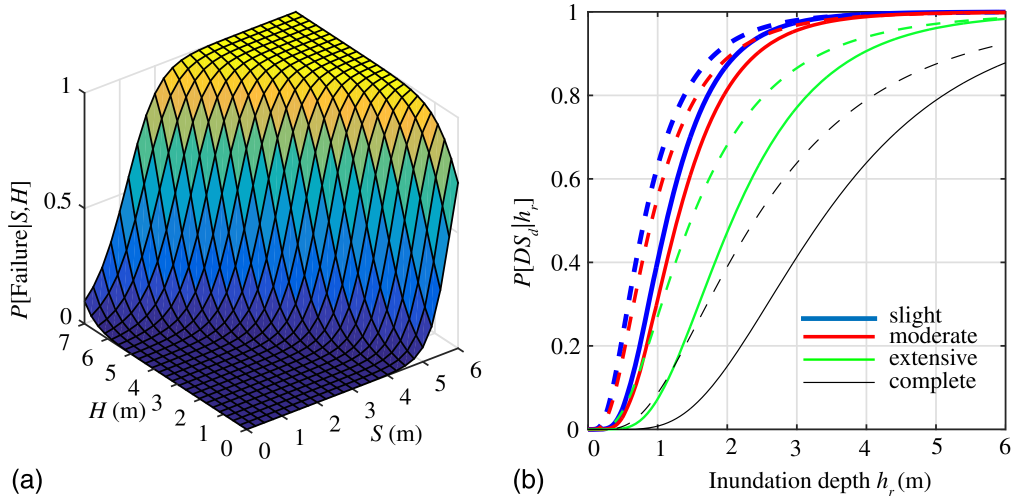

Fig. 5. (a) Parameterized fragilities as a function of surge and wave height for MSSS concrete bridges in South Carolina (data from Kameshwar and Padgett 2014); (b) fragility curves for bridges subjected to tsunami hazard for low (solid lines) and moderate (dashed lines) flow rates (data from FEMA 2013)

An assumed default height measured from the top of foundation to the centerline of the bridge deck was used for the development of the fragility curves, and linear scaling for the median parameters was suggested for bridges with different heights. The logarithmic standard deviation of the fragility that essentially introduces the uncertainty in the model was estimated by combining the uncertainties stemming from the demand (i.e., variability of the tsunami *IM*) and the capacity threshold determining each damage state through the common assumption that demand and capacity are statistically independent random variables. Therefore, nominal values for the logarithmic standard deviations of demand and capacity were proposed corresponding to low flow rate and absence of debris, whereas increased values for the standard deviation of the demand were suggested for higher flow rates or when debris may be present. Fig. 5(b) presents fragility curves for bridges subjected to tsunami hazard for low (solid lines) and moderate (dashed lines) flow rates (FEMA 2013). It can be observed that the higher flow rate increases the vulnerability of bridges subjected to tsunami hazard.

Regarding the tsunami restoration models, the Hazus tsunami methodology (FEMA 2013) based on the expert opinion survey used for the fragility derivation, proposed a set of values quantifying the probability of loss of functionality of a bridge given the estimated damage state. Additionally, values for the expected restoration time needed for the bridge to become fully functional given the damage state were proposed. For example, a bridge being in the extensive damage state exhibits 40% probability and it would require 30 days to be fully operational. These values correspond to a more simplified restoration model compared to the ones discussed for the seismic hazard in previous section, since only estimates associated with the extreme case of a bridge being 100% functional for a given damage state are proposed rather than modeling various functionality levels during the recovery process of a damaged bridge. It is noted that the estimates of functionality do not account for effects from the interaction of other systems, such as damaged roadways and loss of power or communications, whereas the restoration time estimates do not incorporate the effects from surge demand or other regional impacts affecting the availability of materials and labor and technical engineering personnel necessary for inspection, repair design, review by local jurisdictions, and construction.

