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Design of bridges with skewed abutments for a target tolerable seismic loss

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ABSTRACT

A comprehensive risk based bridge design procedure through which the effect of abutment skew angle is considered upfront aiming at uniformity in seismic risk is presented. We demonstrate a method for conceptual design of bridges with skewed abutments aiming at uniform performance (i.e., tolerable loss) objectives. In the proposed technique, few simple design aid graphs are developed by estimating the bridge repair cost ratios (RCRs) at different levels of column drift ratios (CDRs) and abutment skew angles. The suggested graphs parameters are estimated by implementing a full Monte Carlo probabilistic simulation over Pacific Earthquake Engineering Research Center (PEER) Probabilistic Based Seismic Assessment (PBSA). To show the significance of the proposed conceptual design method, a two-span Caltrans *ordinary* bridge class is selected for case study while various design parameters such as column diameter, column height to depth ratio, longitudinal reinforcement ratio, and span length are included as design variables. The effect of abutment skew angle on the probability of exceeding an RCR as well as probability of bridge collapse is explored by considering various bridge design parameters. Limitations of the proposed approach are discussed.

1. Introduction

In recent years, much research has been dedicated to the seismic performance assessment of bridges with skewed abutments (aka *Skewed Bridges*). A number of such efforts have focused on modeling considerations and dynamic response of skewed bridges during seismic excitations [1–7]. In particular, Sullivan and Nielson [7] have investigated the effect of abutment skew angle on the fragility of multi-span simply supported steel girder bridges in Central United States. According to their findings, an abutment skew angle larger than 15° has a considerable effect on the bridge's seismic fragility. Fragility analysis of skewed single frame concrete box girder bridges in California was conducted by Zakeri et al. [9] by considering the effects of abutment type; column bent type, and levels of column seismic detailing. Their findings indicate that old bridge fragilities are sensitive to column properties more than abutment skew angle, while in newer bridges the effect of abutment skew angle on the fragility of bridges–particularly ones with seat-type abutments–is considerable. Kaviani et al. [8] explored performance-based seismic assessment of skewed bridges by proposing a multi-phase probabilistic assessment of structural response to seismic excitations (M-PARS). A large number of shear key failure followed by unseating of the superstructure deck were observed in box girder bridges with high skew angles designed after year 2000. The probability of shear key failure is considerably high for single-column bridges according to their findings.

Seismic response prediction and modeling consideration of Painter over pass-bridge in California–a box-girder bridge with skewed abutments–by Ramanathan et al. [10] clarified the effect of abutment skew angle as well as bridge modeling parameters on seismic response and damage fragility curves. The effect of unequal column height on damage fragility curves of skewed multi-span simply supported I-girder concrete bridges in Central and Southeastern United States (CSUS) was investigated by Zamani Noori et al. [11]. They showed that component damage fragilities become more sensitive to unequal column height ratio for large abutment skew angles particularly in higher damage states.

The effectiveness of 10 different retrofit measures on the fragility of skewed box girder bridges in California was examined by Zakeri et al. [12]. They concluded that the effectiveness of retrofit measures is highly dependent on abutment skew angle. Yang et al. [13] have recently analyzed six types of skewed bridges in CSUS; they looked into columns designed pre-1975 (i.e. non-seismic columns) and post-1994 (i.e. seismic columns). Based on their findings, incorporating seismic design concepts in proportioning columns can effectively reduce bridge vulnerability while increasing shear key capacity through seismic retrofit can transfer more seismic demands to columns and abutments.

Despite extensive efforts to evaluate seismic response of skewed bridges, a comprehensive research to assess seismic risk of such bridges has not been conducted. The research effort summarized herein presents a method to include the effect of abutment skew angle in

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performance-based seismic design of bridges. A full Monte Carlo probabilistic simulation over PEER's PBSA approach is performed by considering the correlation between Engineering Demand Parameters (*EDPs*) at different components. The bridge annual probability of exceeding different levels of Repair Cost Ratio (RCR)–ratio of repair cost to replacement cost–and bridge collapse are explored for various bridge geometric parameters while considering abutment skew angle. The results of this research clarify the effect of abutment skew angle on the annual probability of column failure (demand > capacity) in term of reliability index, β. The proposed design aid graphs can assist the designers by providing a practical tool to proportion bridge columns based on the allowable probability of exceeding tolerable level of loss based on the bridge abutment skew angle.

