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Bridge classes for regional seismic risk assessment: Improving HAZUS models

Sujith Mangalathu [a], Farahnaz Soleimani [a], Jong-Su Jeon [b,*]

[a] School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA 30332, USA
[b] Department of Civil Engineering, Andong National University, Andong, Gyeongsangbuk-do 36729, Republic of Korea

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ABSTRACT

Recent efforts are oriented towards the regional risk assessment of bridge inventories by grouping bridge classes which are expected to have similar performance or damage measures during a seismic event. Although HAZUS represents the current state of the art in grouping the bridge classes, it suffers many drawbacks such as grouping based on traditional subjective approach, failure in explicitly addressing the effect of design and geometric attributes, and neglecting the effect of abutment type. A critical review of the HAZUS grouping and associated fragilities highlights the need for a more refined sub-binning of bridge classes for a reliable estimate of the seismic risk. Towards the objective of a performance-based grouping of bridge classes, a new grouping technique rooted in the statistical technique called analysis of variance is suggested in this paper. The approach is used to improve the HAZUS grouping through the case studies of California box-girder bridges. The proposed approach identifies more sub-classes than the HAZUS grouping, and the significance of the proposed grouping is demonstrated through the comparison of fragility curves of three bridge classes identified by the proposed grouping methodology.

1. Introduction

It is a common practice these days to generate fragility curves that are applicable to a class of bridges from a regional risk assessment perspective. The bridges in a particular class (or group) are expected to have similar performance or damage during an earthquake. The identification of bridge parameters that yield distinct seismic performance to bridges is an important step in this procedure. The failure in creating distinct bridge groups leads to a non-realistic estimation of the seismic vulnerability. Such a non-realistic estimation swaps the decisions of agencies involved in post-emergency disaster-management measure. As fragility curves generated for bridge classes are currently employed in post-earthquake disaster management and recovery [1,2], it is critical that bridges in a particular group suffer similar damage and operational consequences.

HAZUS [3] is the most comprehensive document in grouping bridge classes and seismic vulnerability estimation. HAZUS classified bridge classes based on the seismic design, number of spans, span length, bent type, span continuity and span discontinuity, and suggested fragility relationships for the grouped bridge classes. However, the grouping suffers several shortcomings such as not addressing the evolution in seismic design philosophy, the effect of column cross-sections, abutment configurations, to name a few. The fragility relationships suggested in HAZUS are based on simple two-dimensional (2-D) analyses of bridges and do not account for the material, structural, and geometric uncertainties. Many researchers have pointed the need to sub-bin the bridge classes beyond HAZUS [4,5], and a critical review of the HAZUS grouping and fragility relationships is given in the next section. Moschonas et al. [6] developed a classification scheme for Greek bridges according to pier type (single column cylindrical, single column rectangular, multi-column, wall-type), deck type (slab, box-girder, simply supported precast-prestressed beams connected through continuous RC slab), and pier-to-deck connections (monolithic bearings and combination). These authors identified 11 representative bridge groups and generated fragility curves using an analytical methodology. Avsar et al. [7] classified modern highway bridges in Turkey on the basis of the number of spans (single versus multiple), bent type (single versus multiple), and skew angle (negligible versus significant, chosen to be >30°). Banerjee and Shinozuka [8] classified bridge classes in California, depending on the span type (single or multiple), skew angle (0–20°, 20–60°, and >60°), and soil type. Ramanathan [4] classified California bridge classes based on the superstructure type, number of columns, design era, and abutment configurations. In all the

* Corresponding author.
E-mail addresses: sujithmss@gatech.edu (S. Mangalathu), farahnazsoleimani@gmail.com (F. Soleimani), jsjeon@anu.ac.kr (J.-S. Jeon).

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cited works, bridges were grouped by engineering judgment (traditional grouping), and there is no consensus on the attributes that dictate their seismic performance. As noted by Mangalathu et al. [5], it is not clear whether such a subjective identification can identify all the significant bridge attributes. Also, the traditional grouping is not warranted to yield a reliable grouping and might lead to a non-realistic estimation of seismic risk assessment. The limitations of the traditional subjective grouping approach have motivated the development of performance-based grouping approaches.

Mangalathu et al. [5] suggested a performance-based grouping approach using a statistical technique called Analysis of Covariance (ANCOVA). This technique compares the probabilistic seismic demand model (PSDMs) of different bridge classes. A PSDM is defined as the probability distribution of structural demands (*D*) conditioned on the ground motion intensity measure (*IM*). However, their work is limited to two-span box-girder bridges and did not consider the effect of design era and foundation type. Mehr and Zaghi [9] used an analysis of variance (ANOVA) to compare the response of single-frame and multi-frame bridges and grouped them accordingly. The above cited works on the performance-based grouping [5,9] are limited in their scope to some specific bridge classes and did not account for many design attributes. Thus, a gap currently exists between literature and current practice to have a performance-based grouping methodology that can group the bridge classes including various design attributes such as the cross-section, number of spans, number of frames, span continuity, design philosophy (code requirements), and pier type.

The objectives of this research are: (1) to critically review the HAZUS bridge classification and the fragility relationships based on recent studies and damage data collected in recent earthquakes, (2) to identify whether it is rational to go beyond the HAZUS grouping and fragility relationships, (3) to suggest a performance-based grouping strategy (instead of traditional subjective grouping) to group bridge classes with statistically similar performance and damage measure, (4) to account for the effect of design eras, cross-sections, number of spans, number of frames, abutment types, span continuity, and pier types in grouping bridge classes, and (5) to group the concrete bridge inventory in California. Such a study leads to a realistic estimation of the seismic risk and loss assessment in California. The scope of the study is limited to reinforced concrete box-girder bridges as they are the most common bridge type in California [4].

This paper groups the bridge classes based on a statistical technique called ANOVA [10,11] to determine whether there are any significant differences between the means of seismic demands of two or more (independent) bridge groups. The seismic demands are estimated through a set of non-linear time history analyses (NLTHAs) of bridge models in OpenSees [12]. A suite of 30 ground motions with different soil classes and magnitudes is selected, which can capture the seismic risk in California. Various demand parameters such as column curvature ductility, abutment active, passive, and transverse displacements are considered in this research. Fischer least significant difference method for multiple comparisons is employed after performing ANOVA to identify the bridge classes with similar performances. The insights from this sensitivity study coupled with recent studies on the seismic response of bridges are used to improve the HAZUS grouping of concrete bridge classes. The significance of the grouping is demonstrated in this research by developing fragility curves of two bridge classes grouped by the number of spans.

2. Need to improve HAZUS

HAZUS, by far, is the most comprehensive document in grouping bridge classes and estimating their seismic vulnerability. HAZUS grouped bridge classes with similar damage/loss characteristics and suggested fragility relationships to the grouped bridge classes. This section summarizes the HAZUS grouping and fragility relations and discusses their advantages and disadvantages. Fig. 1 shows the HAZUS-based grouping for the California bridge inventory, and HAZUS defines four damage states based on the extent of damage to the bridge structures during a seismic event: slight, moderate, extensive, and complete, as provided in Table 1. Table 2 presents the grouping and fragility relationships for concrete bridges suggested by HAZUS. Ramanathan (2012) identified the optimal intensity measure for the class of California concrete box-girder bridges as the spectral acceleration at 1.0 s ($S_{a\text{-}1.0s}$, in g). Following this work, this research will use $S_{a\text{-}1.0s}$ as the ground motion intensity measure to estimate bridge fragility characteristics.

