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Y. Xie\textsuperscript{1} and J. Zhang\textsuperscript{2}

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Probabilistic Seismic Evaluation of Bridges with Rocking Column-Foundation

Y. Xie\(^1\) and J. Zhang\(^2\)

**ABSTRACT**

Rocking column-foundation system is a promising design concept for bridge columns to reduce drift responses under earthquake hazards. This study assesses the seismic fragility of the rocking column-foundation system within a probabilistic framework. First, statistical tools are used to sample the class of single-column highway bridges in California that are designed with rocking isolation. The transient drift and rocking responses of the system are solved analytically using the equations of motion that incorporate superstructure mass inertia, column flexibility, the uplift condition, and rocking impact mechanisms. Subsequently, the probabilistic seismic demand analysis is constructed to identify the optimal intensity measures for conditioning the seismic demands of the system under three different conditions: (1) the system remains full contact throughout the time history; (2) the system experiences a coupled response of column vibrating and foundation uplifting; and (3) the system overturns under strong ground excitations. Finally, fragility analyses are conducted to estimate the damage and overturning probability of the rocking system under different levels of earthquake hazards. The superior seismic performance of the rocking system is further verified by comparing its fragility curves with those with the conventional fixed-base condition.

**Introduction**

Highway bridges are in general susceptible to earthquake damages, which require either detailed seismic design to ensure sufficient ductility in the columns [1] or the use of retrofit measures [2] and protective devices and systems [3]. Among various innovative systems, the new design concept of rocking column-foundation features a detached interface between the column footing and the rigid support underneath, which allows the bridge to uplift freely when subjected to strong earthquakes. This rocking system would be desirable to reduce overall seismic damages, minimize construction and repair time, and achieve lower cost. The rocking dynamics with rigid interface for small freestanding rigid blocks (e.g., electric transformers, tombstones) have been the subject of intense investigation during the past several decades [e.g., 4, 5]. In contrast to small rigid blocks, the rocking system proposed here for bridges requires considering the column flexibility.

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Albeit previous researches have characterized and predicted the seismic responses of flexible rocking structures using an idealized mechanical model [e.g., 6-8], it still remains unclear whether the system can be reliably implemented in a portfolio of highway bridges. The reasons are: (1) the rocking system is likely to overturn when subjected to earthquakes that consist of significant coherent velocity pulses [6]; (2) the salient rocking responses are substantially affected by ground motion characteristics [7]; (3) geometric parameters such as the size and slenderness of the rocking system have competing relations to affect the system stability [9].

This paper develops the seismic fragility curves for flexible rocking bridges. First, the nonlinear equations of motion of the system are solved numerically. Concurrently, a probabilistic framework is constructed for a portfolio of single-column continuous concrete box-girder bridges in California that are designed with rocking isolation. The framework features: (1) the statistical sampling of bridge cases; (2) the optimal probabilistic seismic demand model (PSDM) for column drift demand; and (3) the overturning probability of the rocking bridge. Furthermore, fragility curves are compared between rocking bridges and conventional designs (i.e., the fixed-base conditions) to evaluate the effectiveness of the rocking design.

**Rocking Column-foundation Design for Single-column Bent Bridge Class**

The bridges considered for rocking column-foundation typically are two- or three-span continuous concrete box-girder bridges with single-column bents. Geometric and material properties of this bridge class are based on in-house database obtained from the Caltrans along with a thorough review of bridge plans. Table 1 lists the uncertainty distributions of material properties and geometric parameters considered for the selected bridge class [10]. In addition, other parameters, such as column diameter and number of deck cells, contain discrete values with certain percentages of distribution, which are based on the California bridge inventory analysis by Mangalathu [11].