2. Skewed bridge design framework for a target tolerable seismic loss

2.1. Background

Over the past decade, various methods for seismic risk assessment of structures have been proposed; the loss estimation framework proposed by Pacific Earthquake Engineering Research (PEER) Center [14–17] is considered more than other approaches in earthquake engineering research community (e.g., Consequence-Based Engineering Approaches [18]). The PEER's PBSA assessment framework relies on the following four modules: (1) seismic hazard analysis, (2) structural demand analysis, (3) component damage analysis and (4) component loss assessment. PBSA is accomplished by stringing the aforementioned four modules through total probability theorem (see Eq. (1)). The four variables associated with the aforementioned steps are: (1) Intensity Measure (*IM*), (2) Engineering Demand Parameters (*EDP*), (3) Damage measure (*DM*), and (4) Decision variable (*DV*). $G_{i|j}$ is the complementary cumulative distribution function (CCDF) of interim variable i conditioned on interim variable j. A brief description on each module pertaining to the research presented herein is discussed in the following.

$$P(DV > dv|IM = im) = \int\int G_{DV|DM}(dv|dm)|dG_{DM|EDP}(dm|edp)| |dG_{EDP|IM}(edp|im)| \quad (1)$$

2.1.1. Ground motion intensity measure (IM)

Ground motion intensity measure, *IM*, represents the intensity of seismic shaking at the site of the structure. *IM* plays an important role in seismic demand analysis where the structural response is estimated as a function of the chosen intensity measure. There are different *IMs* commonly used to represent the ground motion such as peak ground acceleration (*PGA*), peak ground velocity (*PGV*) and spectral acceleration at the first mode period of the structure (*Sa*). In this research, *PGA*, is selected as the *IM;* Padgett et al. [19] showed that *PGA* is an efficient *IM* for probabilistic studies of bridge portfolios.

Assembling a suit of ground motions that accurately characterizes the seismic hazard at the location of the bridge is crucial for seismic risk analysis. Another important aspect is to propagate uncertainty in the realization of hazard characteristics such as magnitude and epicentral distance. In order to satisfy the aforementioned requirements, a suite of 160 ground motions assembled by Baker et al. [20] for the PEER Transportation Research Program is adopted for the purpose of this study. The selected ground motions do not target a specific hazard and pertain to shallow crustal earthquakes with magnitudes ranging from 4.3 to 7.9. This set consists of 160 broad-band ground motions related to moderately large earthquakes at small distances. The set includes a group of 40 ground motions with strong velocity pulses experiencing near-fault directivity effects. The details of the selected suite can be found in Baker et al. [20].

2.1.2. Engineering demand parameters (EDPs)

Engineering Demand Parameters describe the seismic response of structural components; they are usually described in a matrix format. The rows of *EDP* matrix represent different realizations of the *EDP* vector. Assuming that ground motions used in seismic demand analysis represent independent earthquake events, the rows of this matrix are statistically independent. However, the *EDP* values in different components for a given ground motion (column of the matrix) are correlated [21]. These *EDPs* are estimated as a function of the *IM* through probabilistic seismic demand analysis. The response parameters should be selected wisely to provide a meaningful mapping to component damages. Before the selection of response parameters, bridge components are classified into performance groups to make the component assessment manageable. Each performance group consists of one or more bridge components whose performance is similarly affected by a particular engineering demand parameter.

The performance groups and their related *EDPs* considered in this study are: (1) PG-1 and column maximum drift (as its *EDP*), (2) PG-2 and column maximum residual drift, (3) PG-3 and superstructure deck maximum displacement, (4) PG-4 and maximum translational displacement of the bridge footing, (5) PG-5 and abutment maximum longitudinal displacement, (6) PG-6 and settlements related to residual vertical abutment displacement in the approaches, and (7) PG-7 and shear key maximum displacement. The aforementioned performance groups and their related *EDPs* are presented in Table 1.

2.1.3. Component damage measure (DM)

Damage Measure (*DM*) is a parameter that quantitatively describes the damage incurred by a structural component during seismic shaking. *DM* is often divided into a set of discrete damage states (*DS*), where each state is related to a distinct set of repair actions that would need to be implemented in order to bring the performance group back to its original undamaged state. The damage analysis is implemented to predict the damage at the performance group level as a function of the selected *EDP*.

For PG-1, three different damage measures are considered: concrete spalling, bar buckling [22], and complete failure [23], while PG-2 incorporates three damage states: thickening of the column, re-centering of the column, and column failure [24]. Refinishing of 25% and 50% of the superstructure deck area are two damage measures considered for the superstructure deck (PG-3). The damage measures of column and abutment foundations (PG-4) consist of enlarging pile cap and adding piles.

Since different components such as expansion joints, back wall and approach slabs at the bridge abutments have related repair methods, these components are lumped together as a single performance group (PG-5). Replacing joint seal assembly, replacing joint seal assembly as well as back wall repair, and replacing joint seal assembly, back wall, and approach slab are the three damage states assumed for PG-5. For PG-6 (abutment approach slab), vertical settlement related to the gradient of 1/62.5 and 1/31.5 are selected for the first and second damage states of approach slab, respectively [24]. The shear keys damage states are defined based on the damages detected in experimental tests at UC San Diego [25]. The first yield of the shear key on the backbone curve is defined as the first damage state. Repairing minor spalls and injecting cracks with epoxy are the repair strategies in this damage state. In the second damage state, the shear key reaches its ultimate strength and should be replaced. The damage states and their repair strategies related to different performance groups are presented in Table 1.

2.1.4. Decision variable (DV)

Decision variables are conduits relating the performance of the structure to decision makers. In this research, the bridge Repair Cost Ratio (RCR)–ratio of repair cost to replacement cost–of performance groups are selected as the DV. Repair cost in different performance groups at different levels of damage are estimated based on the