The salient features noted from the critical review of HAZUS grouping and fragility relationships for bridges in California are noted below:

- HAZUS classifies bridges into two design eras: pre-1975 and post-1975. However, bridge design philosophies in California is significantly influenced by the 1971 San Fernando and the 1989 Loma Prieta earthquakes. The extensive damage in the 1989 Loma Prieta earthquake forced the California Department of Transportation (Caltrans) to solicit the Applied Technology Council (ATC) to conduct a detailed study and to provide recommendations for design standards, performance criteria, and practices. The recommendations from ATC-32 [13] were incorporated in Caltrans design manuals [14]. Ramanathan [4] showed that the fragility curves are highly influenced by these design philosophies; the seismic vulnerability decreases with the evolution in column design philosophy. Therefore, it is necessary to separate the post-1975 bridge class based on the evolution in seismic design philosophy.
- HAZUS classifies the bridge classes that are not addressed in the main classification as the *other bridge group*. The *other bridge group* represents the high-risk bridge inventory. This classification leads to a situation where multi-frame bridges that are not explicitly addressed in the main group are in the non-classified group, although the seismic vulnerability of multi-frame bridges is much lower than continuous box-girder bridges [9]. Therefore, HAZUS classifications significantly overestimate the seismic vulnerability and loss assessment of multi-frame bridges.
- Although HAZUS classifies the bridges based on the abutment type (monolithic versus non-monolithic, which is inferred as diaphragm versus seat abutments per the recent seismic notions), HAZUS does not suggest explicit fragility relationships based on the abutment type. Ramanathan [4] noted that (1) the demand models as well as fragilities for various bridge components and bridge system are drastically different for bridges with diaphragm and seat abutments and (2) the diaphragm abutments are less vulnerable than the seat abutments in pre-1990 bridges and the trend is reversed in post-1990 bridges.
- HAZUS fragility relationships were developed using a limited number of parameters and simplified two dimensional analyses and did not account for the uncertainties in geometric and material attributes for bridge classes such as the number of spans, span length, deck width, and column height. Also, other researchers [1,4,15] have criticized the use of capacity spectrum method for the generation of fragility curves in HAZUS. The capacity spectrum method estimates the capacity of bridge in the form of a pushover curve and the demand in the form of a response spectrum. The inability to account the higher-mode contributions and vulnerability of other components leads to a non-reliable estimation of fragility curves.

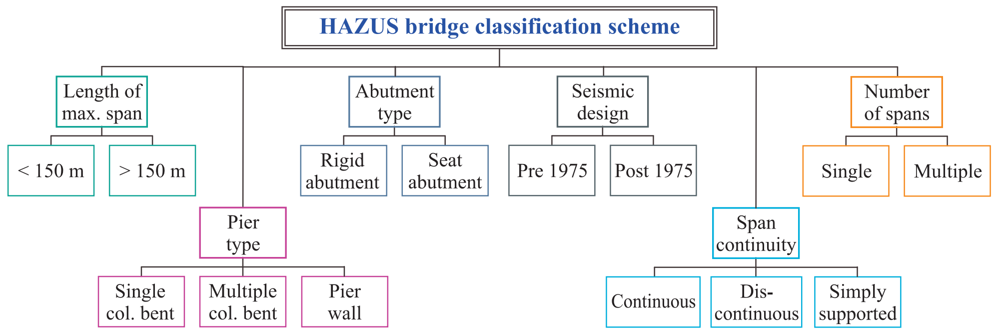

Fig. 1. HAZUS grouping of California box-girder bridge inventory.

Table 1
HAZUS definition of limit states.

Damage states	Definition of damage states
Slight	Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair) or minor cracking to the deck
Moderate	Column experiencing moderate (shear cracks) cracking and spalling (column structurally still sound), moderate movement of the abutment (<50 mm), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure or moderate settlement of the approach
Extensive	Column degrading without collapse – shear failure - (column structurally unsafe), significant residual movement at connections, or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments
Complete	Column collapsing and connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure

- HAZUS considers the vulnerability of bridges to be governed by columns alone. As pointed out by Ramanathan [4], columns are not always the critical components and neglecting the damage to bearings, abutments, and shear keys underestimates the bridge vulnerability.
- HAZUS suggests the same fragility relationships for bridge classes HWB10 and HWB22, and for HWB11 and HWB23. It can be inferred from the same fragility relationships that the type of superstructure (reinforced versus prestressed concrete) is not a significant parameter for the bridge fragilities. This is consistent with similar conclusions noted in recent studies [4,16]. However, slab bridges, T-girder, and box-girder bridges are classified in the same group, but Ramanathan [4] showed that these bridge classes do not have similar fragility curves.
- A comparison of fragility relationships of the bridge classes HWB8 and HWB10, HWB9 and HWB11, HWB18 and HWB20, and HWB19 and HWB21 shows that single column bents (SCBs) are more vulnerable than multi-column bents (MCBs). The previous studies [4,5] showed that SCBs are less vulnerable than MCBs for two- and three-span box-girder bridges.
- While comparing the fragility relationships of the bridge classes HWB22 and HWB23, and HWB10 and HWB11 for the moderate, extensive and complete damage states, the effect of design eras does not have an influence on the fragility relations. Such a conclusion contradicts the research explicitly focusing on the effect of design eras [4,17].
- HAZUS suggests the same fragility relationships for single-span bridges, irrespective of the design eras (HWB3 and HWB4). Although it might hold for diaphragm abutment bridges, it is clearly not the case for seat abutment bridges as there is an increase in the seat-width provision for newer era bridges. Since span-unseating or bearing are the critical components for single-span seat abutment bridges, new era single-span seat abutment bridges are less vulnerable than their counterparts from previous eras as a result of the increased seat-width.
- HAZUS fragility relationships suggested that simply supported bridges are more vulnerable than continuous bridges. Ranf et al. [1] utilized the damage data from the Nisqually Earthquake in 2011 to reveal that it is not true for lower damage

Table 2
HAZUS grouping and fragility relationships for bridge classes in California.

Class	Year built	Description (acronym in HAZUS)	Median value of fragility curve in terms of $S_{a\text{-}1.0\ s}$ (g)				
			Slight	Moderate	Extensive	Complete	Dispersion
HWB1	<1975	Major bridge – Length > 150 m	0.40	0.50	0.70	0.90	0.6
HWB2	≥1975	Major bridge – Length > 150 m	0.60	0.90	1.10	1.70	0.6
HWB3	<1975	Single span	0.80	1.00	1.20	1.70	0.6
HWB4	≥1975	Single span	0.80	1.00	1.20	1.70	0.6
HWB6	<1975	Multi-column bent, Simple support, Concrete	0.30	0.50	0.60	0.90	0.6
HWB7	≥1975	Multi-column bent, Simple support, Concrete	0.50	0.80	1.10	1.70	0.6
HWB8	<1975	Single column, Box-girder, Continuous concrete	0.35	0.45	0.55	0.80	0.6
HWB9	≥1975	Single column, Box-girder, Continuous concrete	0.60	0.90	1.30	1.60	0.6
HWB10	<1975	Continuous concrete (not HWB8/HWB9)	0.60	0.90	1.10	1.50	0.6
HWB11	≥1975	Continuous concrete (not HWB8/HWB9)	0.90	0.90	1.10	1.50	0.6
HWB18	<1975	Multi-column bent, Simple support, Prestressed concrete	0.30	0.50	0.60	0.90	0.6
HWB19	≥1975	Multi-column bent, Simple support, Prestressed concrete	0.50	0.80	1.10	1.70	0.6
HWB20	<1975	Single-column, Box-girder, Prestressed concrete continuous	0.35	0.45	0.55	0.80	0.6
HWB21	≥1975	Single-column, Box-girder, Prestressed concrete continuous	0.60	0.90	1.30	1.60	0.6
HWB22	<1975	Continuous concrete (not HWB20/HWB21)	0.60	0.90	1.10	1.50	0.6
HWB23	≥1975	Continuous concrete (not HWB20/HWB21)	0.90	0.90	1.10	1.50	0.6
HWB28	All other bridges that are not classified		0.80	1.00	1.20	1.70	0.6

states. As there are not enough data for higher damage states, it is not certain whether the HAZUS fragility relationships (simply supported bridges are more vulnerable than continuous bridges) hold for higher damage states. This work also indicated that HAZUS fragility relations overestimate the damage for simply supported bridges.