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Distribution</th>
<th>Type</th>
<th>μ</th>
<th>σ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compressive strength ($f_c$)</td>
<td>Mpa</td>
<td>Normal</td>
<td></td>
<td>29.03</td>
<td>3.59</td>
</tr>
<tr>
<td>Reinforcing steel yield strength ($f_y$)</td>
<td>Mpa</td>
<td>Lognormal</td>
<td></td>
<td>465</td>
<td>37.3</td>
</tr>
<tr>
<td>Longitudinal reinforcement ratio ($\rho$)</td>
<td>%</td>
<td>Uniform</td>
<td></td>
<td>2.25</td>
<td>0.52</td>
</tr>
<tr>
<td>Damping ($\xi_n$)</td>
<td>--</td>
<td>Normal</td>
<td></td>
<td>0.045</td>
<td>0.01</td>
</tr>
<tr>
<td>Span length</td>
<td>m</td>
<td>Lognormal</td>
<td></td>
<td>31.78</td>
<td>8.74</td>
</tr>
<tr>
<td>Approach to main span ratio (three-span bridge) ($L_2/L_1$)</td>
<td>--</td>
<td>Normal</td>
<td></td>
<td>0.57</td>
<td>0.13</td>
</tr>
<tr>
<td>Deck width ($B_d$)</td>
<td>m</td>
<td>Lognormal</td>
<td></td>
<td>9.78</td>
<td>1.98</td>
</tr>
<tr>
<td>Deck depth-to-span ratio ($H_d/L$)*</td>
<td>--</td>
<td>--</td>
<td></td>
<td>0.055</td>
<td>--</td>
</tr>
<tr>
<td>Column height ($H$)</td>
<td>m</td>
<td>Lognormal</td>
<td></td>
<td>6.63</td>
<td>0.87</td>
</tr>
</tbody>
</table>

* Depth-to-span ratio for bridge deck is considered as a constant of 0.055 for reinforced concrete box girder bridges per Mangalathu [11].

A typical bridge configuration at center bent is given in Fig. 1(a). The rocking system designed in this study features a detached rocking interface between the bottom of the footing and the rigid support underneath (Fig. 1(b)). Ideally, the footing width ($2b$) should be determined...
through an iterative design process, such that the bridges are able to uplift and rock under major earthquakes. In addition, the bridges shall remain stable (i.e., not overturned) when subjected to strong earthquakes. A tentative design with the footing width to column width ratio \(2b/b_c\) holds a uniform distribution (with mean value \(\mu = 1.5\) and standard deviation \(\sigma = 0.17\)) is considered.

(a) Genetic geometry of the selected bridge class

(b) Schematic of the rocking column-foundation for bridges

Figure 1. Selected bridge class with rocking column-foundation.

Total 140 statistically significant yet nominally identical bridge cases are generated by sampling across the range of parameters using the Latin Hypercube Sampling technique, which partitions each parameter distribution into 140 intervals of equal probability, and randomly selects one sample from each interval. As a result, the distribution of the rocking slenderness ratio \(\alpha = \tan^{-1}\left[b / (H + H_d / 2)\right]\) (dimensions are shown in Fig. 1) ranges from 8˚ to 15˚. In addition, the bridge’s vibrational natural period \(T_n\) ranges from 0.4s to 1.0s.

Analytical Modeling and Dynamics of the Rocking System

Analytical Modeling

Since the self-weights of the column and the footing are much less than the participating weight from the deck, the rocking bridge can be idealized as a two degree of freedom system (Fig. 1(b)) [6, 8, 12]. The idealized model consists of a point mass on an axially rigid yet translational flexible column that is connected to the rigid footing at the bottom. The system variables are the column translational drift, \(u\) and the rigid body rotation of the footing, \(\theta\).

Following Lagrange’s equation of energy equilibrium, the nonlinear equations of motion (EOMs) of the system at the uplift phase can be derived as [8]:

\[
\ddot{\theta} \left[H_t^2 + b^2 - \text{sgn}(\theta)2H_tu + u^2\right] + H_t \ddot{u} - 2b\dot{u}\text{sgn}(\theta)b - u \right]

-g[H_t \sin \theta - \text{sgn}(\theta)b \cos \theta + u \cos \theta] = -\ddot{u}_g[H_t \cos \theta + \text{sgn}(\theta)b \sin \theta - u \sin \theta] \tag{1}

