SEISMIC RELIABILITY ASSESSMENT OF ROCKING BRIDGE BENTS WITH FLAG-SHAPED HYSTERETIC BEHAVIOR

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Abstract

Rocking as a means of seismic isolation relies on utilizing the rotational inertia of the structure through (purposely) activated dynamic motion (i.e. rigid body rotation). This is a radically different concept than conventional seismic design that regards dynamic motion as an “unpleasant” by-product of structural deformation. Rigid body rotations around predefined pivot points isolate the structure during strong ground motions. Therefore, stresses on the structure are significantly reduced, relieving it from deformation and damage. The mechanical configuration of the rocking frame has been proposed as a “damage avoidance design” for bridges. The rocking frame can be either freestanding or hybrid when supplemented with (unbonded) central tendons and energy dissipaters exhibiting flag-shaped hysteretic behavior. From the standpoint of practical bridge engineering applications, a question that persists is that of the seismic reliability of the (hybrid) rocking behavior.

Motivated by the unpredictability of the seismic behavior of rocking structures, this study deploys a probabilistic approach to assess the seismic fragility of hybrid rocking bridge bents. It focuses on slender bridge frames that exhibit planar rigid rocking behavior with negative, zero, or positive post-uplift lateral stiffness. Another limitation of this work is that it ignores the fracture of the supplemental devices. This practically means that frames with zero and/or positive lateral stiffness do not overturn no matter how large the rocking rotation is. The analysis herein aims to shed light on the influence of the characteristics of the hybrid frame (stiffness of the tendons and hysteretic characteristics of the dissipaters) on the seismic fragility of the bridge bent. To this end, it compares various hybrid systems with the archetypal freestanding rocking frame. The analysis considers ground motions with near-fault characteristics, either pure coherent pulses, or synthetic ground motions that include also a stochastic high-frequency component.

This study offers analytical fragility curves under (synthetic) near-fault ground motions. The proposed fragility curves are either based on a univariate or bivariate intensity measures. In both cases, the fragility analysis hinges on dimensionless and physically consistent intensity measures which minimize the scatter of the response. This work shows that there is a peak ground acceleration limit below and above which the response of the (hybrid) rocking frames scales differently. The results indicate that the examined rocking structures are more vulnerable to pure low-frequency pulses than to synthetic ground motions with additional high-frequency stochastic component of the same PGV/PGA value. Further, none of the rocking design solutions (hybrid or freestanding) outperforms all others in all cases. Thus, identifying optimal design solutions is an issue that merits further study. Finally, the analysis verifies the sensitivity of the examined rocking structures to more than one parameters of the considered near-fault ground excitation.

Keywords: rocking bridges, flag-shaped hysteretic behavior, self-centering, fragility curves, probability of exceedance
1. Introduction

Rocking behavior is one of the reasons many ancient structures survived millennia in earthquake-prone regions [1]. A peculiar characteristic of rocking dynamics is the negative stiffness that prevents the structure from resonance under constant frequency (e.g., harmonic) excitation [2]. This aroused the interest of many researchers to study the dynamics of various rocking structures ([3-10] among others). Of particular interest for bridge engineering is the rocking frame configuration of Fig. 1 proposed originally by [11]. In this context, [12-14] revealed that rocking piers can exhibit large drifts (6% to 10%) with minor damage and/or residual displacements. Experimental studies [15, 16] showed that the residual drifts of posttensioned rocking columns are negligible (0.4%) compared to the pertinent drifts of conventional monolithic columns (6.8%) with the same peak drift ratios; in accordance with [17].