- Although HAZUS classifies the bridges without considering the number of frames, Mehr and Zaghi [9] showed that one frame bridges do not have similar fragility curves to multi-frame bridges, on the basis of three-dimensional (3-D) NLTHA results.
- An extensive plan review of the California bridge inventory revealed various column cross-sections such as rectangular, circular, and oblong. These cross-sections occupy a major portion of concrete bridges in California, and recent studies [5,18] showed that the bridges with circular and rectangular column cross-sections have different seismic demands and fragilities. Additionally, a companion work by the authors [19,25] showed the difference between the performances of bridges with various column cross-sections; bridges with oblong columns are the least vulnerable.
- Although HAZUS classifies the bridges per the length (length > 150 m and length < 150 m), it is not clear whether the length indicates the frame length or the bridge (total) length.
- As pointed out by Ramanathan [4], there is a mismatch between the damage state definitions used in the fragility analysis and overall bridge functionality in HAZUS. Such a discrepancy causes a problem to Departments of Transportation officials in emergency response decisions.
- HAZUS classified the bridges into single-span and multi-span. A supplementary study is needed to identify whether all the bridges with more than two spans can be lumped together into a single group.
- The effect of pier shaft foundation type is not addressed in HAZUS.

Given the key points noted from the critical review above, it is clear that the HAZUS grouping and fragility relationships need significant improvement. Also, it is rational to advance the grouping of bridge classes from a traditional engineering judgment perspective to performance-based perspective. A performance-based grouping strategy is suggested in this research, which groups the bridge classes based on statistically similar performances and will be explained in the following sections.

3. Grouping of bridge classes by ANOVA

ANOVA is a statistical technique that has been used to analyze the differences among group means. It tests the hypothesis ($H_0 : \mu_{D_1} = \mu_{D_2} = \cdots = \mu_{D_k}$) of whether the mean seismic responses of different bridge classes are equal (Fig. 2) and can group the bridge classes accordingly. In ANOVA, the model is assumed as

$$D_{ij} = \mu_{D_i} + \varepsilon_{ij}, \quad 1 \leqslant j \leqslant n, \quad ,1 \leqslant i \leqslant k \tag{1}$$

where n is the sample size, k is the number of bridge classes, D_{ij} is the response of the jth sample in ith bridge class, μ_{D_i} is the treatment mean of ith bridge class, and ε_{ij} quantifies the difference between the observation D_{ij} and μ_{D_i}. The assumptions underlying ANOVA are (1) the responses are mutually independent, (2) response variance is homogenous, and (3) samples are mutually independent. If these assumptions are seriously violated, the conclusions based on this model are erroneous [20]. A detailed explanation of ANOVA can be found in [10,11]. The grouping strategy adopted in this research is given below.

Step 1: Select possible combinations of bridge configurations.
Step 2: Using Latin Hypercube Sampling (LHS) method, select N ground motions from the suite of ground motions assembled for fragility analysis. The ground motions are selected based on the distribution of $S_{a\text{-}1.0\ s}$ of ground motions.
Step 3: Analyze each bridge configuration in OpenSees [12] for the selected N ground motions.
Step 4: Collect outputs (or response values) including curvature ductility, bearing displacement, abutment active/passive/transverse displacement, etc.
Step 5: Conduct ANOVA to evaluate the sensitivity of each component to the variation in bridge configurations. The results can be more easily inferred in terms of p–value. The p–value is the evidence against a null hypothesis or the probability that the variation between groups occurs by chance. The p–value can be interpreted as the probability of such an *extreme* value of the test statistic when H_0 is true.
Step 6: Perform Fischer Method on ANOVA outputs to group the bridge configurations which have statistically similar responses. Fisher method compares all pairs of groups while controlling the individual error rate. It identifies the highest sensitive group and checks a null hypothesis whether the mean values of other groups match with the highest sensitive one. If it does, they will be grouped together. If it does not, it will check the second highest sensitive group and check whether the mean value of the remaining groups matches with the second highest sensitive group. The procedure is repeated until all the members are grouped.

4. Bridge configurations, numerical modeling procedure, and ground motion selection

This research demonstrates the bridge grouping with box-girder bridges as they occupy the major bridge portion in California [4]. Consistent with the work of Ramanathan [4], bridges are classified into (1) Pre-1971 design era (era 11, hereafter), (2) 1971–1990 design era (era 22, hereafter), and (3) Post-1990 design era (era 33, hereafter) based on the evolutions in the seismic design philosophy. The general layout of a two-span box-girder bridge is illustrated in Fig. 3. Column bents can be either single column or multi-column bent. The possible number of columns for a multi-

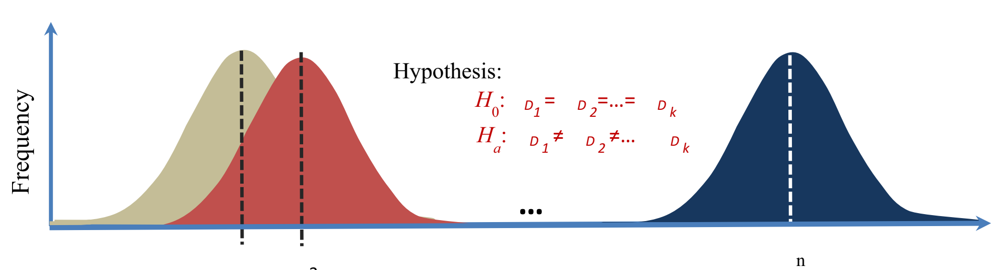

Fig. 2. ANOVA hypothesis.