\ddot{\theta} H_t + \ddot{u} + 2\xi_n \omega_n \ddot{u} + \theta^2\text{sgn}(\theta)b - u\right] - g \sin \theta + \omega_n^2 u \right] = -\ddot{u}_g \cos \theta \tag{2}
\]

where \(H_t\) is the effective height that equals to \(H + H_d/2\); \(b\) is the half width of the footing; \(\omega_n\) is the vibrational natural frequency of the column; \(\xi_n\) is the associated damping ratio; \(g\) is the gravitational constant; \(\ddot{u}_g\) is the acceleration magnitude of the input ground motion; and sgn is the signum function. The nonlinear and coupled EOMs can be solved numerically using ordinary
differential equation solvers in MATLAB.

When the ground motion is not strong enough to uplift the system, Eq. 2 will lead to the force equilibrium for the full contact condition; and Eq. 1 turns out to be the moment equilibrium around the pivot points. At full contact, Eq. 2 can be simplified as:

\[
\ddot{u} + 2\xi_n \omega_n \dot{u} + \omega_n^2 u = -\ddot{u}_g
\]

Eq. 3 indicates that before uplift, the structure responds identically to a typical SDOF oscillator, which can be fully characterized by the natural frequency \(\omega_n\) and damping ratio \(\xi_n\). Eqs. 1 and 2 fully captures the dynamics of the rocking system at different rocking phases. However, at each time when the rocking phase alters, instantaneous impact happens at the interface with some kinematic energy loss. Such non-continuous energy loss can be captured by assigning reduced velocity initial conditions right after each impact. The conservation of angular momentum is adopted here to calculate the post-impact velocity [8].

![Figure 2. Three response conditions of the rocking system under pulse inputs (\(a_p\) and \(\omega_p\) denote the magnitude and the frequency of the pulse inputs).](image)

**Three Response Conditions**

Rocking bridges respond to earthquake excitations with three different conditions. A testbed analysis is carried out for the rocking system with following parameters: the size of the bridge \(R = \sqrt{H_e^2 + b^2}\) is 6m (equivalently, the rocking frequency content is \(p_0 = \sqrt{g/R} = 1.28\text{rad/s}\)); the slenderness \(a = 20\˚\); the vibrational natural frequency is assumed as \(\omega_n = 10p_0 = 12.8\text{rad/s}\); and the system damping ratio \(\xi_n\) is 5%. Fig. 2 presents the system responses under a cosine-type pulse input with changing magnitudes and frequencies. As shown in Fig. 2, the bridge responds distinctly when the inputs change. Specifically, when the pulse input features a large characteristic length (i.e., \(2\pi a_p/\omega_p^2\) as defined by Makris and Black [13]), the rocking system will overturn with two different modes: Mode 1 with a single impact and Mode 2 without any impacts [4]. Conversely, the bridge will remain full contact and behave as a SDOF oscillator if the pulse input is not strong enough. It is also noted from Fig. 2 that the bridge primarily responds with a coupled rocking and oscillation behavior (i.e., the rocking area shown in the figure is much larger than the other two), which favors the utilization of the rocking column-foundation concept.
Fragility Models of Rocking Bridges

Fragility Model Approach

Analytical fragility models are used to develop the fragility curves in this study [2, 14]. The log-normal distribution assumption is utilized to convolve the bridge engineering demand parameter (EDP) model with the capacity limit state (LS) model:

$$P = P[EDP \geq C | IM] = \Phi \left[ \frac{\ln(S_{EDP} / S_c)}{\sqrt{\beta_{EDP|IM}^2 + \beta_c^2}} \right]$$

where $S_{EDP}$ and $S_c$ are the median values of the demand model and capacity model, respectively; $\beta_{EDP|IM}$ is the logarithmic standard deviation (dispersion) of the demand conditioned on the ground motion intensity measure (IM); $\beta_c$ is the dispersion of the capacity limit state model. The median demand $S_{EDP}$ and the dispersion $\beta_{EDP|IM}$ can be calculated by a linear regression analysis in the log-transformed space for the $EDP$-$IM$ pairs:

$$S_{EDP} = a \cdot IM^b$$

$$\beta_{EDP|IM} = \sqrt{\frac{1}{N - 2} \sum_{i=1}^{N} [\ln(edp_i) - \ln(S_{EDP})]^2}$$

where $a$ and $b$ are regression coefficients; $edp_i$ is the $i^{th}$ realization of the maximum demands obtained from simulations and $N$ is the number of simulations.