Such self-centering systems aim to eliminate residual displacements after strong earthquakes. Many researchers ([18-23] among others) combined additional re-centering with energy dissipation devices and proposed “hybrid rocking systems” with flag-shaped hysteretic behavior (FSHB). These studies compared numerically and/or experimentally the seismic performance of rocking versus conventionally designed monolithic piers. The design premise of these re-centering rocking structures is to concentrate the potential damage to the sacrificial replaceable energy dissipaters isolating the remaining structure from damage. The rocking piers showed higher self-centering capacity and lower residual displacements compared to the monolithic solution. In this context, [24] examined a hybrid rocking frame with columns unequal in height (asymmetric) and compared its stability with the pertinent symmetric configuration. Recently, the authors [25] examined hybrid FSHB rocking frames and revealed the diverse role on the seismic performance of the prestress. In particular, prestressing the tendons can be beneficial for small rocking rotations and indifferent or even detrimental for large rocking rotations as the size of the columns increases. Further, that study verified the sensitivity of the different rocking design solutions to the characteristics of the considered ground excitations.

Even though it is hard to overstate the significance of the deterministic methods in investigating the rocking behavior, rocking dynamics remains unpredictable. This increased incentive to assess the seismic performance of rocking behavior via probabilistic methods. In this context, [26] developed overturning fragility curves for unrestrained blocks, based on physical and numerical experiments, while [27, 28] offered fragility curves for bridges with rocking foundations and frames with rocking columns respectively. Recently, [29] offered normalized fragility curves that estimate the overturning tendency of freestanding rocking frames and unveiled the existence of a critical peak ground acceleration limit, above and below which the rocking response scales differently.

Building on previous work on hybrid FSHB rocking frames [25], the main objective of the present paper is to assess the seismic fragility of hybrid FSHB rocking bridge bents. To this end, it compares the fragility of hybrid rocking frames with negative, zero and/or positive stiffness with the pertinent fragility of the archetypal freestanding frame when subjected to near-fault ground excitations adopting a probabilistic framework.

2. Analytical modelling

This section examines (analytically) a hybrid rocking bridge bent which exhibits flag-shaped hysteretic behavior. The analysis considers slender structures designed to rock and it ignores sliding between the contacting bodies. The rocking frame of Fig. 1 is enhanced with central (linear-elastic) slack (unbonded) tendons and (nonlinear-hysteretic) buckling-restrained braces (BRBs) at the bottom of the piers. This study ignores the fracture of the tendons and the dissipaters, which for a different hybrid rocking system is discussed in [24]. To describe the hysteretic behavior of the BRBs, the Bouc-Wen model is adopted [30, 31]. The restoring dissipating force from each BRB is expressed as:

\[ F_D = ε k_d u + (1 - ε) k_d u_s z(t) \]
where $\varepsilon$ is the post-yield to pre-yield elastic stiffness ($k_d$) ratio, $u$ is the axial deformation of the brace and $u_y$ is the yield displacement. $z(t)$ is a dimensionless hysteretic parameter that is governed by:

$$z(t) = \frac{1}{u_y} \left[ \dot{u}(t) - \gamma |\dot{u}(t)| z^{|\varepsilon - 1|} - \beta \ddot{u}(t) |z|^n \right]$$ (2)

Following [32], parameters $\beta, \gamma, n$ and $\varepsilon$ control the hysteresis and are taken equal to 0.55, 0.45, 1 and 0.025 respectively, while the yield displacement is considered 3.5 mm.

2.1 Flag-shaped hysteretic behavior

Assuming positive (clockwise) rotations, the total restoring moment (for small rocking rotations and slender structures) offered by the weight of the two columns and the cap-beam and the supplemental re-centering and energy dissipation devices is equal with [25]:

$$M_R = \frac{1}{2} \left[ m_{AB} g R + m_{BC} g R + \frac{m_{AB} + m_{BC}}{R^2} \right]$$ (3)

This work assumes the design parameters $\rho_t$ and $\rho_d$ vary within $0 \leq \rho_t \leq 0.7$ and $0 \leq \rho_d \leq 5$. On the other hand, the overturning moment induced by the ground excitation ($M_{ot}$) becomes:

$$M_{ot} = \frac{1}{2} \left[ m_{AB} g R + m_{BC} g R \right]$$ (5)