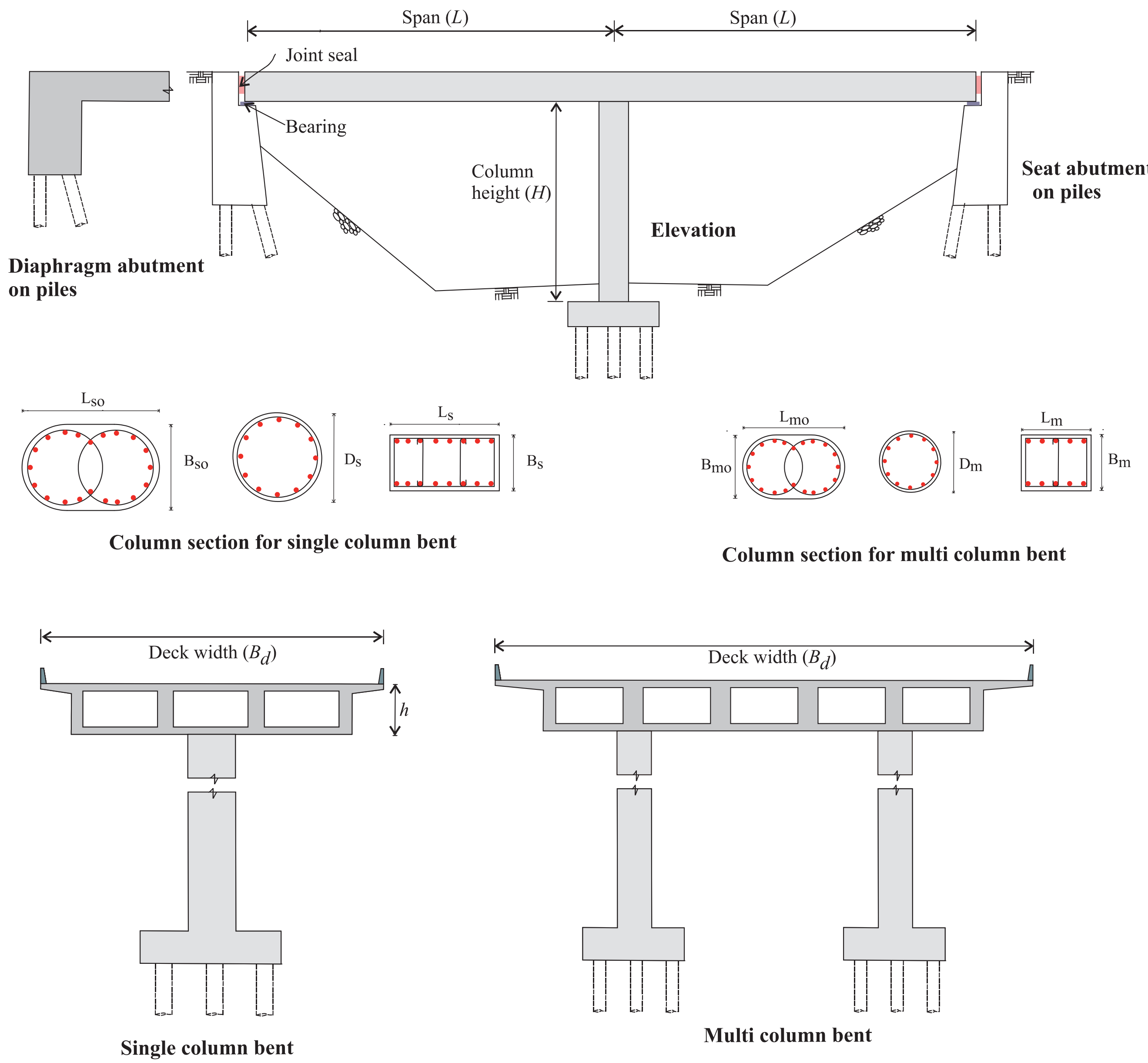

Fig. 3. General layout of a two-span concrete box-girder bridge and possible configurations of column cross sections.

column bent bridge is as follows: two- and three-column for era 11, two-, three-, four-, and five-column for era 22, and two-, three-, four-, and five-column for era 33 [4]. The abutments can be either of seat or diaphragm type. The fundamental difference between the diaphragm and seat abutments is that the latter allows superstructure movement independent of the abutment while the former does not. Various cross-sections such as rectangular, circular, or oblong shape are also adopted in this research.

4.1. Numerical modeling procedure

Although a more detailed description of the analytical modeling can be found in [5], the general approach is briefly presented herein. Fig. 4 shows a 3-D numerical model of box-girder bridges including the response of various bridge components, which is created in OpenSees [12]. The superstructure is modeled using elastic beam column elements, and transverse deck elements are modeled as rigid elements. Columns are modeled using fiber-type displacement-based beam column elements, foundations are modeled using linear and translational springs, poundings are modeled using contact elements [21], and shear keys are modeled based on the experimental work of Silva et al. [22]. Abutment responses comprise earth pressure response (passive resistance of the backfill) and structural response (pile resistance or abutment action on spread footing). The passive response of the abutment backwall is simulated using the hyperbolic soil model of Shamsabadi et al. [23], while the response of the piles is simulated using a trilinear material model of Mangalathu et al. [5].

4.2. Material and geometric uncertainties

A number of sources of uncertainties (aleatoric or epistemic) for the selected class of bridges are considered in this research and are provided in Table 3. This table presents the mean value (μ), standard deviation (σ), and the associated probability distribution of various input variables. These values are derived from an extensive plan review of California bridges and thus mimic the actual California bridge inventory [5].

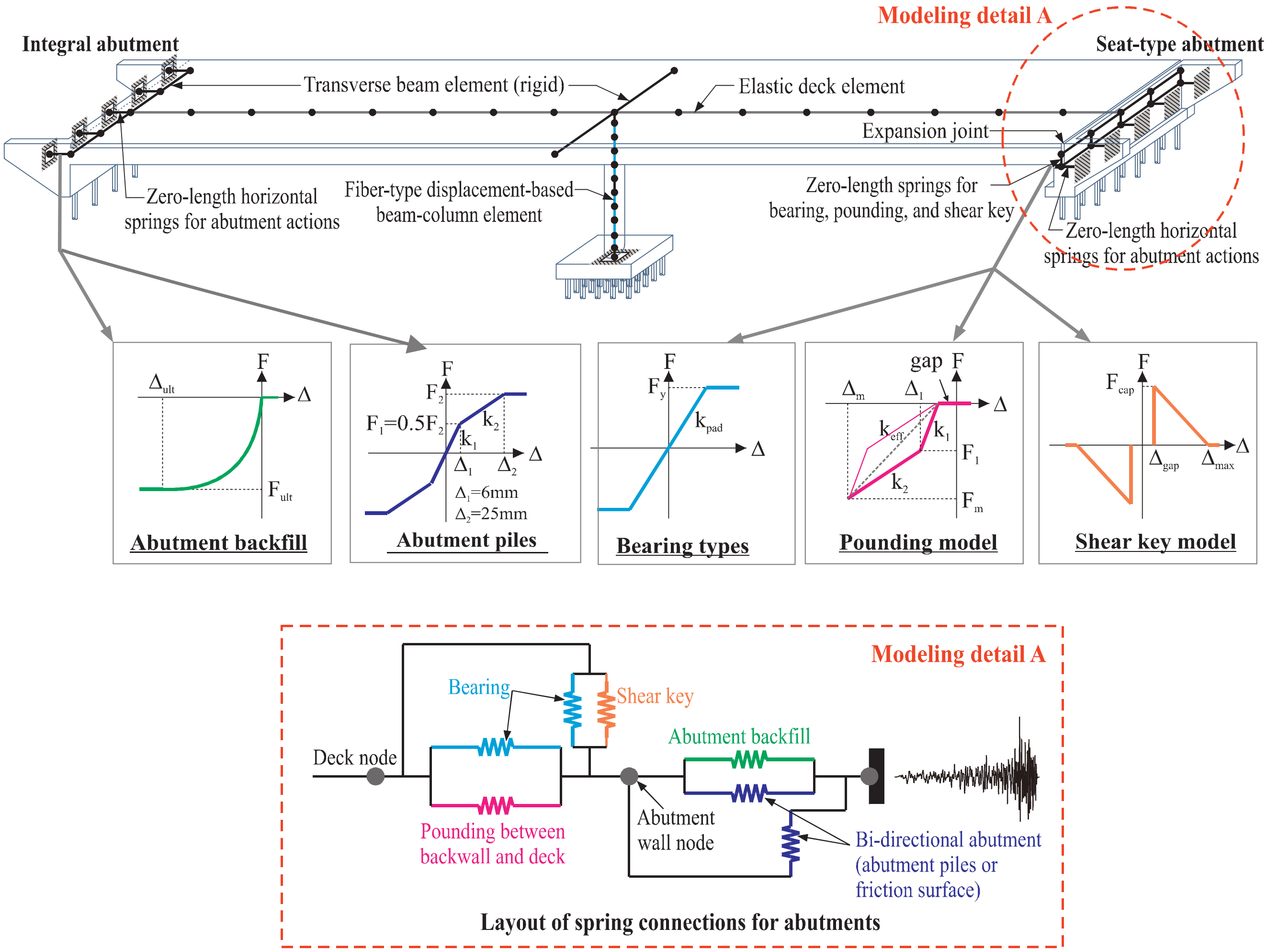

Fig. 4. Numerical modeling of various bridge components.