Fragility Models for Riverine Flood Hazard

Similar to the tsunami hazard, fragility models of highway bridges for riverine flood are very limited. The Hazus methodology for quantifying losses due to flood hazard (Hazus-MH 2009) reported fragilities of highway bridges in a discrete form of probability matrices using data from the National Bridge Inventory database (FHWA 1995). It is noted that because of the lack of comprehensive bridge damage data due to riverine flood hazard, the proposed empirical fragilities are estimates that could be calibrated for different bridge deck materials (e.g., concrete, steel, wood) in future releases of the Hazus flood methodology manual (Hazus-MH 2009) if more damage data become available. However, these updates have not taken place yet. In particular, Hazus provided probability of failure values $P(\text{Failure}|IM, SV)$ as a function of the flood return period (corresponding to the *IM* of the flood hazard) and the scour vulnerability (*SV*) rating assigned to the bridge after appropriate scour evaluation (Richardson and Davis 2001). Therefore, essentially a single damage state was prescribed corresponding to failure defined to be the loss of functionality due to flood/scour damage and qualitatively represents damages that would lead to losses equal to 25% of the replacement cost (Hazus-MH 2009). The latter value is an estimate, which is qualitatively associated with damage levels such as scour/undercutting of a single pier or collapse of a span. Different scour vulnerability levels (i.e., *SV*) and flood hazard intensity levels were prescribed in the proposed methodology and a distinction was made regarding the susceptibility of bridges to flood-induced failure based on the bridge's span type. Stein et al. (1999), using the National Bridge Inventory database (FHWA 1995), developed a model that provides estimates for the probability of scour failure based on waterway adequacy, which is an indicator for the extent that the bridge restricts the channel, characteristics of the bridge's route (e.g., interstate or expressway), scour vulnerability, and channel protection. The failure mode corresponds to occurence of scour. Finally, more recently, Turner (2016) developed fragility curves for eight existing bridges in Colorado considering a single damage state corresponding to structural failure of the bridge superstructure due to riverine flood-induced hydrodynamic lift forces. Similar to the hurricane-induced surge and wave (i.e., coastal flood) hazard, no restoration model for riverine flood is found in the published literature.

Fragilities for Concurrent and Cascading Multihazard Events

In addition to individual hazards, highway bridges are frequently exposed to multihazard events and/or effects occurring either concurrently (e.g., earthquake and flood-induced scour) or in a cascading manner (e.g., mainshock-aftershock earthquake sequences or earthquake triggered tsunamis). The increased vulnerability that these combinations of hazards can impose to bridges has recently stimulated the interest of researchers towards developing fragility models for multihazard scenarios. In the following subsections, fragility models for concurrent and cascading hazards such as earthquake in the presence of scouring effects and earthquake sequences, respectively, are discussed. It is noted that the focus of the discussion on these particular multihazard fragility models was dictated by the lack of the other relevant models in the literature.

Concurrent Joint Hazard Fragilities: Earthquake and Flood-Induced Scour

Although the likelihood of simultaneous occurrence of earthquake and flood during the lifetime of a bridge is low, bridges located in earthquake-prone and flood-prone regions can be pre-exposed to flood-induced scour when subjected to seismic events resulting in a potential amplification of the structure's vulnerability. Hence, earthquake and scour can be considered as a concurrent joint noncorrelated hazard combination and a few notable joint fragility studies have been published recently (Alipour and Shafei 2012; Alipour et al. 2013; Banerjee and Ganesh Prasad 2013; Ganesh Prasad and Banerjee 2013; Wang et al. 2014a, b; Yilmaz et al. 2016; Gehl and D'Ayala 2016). These fragilities are expressed as

$$\text{Fragility} = P[DS_d | IM, IM_{\text{scour}}] \tag{7}$$

where IM_{scour} = intensity measure of the scour effect, with scour depth at bridge foundation being the most commonly adopted measure. Eq. (7) expresses the probability that a specific damage state DS_d is met or exceeded conditional on given values of IM and IM_{scour}. Similar to the fragilities for bridges subjected only to seismic hazard, the lognormal functional form is usually adopted for the earthquake and scour fragility models expressed as

$$P[DS_d | IM, IM_{\text{scour}}] = \Phi\left[\frac{\ln(IM) - \ln(m_{d,sc})}{\zeta_{d,sc}}\right] \tag{8}$$

where $m_{d,sc}$ and $\zeta_{d,sc}$ = lognormal parameters under the combined effect of earthquake and scour hazards.

In particular, Alipour and Shafei (2012) and Alipour et al. (2013) developed fragility curves for different variations of multispan concrete box girder bridges in southern California using nonlinear time-history analysis. The fragility curves corresponding to the four common damage states are described through Eq. (8) for different levels of scour depth using the methodology proposed by Shinozuka et al. (2000). Following the same methodology, Ganesh Prasad and Banerjee (2013) and Banerjee and Ganesh Prasad (2013) derived earthquake and scour fragilities for multispan concrete box girder bridges with various number of spans located in Sacramento County, California. More recently, Yilmaz et al. (2016) generated fragility curves and fragility surfaces for the four common damage states for two California concrete box girder bridges. Following a different approach than in the previous studies, Yilmaz et al. (2016) recognized that scour depth is a consequence of the flood hazard, rather than the source of the hazard itself. Hence, flood return periods and peak annual flow discharge were adopted as IM_{scour} for the proposed fragility curves and surfaces, respectively. This approach allowed consideration of varying scour depths across the multiple piers of a bridge. Wang et al. (2014a) developed fragility models corresponding to the complete damage state for different components of three common bridge classes across the United States, seismically designed single-frame concrete box girder bridges with integral piers in California, older nonseismically designed MSSS concrete girder bridges typical in the CSUS, and recently designed MCS concrete girder bridges. As opposed to the other studies discussed, a PSDM based on regression similar to the one in Nielson and DesRoches (2007b) was used. Wang et al. (2014b) derived fragility surfaces corresponding to the complete damage state for the same class of concrete box bridges in California. These fragilities were constructed through a multihazard PSDM based on regression that was also proposed in the study. Finally, Gehl and D'Ayala (2016), using system reliability methods and Bayesian networks, derived fragility surfaces for a MSSS concrete bridge expressed as a function of peak ground acceleration and flow discharge to describe the intensity of the seismic and flood hazard, respectively.