Fig. 1 – The examined hybrid FSHB rocking bridge bent (a) during counter-clockwise rotation, (b) at rest position and (c) during clockwise rotation
In the tendon's and the dissipater's stiffness cause increase in the overall lateral stiffness of the structure. For positive stiffness systems, it holds (Eq. (3)):

$$\rho_\lambda + \rho = \frac{1}{4}$$

(6)

### 2.2 Dynamics of the hybrid rocking bridge bent

Rocking commences when the overturning moment becomes equal with the restoring moment (Eqs (3), (5)). Then, the minimum ground acceleration to initiate rocking becomes: $a_{g,\min} = g \tan \alpha$. When the ground acceleration exceeds this limit, the frame initiates rocking. The equation which describes the motion of the hybrid FSHB rocking bent can be derived using the Lagrange’s equation:

$$\frac{d}{dt} \left( \frac{\partial L}{\partial \dot{\phi}} \right) - \frac{\partial L}{\partial \phi} = Q$$

(7)

where $L=T-V$ in which $T$ is the total kinetic energy, $V$ is the total potential energy of the system and $Q$ is the generalized force. Following [25], the equation of motion becomes:

$$\ddot{\phi} = -\frac{1 + 2\gamma_\alpha}{1 + 3\gamma_{\gamma}} p^2 \left[ \sin(\alpha \text{sgn}(\phi) - \dot{\phi}) + \frac{\dot{\phi}}{g} \cos(\alpha \text{sgn}(\phi) - \dot{\phi}) \right] + 4(\rho_\lambda + \rho \gamma) \sin \phi + 8(1 - \varepsilon) \rho \frac{\dot{\phi}}{2} z(t)$$

(8)
where \( p \) is the frequency parameter of the column of the frame equal with \( p = \sqrt{3g/4R} \). \( \text{sgn}(\phi) \) is the sign function and \( \gamma_m = m_{nc}/2m_{ab} \) is the mass ratio. \( \phi_y \) is the rotation which corresponds to the yield displacement \( u_y \) of the BRBs. During rocking, the smooth motion of the frame is interrupted by nonsmooth events (impacts). Therefore, (kinetic) energy is lost at each impact. To capture the impact behavior, this study adopts the classical impact model [33] according to which the coefficient of restitution \( \eta \) is defined as the ratio of the post- to pre-impact angular velocity. Applying the momentum-impulse theorem the coefficient of restitution becomes [24]:

\[
\eta = \frac{\dot{\phi}^+}{\dot{\phi}^-} = \frac{1 - \frac{3}{2} \sin^2(\alpha) + 3\gamma_m \cos(2\alpha)}{1 + 3\gamma_m}
\]

The analysis herein adopts a coefficient of restitution equal to 0.92.

3. Seismic response analysis

3.1 Pulse-type and synthetic ground motions

To assess the seismic reliability of the (freestanding and hybrid FSHB) rocking frame of Fig. 1, this study employs pulse-type ground motions [34] and combined synthetic ground motions (CSGMs) [35, 36]. CSGMs are a combination of low-frequency pulses with high-frequency components constructed according to the stochastic method of Boore [37]. The Mavroeidis and Papageorgiou (M&P) wavelet [34] simulates the near-fault low-frequency component of the CSGMs. Its velocity time-history expression is:

\[
\dot{u}_p(t) = \begin{cases} 
\frac{A}{2} \cos\left(\frac{\omega_p}{\gamma_p}(t-t_0)\right) & \text{if } t_0 - \frac{\pi\gamma_p}{\omega_p} \leq t \leq t_0 + \frac{\pi\gamma_p}{\omega_p} \\
0 & \text{otherwise}
\end{cases}
\]

where \( A, \omega_p, \gamma_p \) and \( t_0 \) describe the velocity amplitude of the envelope of the pulse, the angular frequency, the phase angle, the number of half cycles and the time shift to specify the epoch of the envelopes peak respectively. The predominant period \( T_p = 2\pi/\omega_p \) (in sec) and the velocity amplitude \( V_p \) (in cm/sec) of the pulse for every moment magnitude \( M_w \) and distance from the fault \( R_{rup} \), are obtained from [38]:

\[
\begin{align*}
\log(T_p) &= -2.9 + 0.5M_w \\
\log(V_p) &= 2.04 - 0.032R_{rup}
\end{align*}
\]

where the distance \( R_{rup} \) is defined as the closest distance of the site to the fault [34]. This study assumes that the moment magnitude \( M_w \) ranges from 5.5 to 7.5 with a step of 0.5 while the distance \( R_{rup} \) varies from 5 to 20 km with a step of 2.5 km [1]. Further, it is assumed that the logarithms of \( T_p \) and \( V_p \) follow a normal distribution with standard deviation equal to 0.143 and 0.187 respectively, while \( \gamma_p \) and \( V_p \) follow also a normal distribution with mean values 1.93 and 1.83 and corresponding standard deviations 0.47 and 0.98 accordingly [38]. For each of the \( M_w, R_{rup} \) combination, 100 samples of the random variables \( V_p, T_p, \gamma_p \) and \( V_p \) are generated, and from Eq. (10) 3500 M&P pulses are constructed. Details of the procedure followed to generate the pertinent M&P pulses and the CSGMs are given in [29, 35, 36] and references therein.

3.2 Engineering demand parameter and intensity measures

This section evaluates the seismic performance of the freestanding and hybrid FSHB rocking bridge bent adopting a probabilistic framework. Consider the frame of Fig. 1 with a cap-beam 13 m wide and height \( 2h = 2 \) m. The frame has two (square) columns with \( 2b = 1.4 \) m base length, same density and height \( 2H = 9.8 \) m each. Both columns have the same slenderness \( \alpha = 0.14 \text{ rad} \) (8.1°) and frequency parameter \( p = 1.22 \text{ rad/s} \), while the distance between them is \( L = 8 \) m. The cap-beam/column mass ratio \( \gamma_m \) is taken as 5. For the purposes of the
present section, appropriate engineering demand parameters (EDPs) and intensity measures (IMs) need to be defined. This study adopts as an EDP the absolute peak rocking rotation $\phi_{\text{max}}$ scaled with respect to the slenderness $\alpha$ of the column of the frame:

$$E\text{DP} = \frac{|\phi_{\text{max}}|}{\alpha} \quad (12)$$

Eq. (12) implies that for values larger than zero the frame rocks, whereas for high enough values (e.g. $E\text{DP} > 1.5$) the frame overturns. Recall that, this study assumes that only frames with negative stiffness overturn. Zero and positive stiffness systems do not overturn regardless of the rocking rotation. Hence, the two pertinent performance levels which assess the vulnerability of the rocking bridge bent are: (i) $LS1$, which denotes the initiation of rocking and (ii) $LS3$, which corresponds to excessive rocking rotations for systems with zero and/or positive stiffness and overturning for negative stiffness frames. In addition, an intermediate limit state $LS2$ which indicates safe rocking during the seismic response is also considered. The threshold value of $LS2$ is assumed to be an order of magnitude smaller than the slenderness [29].