4.3. Ground motions

This research uses the suite of ground motions assembled by Baker et al. [24] for the fragility assessment of California bridges. It consists of 120 ground motions associated with moderate-to-strong earthquakes at small distances and 40 ground motions with strong velocity pulses characteristics of sites experiencing near-fault directivity effects. The entire suite of ground motions is scaled by a factor of two [4] to have sufficient response data of IMs higher than Palmdale spectrum (the highest probabilistic design hazard level in California A sampling technique called LHS is used in this study to select 30 ground motions for sensitivity analysis, based on the probability distribution of IM ($S_{a\text{-}1.0s}$ in the current study). LHS provides a stratified sampling scheme to cover the probability space of the random variables. Compared to Monte Carlo simulation (MCS), LHS samples ground motions from the entire distribution of IM. The use of LHS ensures the selection of ground motions based on the seismic hazard and probability distribution of IM of the region. It is noted that the inclusion of ground motions more than 30 yields the same grouping as identified by the 30 ground motions selected by LHS. The histogram of peak ground accelerations (PGAs) and the response spectra of the selected 30 ground motion suite are shown in Fig. 5. Note that the use of LHS ensures the selection of ground motions in all the likely possible scenarios. It is seen from Fig. 5 that only three ground motions with PGA higher than 0.8 g are chosen because the ground motions with such a high magnitude are highly unlikely to happen. This 30 ground motions will be used to assess ANOVA-based grouping to reduce computational efforts. However, the current study uses 320 ground motions to achieve fragility curves of grouped bridges, following the work of Ramanathan [4].

4.4. Results of the ANOVA grouping

To group the bridge classes via ANOVA-based grouping, the input parameters are kept at their mean value. In other words, the uncertainties in the input parameters are not considered while grouping the bridge classes. NLTHA is carried out for the bridge models using the selected ground motions, and the maximum response of various bridge components is recorded. The results of the selected sensitivity and grouping study are provided in this section. The various demand parameters and associated capacity models presented in Mangalathu [25] are indicated in Table 4 and considered in this research. The sensitivity of the seismic demands of two-span bridge configurations to the various design eras and bent configurations is evaluated using ANOVA and is presented in Table 5. Note that ANOVA is employed after transforming the demand parameters into a lognormal space to have a better relationship between the IM and demand parameter [26,27]. As mentioned before, the results are inferred in terms of *p*–value. A smaller *p*–value refers to stronger evidence for rejecting the null hypothesis (H_0), and a cut-off *p*–value of 0.05 is adopted in this research [10]. If the *p*–value is less than 0.05, not all of the population means are equal. It is clearly observed from Table 5 that all the

Table 3
Uncertainty distribution considered in the bridge models.

Parameter	Units	Distribution		
		Type	μ	σ
Concrete compressive strength (f_c)	MPa	Normal	29.03	3.59
Reinforcing steel yield strength (f_y)	MPa	Lognormal	465.0	37.30
Span length (L)				
Two-span	mm	Lognormal	31775	8738
Approach to main span ratio (> two-span bridge)	–	Normal	057	0.13
Deck width (B_d)				
Single column bent	mm	Lognormal	9780	1980
Multi-column bent	mm	Lognormal	11970	2418
Column height (H)	mm	Lognormal	6625	865
Abutment backwall height (H_a)				
Diaphragm abutments	mm	Lognormal	3234	488
Seat-type abutments	mm	Lognormal	2186	441
Abutments on piles - Lateral capacity/deck width (K_{pa})				
Diaphragm abutment	N/mm	Lognormal	1120	404
Seat-type abutment	N/mm	Lognormal	1498	540
Elastomeric bearing pad				
Stiffness per deck width (K_b)	N/mm/m	Lognormal	908	327
Coefficient of friction for bearing pad (μ_b)	–	Normal	0.30	0.10
Gap (g)				
Longitudinal (btw. deck & abutment wall)	mm	Lognormal	23.5	12.5
Transverse (btw. deck and shear key)	mm	Lognormal	12.8	2.58
Mass factor (m)		Uniform	1.25	0.007
Damping (ξ)		Normal	0.045	0.0125
Acceleration for shear key capacity (a_s)	g	Lognormal	1.00	0.20
Longitudinal reinforcement ratio (ρ)				
era 11	(%)	Uniform	1.9	0.08
era 22/era 33	(%)	Uniform	2.35	0.61
Column cross-section				
Single column bent (all eras)				
Circular (D_s)	mm	1676		
Rectangular ($L_s \times B_s$)	mm	1829×1219		
Oblong ($L_{so} \times B_{so}$)	mm	2438×914		
Multi column bent (all eras)				
Circular	mm	1219		
Rectangular ($L_m \times B_m$)	mm	1219×914		
Oblong ($L_{mo} \times B_{mo}$)	mm	1219×914		
Transverse reinforcement				
era 11	13 mm diameter rebar @ 300 mm c.t.c			
era 22	(%)	Uniform	0.6	0.03
era 33	(%)	Uniform	1.05	0.14
Pile group – pile cap and piles				
Translational stiffness (K_{ft})				
Solumn – 1% long. rebar	N/mm	Normal	297716	140101
Column – 3% long. rebar	N/mm	Normal	245178	105076
Rotational stiffness (K_{fr})				
Column – 1% long. rebar	N-m/rad	Normal	4.5×10^9	1.1×10^9
Column – 3% long. rebar	N-m/rad	Normal	6.8×10^9	1.1×10^9

demand parameters in the diaphragm abutment bridges are highly sensitive to the design eras and bent configurations, and thus cannot be grouped together. In the case of the seat abutment bridges, the column curvature ductility and bearing displacement are the most sensitive demand parameters affected by the design eras and bent configurations. Fischer method is carried out on ANOVA to group the bridges with similar performances (seismic demands), and its results are indicated in Table 6. In the table, A, B, C, and D indicate different bridge classes. Additionally, Table 7 presents the Fischer method grouping results of era 11 bridges with seat-type abutments with respect to the number of spans, and Table 8 presents the grouping results of era 11 two-span bridges with respect to the bearing type. It is seen from Table 8 that the type of bearings (elastomeric or rocker) have a significant influence on the bridge responses and hence cannot be grouped together Following inferences obtained from the sensitivity study results in Tables 6–8 are summarized below:

(1) For the two-span seat and diaphragm abutment bridges, era 11 shows a distinct behavior from other design eras (Table 6). It can be inferred that the change of the seismic design philosophy from era 22 to era 33 does not significantly affect the seismic demand of bridge components.
(2) From Table 6, the seismic demand of columns (μ_ϕ) is the component greatly influenced by the design eras and the number of columns per bent in the case of two-span bridge configurations. It requires special attention because the column performance governs the bridge vulnerability in most bridge configurations [5,28].