![Figure 3. Distributions of the selected 140 ground motions with respect to various IMs.](image)

(a) Peak ground acceleration ($PGA$)  
(b) Peak ground velocity ($PGV$)  
(c) Spectra acceleration at 1.0s ($S_{a,1.0s}$)

Ground Motion Selection

The fragility model relies on a significant number of nonlinear time history analyses, which requires selecting a large group of earthquake records. Previous studies have identified that for highway bridges, peak ground acceleration ($PGA$) and spectral parameters at the fundamental period of the bridge (i.e., spectral acceleration and displacement) are good choices of $IM$ [15, 16]. However, since the rocking behavior alters the dynamic characteristics of the bridge, 140 motion records from PEER center strong motion database are selected and used to evaluate optimal $IM$s for rocking cases. Fig. 3 presents the distributions of the selected 140 motions with respect to three major $IM$s, where considerable variances can be observed.
Probabilistic Seismic Demand Model (PSDM)

The PSDM is developed through identification of the optimal EDP-IM pairs for the rocking bridge. The EDPs of interest are peak column drift ratio $Dr_{\text{max}}$ and normalized peak uplift angle $\theta_{\text{max}}/\alpha$ of the foundation. For each EDP, twelve different IMs that are independent of the structural properties are considered. Table 2 lists the abbreviations of the IMs and their corresponding formulae.

<table>
<thead>
<tr>
<th>IM Abbreviation</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>$PGA$ (m/s$^2$)</td>
<td>$\frac{\pi}{2g} \int_0^{T_p} \left[ \ddot{u}_g(t) \right]^2 dt$</td>
</tr>
<tr>
<td>$PGV$ (m/s)</td>
<td>$\frac{1}{u_{g,max}} \int_0^{T_p} \left[ \ddot{u}_g(t) \right]^2 dt$</td>
</tr>
<tr>
<td>$PGD$ (m)</td>
<td>$\int_0^{T_p}</td>
</tr>
<tr>
<td>$CAV$ (m/s)</td>
<td>$\int_0^{T_p}</td>
</tr>
<tr>
<td>$CAD$ (m)</td>
<td>$S_a$ at 0.2s</td>
</tr>
<tr>
<td>$S_a$ at 1.0s</td>
<td>$\frac{\pi}{2g} \int_0^{T_p} \left[ \ddot{u}_g(t) \right]^2 dt$</td>
</tr>
</tbody>
</table>

The characteristics of optimal IMs identified from previous studies are used as the criteria to compare the twelve IMs shown in Table 2 [15, 16]. As for primary metrics, the logarithmic standard deviation $\beta = \beta_{EDP, IM}$ is to measure the efficiency, the modified dispersion $\zeta = \beta/b$ is to measure the proficiency, and the coefficient of determination $R^2_c$ is to measure the goodness of the regression. A good IM will feature small $\beta$ and $\zeta$ values, yet with $R^2_c$ close to 1.0. Moreover, the sufficiency of the IM is evaluated as a secondary factor by calculating the $p$-value for the linear regression of the residuals $\varepsilon$ with respect to ground motion characteristics, such as magnitude ($M$) and distance to epicenter ($r$). A small $p$-value (i.e., less than 0.05) suggests that the regressed coefficients on $M$ or $r$ are statistically significant and hence IM is insufficient.