Fig. 3 plots the response of the freestanding rocking frame with respect to the velocity amplitude $V_p$ (or $\text{PGV}$) and the predominant period of the pulse $T_p$ when subjected to M&P (or CSGMs). Fig. 3 categorizes the results into three groups: (i) the “non-rocking” cases where the frame does not initiate rocking, (ii) the “safe rocking” cases where the frame rocks without overturning and (iii) the “rocking overturning” cases where the frame overturns. Note that, before the frame overturns it must first sustain extensive (“critical”) rocking rotations [25], and overturning occurs when the structure becomes dynamically unstable. Hence, the threshold of $LS3$ does not correspond to any finite rotation, but to numerically infinite values. Observe in Fig. 3 the distinctive line that separates the “non-rocking” from the “safe rocking” cases. The slope of this line corresponds to the minimum ground acceleration for rocking initiation ($\ddot{u}_{g,\text{min}} = a_{g,\text{min}} = g \tan \alpha$). When the low-frequency (M&P) ground motions are infiltrated with the high-frequency component though, this distinctive line vanishes (Fig. 3(b)). This is because the complementary high-frequency component triggers rocking for cases where the M&P pulse alone cannot. Therefore, the seismic response of the freestanding frame becomes less ordered when subjected to CSGMs. Subsequently, for CSGMs the freestanding frame becomes more unstable with more rocking overturning cases than when subjected to low-frequency M&P pulses. Fig. 3 also verifies the sensitivity of the response to different parameters of the ground excitation, since increase of both the $T_p$ and $V_p$ (or $\text{PGV}$ for CSGMs) cause overturning. Similar conclusions are drawn for the hybrid FSHB rocking frames when subjected to M&P pulses and CSGMs. For the economy of space though, the corresponding plots are omitted.

The performance of the considered IMs is evaluated with respect to their ability to reduce the dispersion of the response. For brevity, this study adopts IMs which are found to be the most proficient for freestanding
rocking structures [29, 39]:

\[ IM_1 = \frac{PGA}{g \tan \alpha}, \quad IM_2 = \frac{PGA}{pPGV} \]  

(13)

In addition, it is assumed that the median demand \( D_m \) and the considered IMs follow a scale law

\[ D_m = a \left( IM_x^b IM_y^b \right), \]

which in a logarithmic scale it becomes:

\[ \ln D_m = \ln a + b_1 \ln IM_x + b_2 \ln IM_y \]  

(14)

\( IM_x, IM_y \) denote the adopted intensity measures (Eq. (13)), while \( a, b_1 \) and \( b_2 \) are coefficients determined by multilinear regression analysis. Fig. 4 plots the “safe rocking” cases for the hybrid FSHB rocking bridge bent (with negative stiffness) of Fig. 1 in a three-dimensional space. Fig. 4 shows that the rocking response scales differently for low intensity versus high intensity ground excitations. In particular, the response lies consistently on two different planes regardless of the considered ground excitation, i.e. M&P pulses (Fig. 4(a)) or CSGMs (Fig. 4(b)). The transition between the two planes occurs at constant ground acceleration. This acceleration limit differs for each hybrid rocking frame and depends (mainly) on the dissipater’s stiffness (\( \rho_d \) parameter). Specifically, with the absence of the BRBs (\( \rho_d = 0.0 \)), the transition happens approximately when \( PGA/(gtan \alpha) = 1.4 \), while for FSHB rocking frames with \( \rho_d = 5.0 \) when \( PGA/(gtan \alpha) = 1.5 \). Recall that, for the freestanding frame, the acceleration boundary is approximately at \( PGA/(gtan \alpha) = 1.3 \) [29].

Fig. 4 – Three-dimensional illustration of the “safe rocking” cases of the hybrid FSHB rocking frame with negative lateral stiffness subjected to (a) M&P pulses and (b) CSGMs
4. Fragility analysis

This section compares the seismic fragility of different rocking frames (i.e. freestanding and hybrid) by calculating (i) the conditional probability $P_{ex}$ that a ground motion with $IM = x$ will cause the exceedance of the capacity limit $C$ without the occurrence of overturning, and (ii) the probability $P_{ro}$ that the considered ground motion will cause overturning. Therefore, the failure probability $P_{fr}$ can be expressed as [29]:

$$P_{fr} = P_{ro} + (1 - P_{ro}) P_{ex} (D > C | IM = x)$$  \hspace{1cm} (15)

Recall that, hybrid FSHB rocking frames with zero and/or positive lateral stiffness do not overturn no matter how large the rocking rotation is. Hence, the rocking overturning probability is calculated only for the cases where the structure exhibits negative stiffness. Assuming that both the demand $D$ and the capacity $C$ follow lognormal distribution, the probability of exceedance can be written as:

$$P_{ex} = P_{ex} (D > C | IM = x) = \Phi \left( \frac{\ln x - \mu}{\beta} \right)$$  \hspace{1cm} (16)

where $\Phi()$ denotes the standard normal cumulative distribution function, $x$ are the values of the EDP and $\mu, \beta$ stand for the median and the dispersion (or standard deviation) of the structural demand.