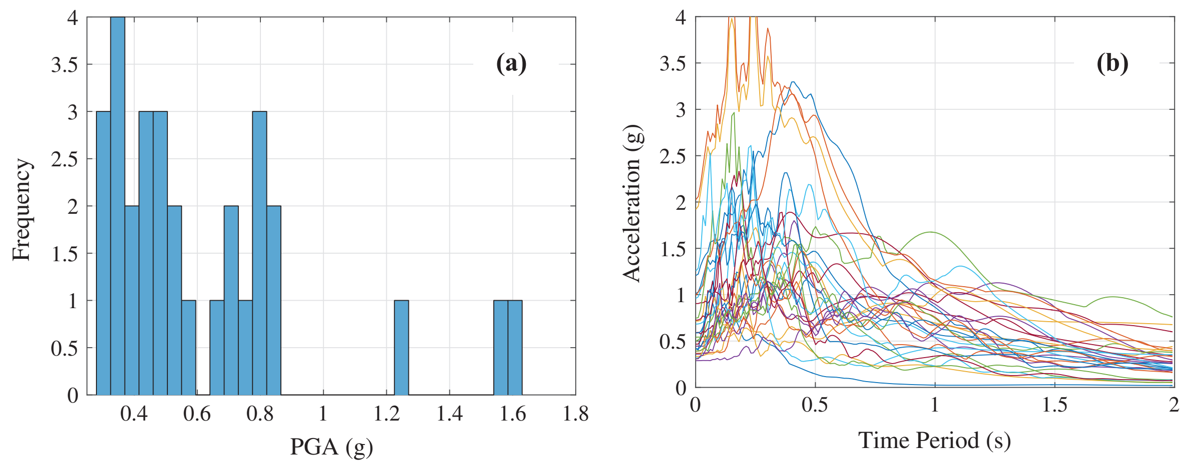

Fig. 5. (a) Histogram of the PGA values of the ground motion suite and (b) acceleration response spectrum of the ground motion suite.

Table 4
Bridge component demand parameters and capacity models for era 11 bridges [25].

Demand parameter	Median value, S_c				Dispersion (β_c)
	Slight	Moderate	Extensive	Complete	
Column curvature ductility (μ_ϕ)	0.80	2.0	5.00	8.00	0.35
Passive abutment displacement (δ_p, mm)	76	254	–	–	0.35
Active abutment displacement (δ_a, mm)	38	102	–	–	0.35
Transverse abutment displacement (δ_t, mm)	25	102	–	–	0.35
Bearing displacement ($\delta_{b,}$ mm)	25	76	–	–	0.35
Superstructure unseating (δ_u mm)	–	–	152	229	0.35

Table 5
p–value from ANOVA for bridges with diaphragm and seat abutments.

Bridge type	p–value for demand parameters				
	μ_ϕ	δ_p	δ_a	δ_t	δ_b
With diaphragm abutment	**0.000**[*]	**0.001**	**0.002**	**0.001**	–
With seat abutments	**0.037**	0.857	0.888	0.503	**0.050**

[*] Values highlighted in bold are significant parameters.

(3) By comparing seismic demands on the abutments (Table 6), the bridge design philosophy and the number of columns per bent have more influences on the diaphragm abutment bridges than the seat abutment bridges. It might be due to the integral connection of the diaphragm abutment bridges at the ends which causes the abutment to share a significant portion of the seismic demand. In the case of the seat abutment bridges, the seismic demand on the abutments is less influenced by the design eras and the number of columns per bent.

(4) The seismic demands of bridges with single-column bent, two-column bent, and multi-column bent (bent with more than two columns, hereafter) are statistically different for the two-span bridge configurations (Table 6), and thus cannot be grouped together from a seismic demand perspective.

(5) In the case of the diaphragm abutment bridges, two-span bridges have distinct seismic performances for all the bridge components from three- to six-span bridge configurations. Although not shown here, similar conclusions are also observed for the seat abutment bridges.

(6) The diaphragm and seat abutment bridges have different seismic demands for all the design eras and bent configurations.

(7) For the seat abutments, the type of bearings (rocker or elastomeric) significantly affects the seismic demand of bridges (Table 8).

The results of the sensitivity study on specific bridge configurations (for example, the effect of number of spans and cross-section on era 11 seat abutment bridges) are shown here. Similar observations are also noted in other design era bridges.

5. Grouping of bridge classes

Fig. 6 shows the proposed grouping, and the classification is carried out based on the abutment type, column cross–section, pier type, number of spans, span continuity, and seismic design to modify the HAZUS grouping of bridge classes for their seismic vulnerability assessment. These attributes are selected based on the current sensitivity study, and insights from previous research on the seismic response of bridges [4–9,16–18,28–38].

Abutment type: The responses of the diaphragm abutment bridges are different from the seat abutment bridges, and thus cannot be grouped together. Two types of bearings are noted for the seat abutment bridges: rocker bearings versus elastomeric bearings. Different response mechanism is associated with these bearings; the governing motion associated with elastomeric bearings is based on sliding, while it is characterized by rocking in case of rocker bearings. Thus, both bearing types yield different seismic responses and associated failures.

Table 6
Results of the grouping for two-span box-girder bridges.

Bridge configurations		Column		Abutment passive		Abutment active		Abutment transverse		Bearing/ Unseating	
		Mean[a]	Group[b]	Mean	Group	Mean	Group	Mean	Group	Mean	Group
With diaphragm abutments	era 11 – 1 column bent	0.86	B C	0.00	B	0.08	B	0.75	C	–	– –
	era 11 – 2 column bent	1.96	A	0.39	A	0.45	A	1.58	A	–	– –
	era 22 – 1 column bent	0.10	D	0.00	B	0.08	B	0.67	C	–	– –
	era 22 – 2 column bent	1.13	B	0.45	A	0.51	A	1.17	B	–	– –
	era 22 – 3 column bent	0.90	B C	0.42	A	0.48	A	1.00	B C	–	– –
	era 22 – 4 column bent	0.73	B C	0.48	A	0.53	A	0.90	B C	–	– –
	era 33 – 1 column bent	0.10	D	0.00	B	0.09	B	0.67	C	–	– –
	era 33 – 2 column bent	1.13	B	0.45	A	0.51	A	1.17	B	–	– –
	era 33 – 3 column bent	0.91	B C	0.42	A	0.48	A	1.01	B C	–	– –
	era 33 – 4 column bent	0.73	B C	0.48	A	0.53	A	0.90	B C	–	– –
	era 33 – 5 column bent	0.65	C	0.49	A	0.54	A	0.83	B C	–	– –
With seat abutments	era 11 – 1 column bent	1.26	B	−0.02	A	0.01	A	−0.05	A	0.94	B
	era 11 – 2 column bent	1.99	A	0.32	A	0.35	A	0.65	A	1.60	A
	era 22 – 1 column bent	0.51	C	0.07	A	0.10	A	−0.02	A	0.93	B
	era 22 – 2 column bent	1.26	B	0.36	A	0.39	A	0.42	A	1.22	A B
	era 22 – 3 column bent	1.03	B C	0.31	A	0.33	A	0.33	A	1.09	B
	era 22 – 4 column bent	0.92	B C	0.25	A	0.28	A	0.23	A	1.04	B
	era 33 – 1 column bent	0.51	C	0.07	A	0.10	A	−0.03	A	0.93	B
	era 33 – 2 column bent	1.26	B	0.36	A	0.39	A	0.42	A	1.22	A B
	era 33 – 3 column bent	1.04	B C	0.31	A	0.33	A	0.33	A	1.09	B
	era 33 – 4 column bent	0.92	B C	0.25	A	0.27	A	0.23	A	1.04	B
	era 33 – 5 column bent	0.76	B C	0.23	A	0.24	A	0.06	A	0.99	B

[a] Mean is shown in a logarithmic scale.
[b] Bridge configurations that do not share common alphabet cannot be grouped together as seismic demands of these bridges are statistically different.