By randomly pairing the 140 bridge models with the selected 140 ground motions, the PSDM is built for each EDP-IM pair of concern. The comparisons of various IMs are firstly carried out with respect to the column drift ratio $Dr_{\text{max}}$. Note that the rocking system may experience three different response conditions under earthquake excitations (i.e., it may remain full contact, may uplift, and may overturn). In this study, a separate analysis is conducted to derive the overturning fragility of the system. Fig. 4 provides the radar plots for the case where only one regression is carried out for the data with both full contact and rocking conditions. The primary IMs are compared by dividing each metric with respect to the pertinent maximum and minimum values. Ideally, the best IM should have the smallest $\beta$ value, $\beta_{\text{min}}$; the smallest $\zeta$ value, $\zeta_{\text{min}}$; and the largest...
$R_c^2$ value, $R_{c,max}^2$. As depicted in the figure, the IMs of $PGV$ and $S_{a,1.0s}$ are superior ones since their normalized metrics are all close to 1 and they outperform $PGA$.

Fig. 5 shows the PSDMs for column drift ratio $Dr_{max}$ conditioned on $PGA$, $PGV$, and $S_{a,1.0s}$. As shown in Fig. 5, $PGV$ and $S_{a,1.0s}$ are strong competitors for the optimal IM with close primary metric values which are superior than $PGA$. In addition, the calculated $p$ values indicate that both ground motion magnitude $M$ and epicentral distance $r$ affect the residuals $\epsilon$ when $PGA$ is used. Fig. 6 compares the primary IMs with the uniform models applied solely on the rocking response data. As is depicted, the uniform PSDMs with respect to $S_{a,1.0s}$ and $PGV$ yield better performance among the twelve IMs. Therefore, it can be concluded that $S_{a,1.0s}$ and $PGV$ are both preferred IMs for the PSDM of column drift ratio $Dr_{max}$. If the hazard computability (as is necessary for fragility analysis) is considered as an additional criterion for selecting an appropriate IM, $S_{a,1.0s}$ is then more desirable than $PGV$ since hazard maps and hazard curves are mostly available in terms of spectral acceleration at specific period values. Therefore, $S_{a,1.0s}$ is picked as the optimal IM for column drift ratio $Dr_{max}$.

Other than column drift ratio $Dr_{max}$, the uplift demand $\theta_{max}/\alpha$ is the other demand parameter of interest. A similar searching process is carried out for the rocking response condition to identify the optimal IM for $\theta_{max}/\alpha$. It is found that $PGV$ and $S_{a,1.0s}$ still excel the rest measures on the primary metrics of $\beta$, $\zeta$, and $R_c^2$. However, the absolute values of the efficiency measure $\beta$ for uplift demand $\theta_{max}$ are much larger than those for column drift demand $Dr_{max}$. For instance, the associated $\beta$ values of using $S_{a,1.0s}$ and $PGV$ are 0.96 and 0.94, respectively. The large $\beta$ values unveil a lack of distinguishable linear trend in the response of $\ln(\theta_{max}/\alpha)$ with respect to univariate IMs, which has also been discovered by Dimitrakopoulos and Paraskeva [17] for rigid rocking blocks. An
applicable way to overcome such vague linear relation of ln(θ_{max}/α) is the use of bivariate IMs [17]. In the current study, this process is omitted by considering following two reasons: (1) without overturning, the damages induced by uplifting and rocking impact is rather concentrated at the rocking interface, which can be substantially alleviated through special treatment of the column [18, 19, among others]; (2) there lack systematic studies to reasonably quantify the capacity limit states of foundation uplifting and rocking impact. Although fragility curves for foundation uplifting are not considered in here, the overturning fragility is developed to reflect the failure probability of bridge induced by rocking.

**Overturning Fragility of the Rocking System**

The overturning probability of the rocking bridge can no longer be treated as a quantification problem between the demand and the capacity. Instead, it is a categorical issue that the system either remains stable or overturns with ‘infinite’ structural responses. Therefore, the scaling-based multiple stripes analysis (MSA) method is adopted for this task [20]. Using the uplift PSDM results, ground motions are scaled with respect to S_{a,1.0s} at discrete levels. At each level, a stripe of 140 ground motions are used to conduct the time history analyses, which provide the fraction of ground motions that cause overturning. The overturning occurrence ratio can then be simply computed at each given S_{a,1.0s} level as:

\[ P_{ovt} = \frac{n_i}{N} \]  

where \( n_i \) is the number of overturning cases at \( i^{th} \) IM level, and \( N \) is the number of total simulation cases at each IM level (\( N = 140 \) in this study). The discrete data points of overturning ratio can be further regressed as a continuous curve by using the maximum likelihood fitting technique [20].