Fig. 5 plots the probability that the examined rocking frames exceed the considered capacity limit for various $PGA/(pPGV)$ ratios, when subjected to the same low-frequency M&P pulses. The examined capacity limits stand for small ($EDP > 0.1$) and large rocking rotations ($EDP > 1.0$). Fig. 5 shows that for small

![Fig. 5](image-url)
PGA/(pPGV) values the hybrid FSHB rocking frames exhibit smaller response rotations compared to the pertinent freestanding. This, however, might come at the cost of higher energy demands by the dissipaters [25]. On the contrary, as PGA/(pPGV) increases the freestanding frame becomes more stable and outperforms the hybrid.

When the examined rocking frames are subjected to CSGMs, due to the additional high-frequency component in the low-frequency M&P pulses, the response becomes less ordered (see Fig. 3(b)). Hence, as Fig. 6 shows, in most cases, the hybrid rocking frames sustain smaller rocking rotations compared to the freestanding. Again, for high enough PGA/(pPGV) values, the performance of the freestanding becomes comparable and slightly surpasses the performance of the hybrid FSHB frame.

This work also calculates the probability $P_{ro}$ that a ground motion with $IM = x$ will cause overturning. Following [29], the overturning probability becomes:

$$P_{ro} = P_{ro}(D \geq \text{overturning} | IM = x) = \Phi \left( \frac{\ln x - \mu}{\beta} \right) = \frac{1}{2} \left[ 1 + \text{erf} \left( \frac{\ln x - \mu}{\beta \sqrt{2}} \right) \right]$$ (17)

where erf is the error function, $x$ stands for the EDP values while $\mu$ and $\beta$ represent the mean value and the standard deviation. Fig. 7 plots the overturning probability for the freestanding and hybrid FSHB rocking frame exhibiting negative stiffness calculated with the aid of the maximum likelihood estimation (MLE) method. Fig. 7 illustrates the mitigated response of the hybrid FSHB rocking frame compared to the freestanding. Recall that, this might come at the cost of increased energy demands by the dissipaters [25], and hence it should be further investigated possibly through additional EDPs. Keeping, however, the overall stiffness of the structure constant and increasing (only) the stiffness of the BRBs ($\rho_d$ parameter), Fig. 7 shows that the response of the hybrid

![Fig. 6 – Probability of exceedance for the examined freestanding and hybrid FSHB rocking frames with $\rho_d = 5$ subjected to CSGMs](image-url)
FSHB frame is further reduced. Interestingly though, an M&P pulse alone with the same PGV/PGA leads to higher probability of overturning. This means that if the M&P pulse is capable of initiating rocking, it is more likely to cause overturning than a CSGM excitation of the same PGV/PGA value.

5. Conclusions

This paper examines the seismic fragility of slender rocking bridge bents with fundamentally different behavior. In particular, it compares the freestanding rocking frame with the pertinent hybrid under (synthetic) near-fault ground motions adopting a probabilistic framework. The hybrid rocking frame is enhanced with central slack tendons and buckling-restrained braces exhibiting flag-shaped hysteretic behavior. The work reported herein indicates that none of the examined rocking configurations is the optimal design solution in all cases. Further, the results unveil that the response of the hybrid rocking frame scales differently for low intensity versus high intensity ground excitations: the peak rocking rotation lies consistently on two distinct planes. The examined rocking structures are more vulnerable to pure low-frequency pulses than to combined excitations, of the same PGV/PGA, values which include pulses together with a stochastic high-frequency component. Finally, the study verifies the sensitivity of the rocking behavior to more than one parameters of the ground excitation.

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7. References