Table 7
Results of the grouping for multi-span bridges.

Bridge configurations	Column		Abutment passive		Abutment active		Abutment transverse		Bearing/Unseating	
	Mean[a]	Group[b]	Mean	Group	Mean	Group	Mean	Group	Mean	Group
era 11 – 2span	1.26	A	−0.02	A	0.01	A	−0.05	A	0.93	A
era 11 – 3span	0.82	A B	−0.49	A B	−0.47	A B	−0.70	A B	0.61	A B
era 11 – 4span	0.55	B	−0.88	B	−0.87	B	−0.85	B	0.40	B
era 11 – 5span	0.53	B	−0.99	B	−0.97	B	−0.97	B	0.36	B
era 11 – 6span	0.52	B	−1.04	B	−1.02	B	−1.03	B	0.36	B

[a] Mean is shown in a logarithmic scale.
[b] Bridge configurations that do not share common alphabet cannot be grouped together as seismic demands of these bridges are statistically different.

Table 8
Results of the grouping for different types of bearings for two-span bridges.

Bridge configurations	Column		Abutment passive		Abutment active		Abutment transverse		Bearing/Unseating	
	Mean[a]	Group[b]	Mean	Group	Mean	Group	Mean	Group	Mean	Group
era 11 – Elastomeric	1.26	A	−0.02	A	−0.01	A	−0.05	A	0.93	A
era 11 – Rocker	0.69	B	−0.57	A	−0.55	A	−0.87	B	0.04	B

[a] Mean is shown in a logarithmic scale.
[b] Bridge configurations that do not share common alphabet cannot be grouped together as seismic demands of these bridges are statistically different.

Column cross-section: On the basis of the column cross-section shape, the bridges are classified into bridges with circular, rectangular, and oblong cross-sections. Interested readers are directed to Mangalathu [25] for a detailed comparison on the fragility curves of bridge columns with different cross-sections. The author indicated that bridges with oblong columns are less vulnerable than circular and rectangular column bridges. The relative vulnerability of the circular and rectangular cross-sectioned bridge columns depends on the number of columns per bent.

Pier type: The sensitivity results show that the number of columns in a multi-column support does not significantly affect the bridge response, and thus does need to be considered as separate classes. The responses for bridge models with three- to five-column bents are statistically similar and can be grouped together. The responses for bridges with two-column bents and single column bents are shown to be distinct, and thus cannot be grouped with other support systems. However, two types of column-footing connection are possible for multi-column bents: pinned at the base (not restraint against rotation) or fixed at the base. Both cases should be treated differently. The responses of pier-wall and pier shafts [6,34,38] are distinct from the column behavior, and thus should be considered separately.

Span range: Single-span bridges (without columns) need to be treated as a separate class due to their unique (and limited) combination of demand parameters. The sensitivity studies consider single frame systems having more spans (from two- to six-spans) to determine if any range could be grouped. Two-span bridges seem to have a distinct seismic performance

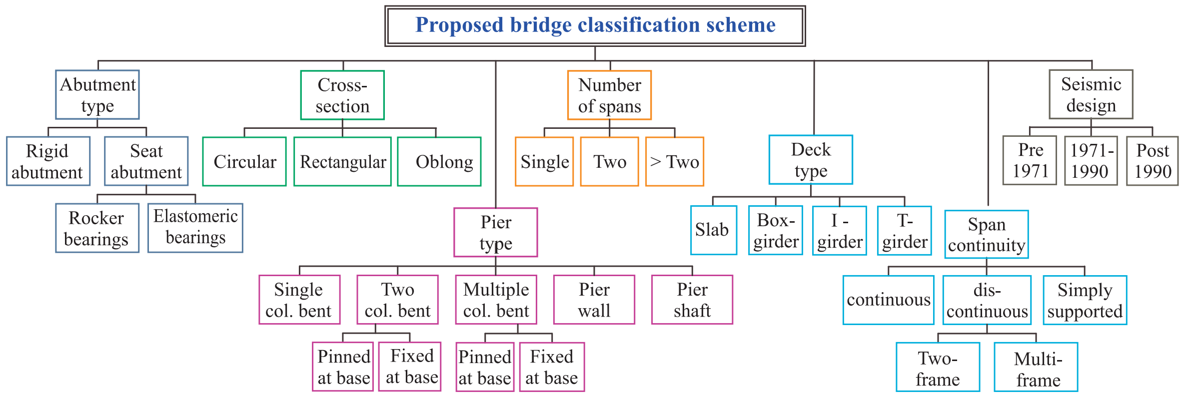

Fig. 6. Proposed grouping of bridge classes.

from other multi-span bridges, and thus cannot be grouped with other bridge classes. From the plan review of bridges, only very few bridges are noted with more than six spans in a single frame. Thus, three groups are suggested based on the span: (1) one-span, (2) two-span, and (3) multi-span (more than two spans).

Deck type: The bridges are classified into slab, box-girder, I-girder and T-girder bridges based on the insights from the sensitivity study. Also, Ramanathan [4] revealed that the PSDMs and fragilities associated with these bridges are different.

Span continuity: Based on the span continuity, bridges are classified into simply supported, continuous and discontinuous (or bridges having more than one frame). Responses for two frame systems are clearly unique, but the distinctions between higher numbers of frames are less clear. The two frame system is thus retained as a distinct class. A larger number of frames such as three frames, four frames, etc. are combined into the multi (3+)-frame. Note that a few numbers of multi-frames are noted from the extensive plan review, and such a compromise seems to be reasonable.

Design code era: Sensitivity results show that bridges built/rebuilt within either the two later design code eras (era 22 and era 33) hold statistically similar seismic demands and could be grouped for the purposes of establishing demand models. However, the design philosophy considerably improves the capacity models, and thus these era bridge models yield distinct fragility curves. Thus, era 22 and era 33 bridges can be grouped from a demand perspective, but not from a fragility perspective. Era 11 bridges are shown to have distinct responses (also capacities) and require the development of separate demand models.

To evaluate the necessity and the significance of the proposed grouping, fragility analysis is carried out for the selected case study bridges. As an illustration, fragility curves for only one bridge class will be developed in the next section, because the fragility analysis for the entire bridge classes is beyond the scope of the current study.

6. Fragility curves

Fragility curves are generated for some selected cases to demonstrate the significance of proposed grouping strategy and the need to go beyond HAZUS grouping and fragility curves. To generate the fragility curves, statistically significant yet nominally identical 3-D bridge models are generated by sampling across the range of the parameters presented in Table 2 using Latin Hypercube Sampling technique and are randomly paired with the selected suite of ground motions. The two orthogonal components of the ground motions are randomly assigned to the longitudinal and transverse direction of the bridge axis. A set of NLTHAs (320 simulations) is performed for all bridge model-ground motion pairs to monitor the seismic demand of various bridge components and to develop their fragility curves. Assuming that both demands and capacities follow a lognormal distribution, the fragility function for a bridge component can be practically defined as a lognormal cumulative density function and expressed as [26]:

$$P[D > C|IM] = \Phi\left[\frac{\ln(S_d/S_c)}{\sqrt{\beta_{d|IM}^2 + \beta_c^2}}\right] \tag{2}$$

where S_d and $\beta_{d|IM}$ are the median and dispersion of the demand conditioned on IM. $\Phi[\bullet]$ is the standard normal cumulative distribution function. S_c and β_c are the median and dispersion of the capacity or limit states. The limit states were derived in such a way that it aligns with the Caltrans design and operational experience [4]. Interested readers are directed to the work of Mangalathu [25] for a more detailed description on the determination of limit states with different design eras. The component fragilities are integrated into system fragilities through the development of joint probabilistic seismic demand models (JPSDMs) and a series system assumption [4,31]. Thus, similar to the component fragilities, the system fragility curves are characterized by a lognormal cumulative distribution function with median (λ) and dispersion (ζ). A detailed description of the fragility methodology can be found in the Refs. [4,25].