Cascading Hazard Fragilities: Mainshock–Aftershock Earthquake Sequences

Past experience (e.g., seismic events in New Zealand in 2011) as well as probabilistic analysis (Kumar and Gardoni 2011) have shown that the likelihood of a structure experiencing sequences of earthquake events such as mainshock and aftershocks is significant. In such cases, it is likely that a bridge sustaining damages due to a mainshock event will not be repaired when subsequent aftershock events occur; hence, its vulnerability can considerably increase, leading to catastrophic consequences. Therefore, it is important that fragility models can capture the effect of cumulative structural damage on bridges due to cascading hazards such as mainshock–aftershock sequences. However, this topic has received limited attention, with the existing studies focusing either to bridges outside of the United States (Franchin and Pinto 2009; Alessandri et al. 2013) or to specific case studies (Dong and Frangopol 2015; Ghosh et al. 2015). Kumar and Gardoni (2014) developed fragility models for single-column seismically designed bridges in California incorporating the impact of structural degradation due to successive earthquakes, not only on the structural capacity but also on the system demand. The former effect was taken into account through development of probabilistic degradation models using Bayesian updating for the affected structural parameters. Combined then with appropriate probabilistic seismic demand (Gardoni et al. 2003) and capacity (Kumar and Gardoni 2011) models, fragilities for flexural and shear deformation failure models were derived, expressed as

$$\text{Fragility} = P\left[\text{Failure} | \mathbf{S_a}^{(m)}, S_a^{(m+1)}\right] \tag{9}$$

where $\mathbf{S_a}^{(m)} = \{S_a^{(i)}; i = 1, \dots, m\}$ is the vector of S_a of the past m earthquakes and $S_a^{(m+1)}$ is the S_a of an anticipated future seismic event.

Summary and Future Research Needs

This paper provides a comprehensive state-of-the-art literature review of existing fragility and restoration models for common highway bridge classes across the United States subjected to earthquakes, hurricanes, tsunamis, and riverine floods, as well as on characteristic cases of concurrent and cascading hazards. The focus of the study is on relevant models corresponding to bridge typologies that are widely used in different U.S. regions rather than on individual bridge structures, to facilitate adoption and implementation of the identified models within computational modeling platforms such as NIST-CORE for risk and resilience analysis and assessment on a regional scale. The paper conducted a detailed identification and classification of existing highway bridge fragility and restoration models based on hazard, geographic region, structural typology, and sophistication level related criteria. Moreover, the study discussed the salient features of the reported models as well as key aspects of the methodologies utilized for their development. The information offered in this review paper can facilitate a comparison of different approaches adopted for development of fragility and restoration models, as well as their availability across a range of hazard cases.

There are extensive literature collections related to seismic fragility models for highway bridges, whereas regarding restoration models a few notable studies exist, although the available models are not as numerous as the fragility models. In particular, the existing seismic fragilities cover a wide range of bridge classes common in different regions across the country, addressing aspects such as design and retrofit conditions, bridge curvature and skewness, liquefaction phenomena, aging, and parameterization based on important bridge structural characteristics. The literature review indicated the versatility and the merits gained through the parameterized and time-dependent highway bridge fragility models and that new research efforts should focus on these models. However, currently these models have been developed for relatively limited bridge classes and damage states. Therefore, future research should focus on further improvement of the current ones, potentially through implementation of more advanced surrogate modeling techniques and extension to a broader range of bridge typologies. Furthermore, in general, it was observed that the existing studies for seismic bridge fragilities addressed almost exclusively mainshock events. Hence, there is a need to investigate the effect of potential structural degradation and accumulation of damage on bridge fragility models due to mainshock-aftershock earthquake sequences for a broader range of bridge classes and damage states, as well as earthquake mainshock and other cascading hazards, such as earthquake and tsunami hazards. In terms of restoration models, although a recently proposed multiparameter model appears to represent different recovery profiles depending on the damage state, a comprehensive calibration of such a model for covering various bridge classes is still missing in the existing literature. Additionally, the literature review of available restoration models underlined the need for appropriate models that are able to reflect differentiations in restoration time with respect to a basic set of bridge characteristics, such as the size of the bridge. Moreover, the existing bridge restoration models are independent of the state of the transportation network after the event occurrence; hence, development of models that account for the potential recovery process variation due to the number and the extent of other damaged bridges or constituent components (i.e., roadways, tunnels, etc.) in the network can greatly facilitate accurate resilience quantification.