**Seismic Fragilities of Rocking Bridges**

As shown in Eq. 4, capacity LS values are needed to develop the fragility curves of the column. Based on the column tests and expert surveys, previous studies have determined various damage index measures for bridge columns. Recently, a new group of curvature ductilities have been proposed for California bridges to account for the evolution of design standards over time [14]. Because the PSDM is built with respect to column drift ratio, the curvature ductility values therein are converted to the corresponding displacement ductility values by considering a linear relation in between [21]. Table 3 lists the associated damage state threshold values for bridge columns that are classified by three significant design eras (i.e., the Pre 1971 design era with poor reinforcement detailing; the 1971-1990 design era with intended flexural failure; and the Post 1990 design era with ductile failure in the columns) [14]. In Table 3, CDT-0, CDT-1, and CDT-2 denote the damage states of concrete cracking; minor cover spalling; and major spalling including larger shear cracks, exposed core and reinforcement yield, respectively. CDT-3 is the collapse state with loss of confinement, longitudinal bar buckling or rupture, or crushing of the inner core and large residual drifts. \( \beta_c \) is the associated dispersion for each damage state, which is assumed as 0.35 [14].

The two failure modes of the bridge (i.e., column collapse versus system overturn) are indeed correlated with each other. While for simplicity, such correlation is not taken into account in this study. For comparison purpose, the column damage probabilities with the fixed-base design are developed by using nonlinear time history analyses with fiber section column elements. Fig. 7
presents the seismic fragility curves of the single-column bridge class in California with and without rocking isolation. In particular, the overturning fragility calculated by using the MSA method is provided in the bottom right figure. As is seen in the figure, the proposed rocking design successfully reduces the seismic fragilities of the single-column bridge class in California. The advantages of using the rocking design can be especially recognized for the bridges with Pre 1971 design criteria, where significant fragility reductions can be observed. Moreover, comparisons of the fragility curves for reaching the CDT-3 (collapse) state demonstrate that the rocking design has the lowest overturning failure probability among all case scenarios. Namely, the overturning failure of the rocking bridges is less likely to happen when comparing with the column collapse.

Fig. 7 verifies the advantage of the rocking isolation design for highway bridges in California.

Table 3. Column curvature and displacement ductility values for different damage states.

<table>
<thead>
<tr>
<th>Damage index</th>
<th>Design Era</th>
<th>CDT-0</th>
<th>CDT-1</th>
<th>CDT-2</th>
<th>CDT-3</th>
<th>$\beta_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement ductility</td>
<td>Pre 1971</td>
<td>0.96</td>
<td>0.98</td>
<td>1.00</td>
<td>1.04</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>1971-1990</td>
<td>1.00</td>
<td>1.20</td>
<td>1.50</td>
<td>1.80</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>Post 1990</td>
<td>1.00</td>
<td>1.60</td>
<td>2.40</td>
<td>3.20</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Figure 7. Seismic fragility comparisons of rocking bridges and conventional bridges.

Conclusions

This study has developed fragility curves for the class of single-column highway bridges in California that are designed with rocking isolation. Distinct dynamic characteristics of the rocking system have been captured and reflected in the seismic fragility analysis. A fragility comparison to the conventional bridges has been carried out. It is found that $S_{a,1.0s}$, the spectral acceleration at the period of 1.0s, is the optimal IM for conditioning both the column drift demand and the normalized peak uplift angle for a portfolio of single-column rocking bridges. The fragility analysis reveals that rocking isolation design yields much lower overturning failure probability.
than that of column collapse and is especially beneficial for aged bridges that have poor seismic design details.

References