To demonstrate the significance of the proposed grouping, the bridge classes are formed based on the number of spans: (1) two spans (hereafter, 2span), (2) three spans (hereafter, 3span) and (3) four spans (hereafter, 4span). To achieve this goal, the bridge type with era 11, seat type abutments, two-column bents, circular column cross-section is selected for this research. HAZUS classifies these bridge groups into a single class ("multiple" in Fig. 1, HWB22 in Table 2), while the proposed grouping classifies these groups into two groups: two-span and multi-span (more than two spans). System fragility curves for the selected bridge groups are developed, and the fragility characteristics are presented in Table 9. The following inferences are noted from the comparison of bridge class fragilities in this table:

(1) 2span bridges are more vulnerable than 3span and 4span bridges in that the median value of fragility curve for 3span and 4span bridges is 10–15% higher than that for 2span bridges. This observation indicates that the HAZUS grouping (to group the bridges with more than two spans into a single group) needs modification.

Table 9
System fragility characteristics for the selected bridge classes.

Bridge type	Sub-classes	Slight		Moderate		Extensive		Complete	
		λ (g)	ζ	λ (g)	ζ	λ (g)	ζ	λ (g)	ζ
Era 11, 2-Col, seat abutment bridge	HAZUS	0.600	0.600	0.900	0.600	1.100	0.600	1.500	0.600
	2span	0.078	0.460	0.170	0.456	0.355	0.458	0.514	0.448
	3span	0.089	0.524	0.195	0.530	0.393	0.516	0.575	0.526
	4span	0.088	0.512	0.192	0.519	0.390	0.513	0.558	0.517

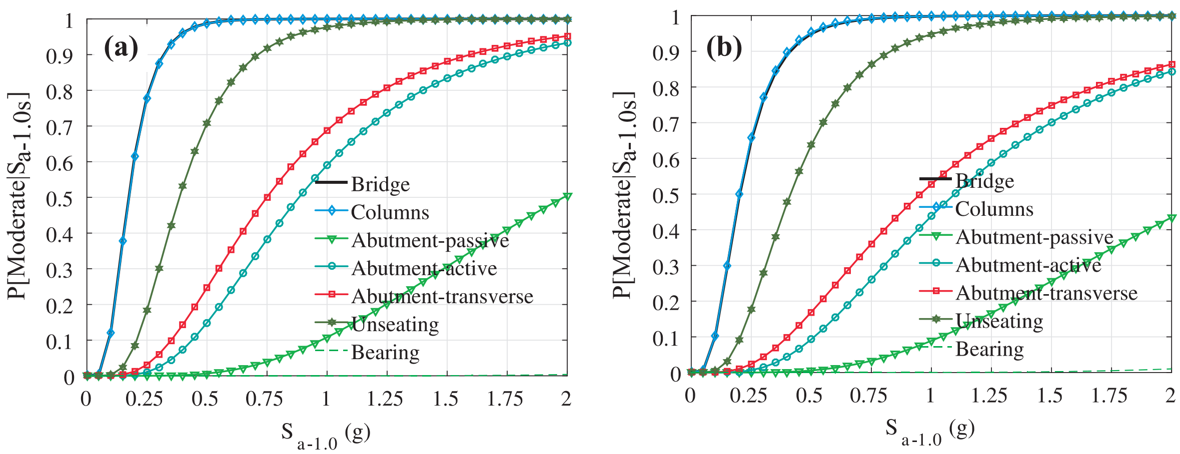

Fig. 7. System and component fragilities for selected bridge classes for moderate damage state: (a) 2span and (b) 3span.

(2) 3span and 4span bridge fragilities are statistically similar, and the difference in the median fragilities between the 3span and 4span bridge classes is less than 3% for the four limit states. It substantiates the proposed grouping methodology to group bridges more than 2 span into a different class.
(3) The bridge classes seem to be more vulnerable than that predicted by HAZUS. It can be attributed to the fact that (1) HAZUS generated fragility curves based on 2-D analyses without accounting for the geometric and material uncertainties, (2) the vulnerability of bridges in HAZUS is dominated by only columns, and (3) HAZUS used response spectrum analysis and capacity curves while this research captures bridge responses based on 3-D NLTHA.
(4) A single value of dispersion equal to 0.6 is suggested by HAZUS for all the bridge classes. The dispersion noted in this research is less than that suggested by HAZUS and seems to vary depending on the bridge class and limit states.

Additionally, Fig. 7 shows the system and component fragilities for 2span and 3span bridge classes for the moderate damage state. Considerable differences between the system and component fragility curves between 2span and 3span bridge classes are observed from these figures, which underscore the importance to group these bridge classes.

7. Conclusions

Regional seismic risk assessment of a highway transportation network is usually carried out by grouping bridge classes, which are expected to have similar performances during a seismic event. The existing grouping method does not cover the entire bridge inventory and is based mostly on engineering judgment and past experience. Among existing grouping methods, HAZUS grouping is the widely accepted one. However, a critical review of HAZUS based on recent studies reveals many drawbacks and shows that the HAZUS bridge grouping and associated fragilities lead to a non-realistic estimation of the seismic demands and associated losses. A new performance grouping method based on analysis of variance (ANOVA) is suggested in this research.

The grouping method is demonstrated with the case studies of concrete box-girder bridges in California. 30 ground motions are selected by Latin hypercube sampling from the suite of ground motions assembled for California for the nonlinear time history analysis (NLTHA) of bridge models. Consistent with the ground motions, 30 3-D bridge models are created in OpenSees [12], and the maximum component responses are recorded for each NLTHA. ANOVA is carried out for the recorded maximum bridge responses to identify whether the mean responses are statistically similar or not. If not, the bridges are grouped by a pairwise comparison through Fischer method. The results are then extended to modify the HAZUS grouping of bridge classes. The proposed grouping identifies more bridge classes than HAZUS, and the grouping is based on the abutment type, column cross-section, bent type, number of spans, span continuity and seismic design (code requirements for bridges).

To examine the significance of the proposed grouping, fragility analysis is carried out for 320 realizations of a selected case study bridge. The selected bridge type is one constructed before 1970 with seat abutments, circular column cross-section, and two-column bents and is grouped into three bridge classes depending on the number of spans: two, three, and four spans. HAZUS classifies these bridge classes into a single group while the proposed grouping approach classifies these bridge classes into two bridge groups: two-span and multi-span (more than two spans). The comparison of bridge fragilities shows that (1) the two-span bridges are approximately 11–15% more vulnerable than the three- and four-span bridges and (2) the difference in median fragilities between the three- and four-span bridges is negligible. Therefore, the comparison of bridge fragilities justifies the proposed grouping approach.

This research improved bridge classifications of HAZUS by considering various structural attributes. Although this research groups the bridge classes for California box-girder bridges, the proposed grouping approach can also be applied to other regions and bridge types by fine-tuning the grouping based on the evolution in seismic design philosophy and other distinct attributes.

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