The existing research regarding fragility and restoration modeling due to hurricane-induced surge and waves on coastal bridges is not as broad and rich as for the models corresponding to the seismic hazard. There are a few available studies related to the development of fragility models covering a limited number of different typologies and damage states, while there is a lack of restoration models specifically for this type of hazard. The review of the relevant fragilities stressed the need for an extension of the existing parameterized fragility models to cover a broader range of typical bridge classes as well as to include additional damage states related to surge and waves, such as bridges with strong connections between superstructure and substructure for which other modes of failure, due to the increased forces transmitted from deck to pier, can be predominant. Furthermore, empirical evidence of bridge deck vulnerability to hurricane surge and waves in Florida, Alabama, Mississippi, and Louisiana following Hurricanes Ivan (2004) and Katrina (2005) revealed two key responses that need to be addressed in fragility models. First, bridge decks at lower elevations survived due to submergence early in the storm surge event when the wind-generated waves were relatively small. Second, bridge decks above the elevation of the crest of the significant wave height (i.e., average of the highest one-third of the waves in the sea state) survived with minimal to no damage. So even though the decks were impacted by some number of larger waves, they were infrequent. Recently published methodologies for estimating wave loads on bridges are dependent upon the bridge deck and storm surge elevation and are cast in terms of the maximum wave height in the sea state. This dependency presumes that the load attains this value only once or that it is continuous in nature. Neither is strictly true, but existing methodologies for load prediction and fragility do not account for either the statistical distribution of wave heights in an irregular sea state or the time-dependent nature of these processes. Waves in an irregular sea state are Rayleigh distributed processes; hence, not only are the representative statistical measures known, but given reasonable event duration, the number of individual wave events can be estimated. Such information should be incorporated into load estimation and fragility models in a meaningful way.

The literature review with respect to fragilities of highway bridges to tsunami hazards revealed that there are very few publications on the topic. Most of the work is based on reconnaissance observations following the 2004 Indian Ocean Tsunami, and the 2011 Tohoku Japan Earthquake and Tsunami. The Hazus-MH tsunami technical manual has proposed fragility functions, which are based on expert judgment. While this is an important first step, due to the lack of data and reports of recent large tsunami events in the United States, simulation-based fragility curves should be developed for prototype U.S. bridges along the coastal communities that can be affected by tsunamis (i.e., California, Oregon, Washington, Alaska, and Hawaii). Furthermore, these models may wish to consider the adoption of more advanced intensity measures such as the momentum flux quantified as the product of the inundation depth with the square of the flow rate, which is a superior predictor of bridge response. Alternatively, tsunami fragility models could be developed by explicitly considering the flow rate as a component of a tsunami vector *IM*.

Regarding riverine flood hazard fragility and restoration models, the existing studies are extremely limited. Future research on this topic should focus on the development of analytical fragility and restoration models for highway bridges that can be particularly vulnerable to riverine flood hazard due to scour effects.

In summary, the review highlighted that the scope of the fragility and restoration modeling related research differs significantly among the various hazard cases. Comprehensive methodologies for the development of analytical models relevant to seismic hazard using advanced methods such as nonlinear time-history analysis have been proposed in the published literature, investigating the effects of a variety of aspects/characteristics on the earthquake performance of highway bridges. Furthermore, damage characterization for the various bridge components in terms of appropriate engineering demand parameters has supported a quantitative and rigorous definition of component damage states. In contrast, fragilities for other hazards such as tsunami and riverine flood relied on approaches based on limited empirical data or expert judgment for their development and damage states were defined qualitatively, hence limiting their applicability. Coastal flood fragilities stand between these two extreme cases, since a few analytical models have been proposed. However, these models as opposed to the ones corresponding to earthquake adopted in general simpler analysis methods and excitation representation, whereas only a single failure mode applicable to a specific type of bridges was considered.

From the preceding discussion it is also evident that, at least for seismic fragilities, component limit states are determined in a relatively detailed and rigorous manner. However, risk and resilience assessment often require system-level fragilities, and even the more advanced currently proposed models rely on assumptions for the definition of bridge-level failure that in general limit their flexibility. Therefore, one of the key challenges in the field of fragility modeling is development of efficient and versatile system-level damage state definitions that are compatible to restoration modeling and can also be augmented to various hazards. Such definitions can ultimately support more accurate and comprehensive multihazard risk and resilience assessment methodologies.

Finally, the complete absence of models related to other hazards that can induce damage to highway bridges, such as strong winds due to hurricane or tornado events, fire, or landslides, underlines the need for future endeavors to address these cases.

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