EFFETCS OF CLASS B SITE ON THE SEISMIC RELIABILITY OF BASE-ISOLATED STEEL SYSTEMS

Paolo Castaldo*, Guglielmo Amendola, Bruno Palazzo

Department of Civil Engineering, University of Salerno, Italy

SUMMARY: Seismic reliability of steel structures isolated using the frictional pendulum system bearings and subjected to artificial earthquake ground motions is studied herein. The superstructure is idealised as a linear shear-type flexible building as well as the FPS devices are described by adopting a widespread model which considers the variation of the friction coefficient with the velocity. The uncertainty affecting both the seismic inputs, modelled as non stationary random processes within the power spectral density method, and the friction coefficient at large velocity is considered through appropriate probability density functions. Incremental dynamic analyses are developed in order to evaluate the fragility curves related to both superstructure and isolation level. Finally, considering the seismic hazard curve related to a site near Sant’Angelo dei Lombardi (Italy), the seismic reliability of the overall steel system is evaluated.

KEYWORDS: Seismic isolation, steel superstructure, power spectral density method, Monte Carlo simulations, Latin hypercube sampling method, seismic reliability

1. INTRODUCTION

In the last decades, seismic dissipation and isolation through friction pendulum system (FPS) [Christopoulos and Filiatrault, 2006; Zayas et al., 1990] have emerged as very effective techniques for the protection of building frames which, even if designed according to the most advanced codes, suffer severe damages under strong earthquake events [De Iuliis et al., 2010; Bonessio et al., 2012]. Over the years, many works have developed new design strategies and methodologies [Castaldo, 2014; Castaldo and De Iuliis, 2014; Castaldo et al. 2016d; Castaldo et al. 2016e; De Iuliis and Castaldo, 2012; D’Ayala and Benzoni, 2012; Longo et al., 2012; Palazzo et al. 2014b; Palazzo et al., 2015] as well as probabilistic analyses in structural dynamics, structural reliability methods, and reliability-based analysis have been presented [Giugliano et al., 2011a; Giugliano et al., 2011b; Longo et al., 2009]. Barroso and Winterstein [Barroso and Winterstein, 2002] evaluated the seismic performance of steel buildings isolated with FPS bearings by taking into account the variability of both the seismic intensity and the record characteristics. Seismic reliability analyses of 2D and 3D systems isolated by FPS bearings have been carried out in [Castaldo et al. 2015; Palazzo et al., 2014a; Castaldo et al. 2016a; Castaldo et al. 2016b; Castaldo et al. 2016c] by accounting for the randomness of both the isolator properties (i.e., coefficient of friction) and of the earthquake main characteristics by defining a reliability criterion to assist the design of the isolator dimensions in plan. In [Castaldo and Tubaldi, 2015], the influence of FPS bearing properties and of the structural parameters on the seismic performance of base-isolated structures.

*Corresponding author: Paolo Castaldo, Department of Civil Engineering, University of Salerno, Italy.
Email: pcastaldo@unisa.it
through the nondimensionalization of the motion of equations is analyzed by providing useful results for seismic reliability analyses. In [Castaldo and Ripani, 2016], the optimal values of the friction coefficient are defined as a function of the system properties and of the soil condition in order to minimize the superstructure response in terms of displacement relative the base mass.

In this work, seismic reliability of steel structures isolated using the frictional pendulum system (FPS) bearings and subjected to artificial earthquake ground motions is studied herein by considering both the friction coefficient and soil characteristics as random variables. The superstructure is idealised as a linear shear-type flexible building as well as the FPS devices are described by adopting a widespread model which considers the variation of the friction coefficient with the velocity [Mokha et al., 1990]. The uncertainty affecting both the seismic inputs, modelled as non stationary random processes within the power spectral density method [Pinto et al., 2004; Shinozuka and Deodatis, 1991; Talaslidis et al., 2004], and the friction coefficient at large velocity is considered through appropriate probability density functions. Incremental dynamic analyses are developed in order to evaluate the probabilities exceeding different limit states related to both superstructure and isolation level. According to the PEER-like modular approach [Cornell and Krawinkler, 2000], in the final part of the work, considering the seismic hazard curve related to a site near Sant’Angelo dei Lombardi (Italy) [NTC08, 2008], regarding a structural steel system isolated by FP bearings with a design life of 50 years, reliability curves are derived.

2. EQUATION OF MOTION OF BASE-ISOLATED STRUCTURES

The equation of motion governing the response of a single-degree-of-freedom (SDOF) system on single concave FPS isolation devices to the seismic input $\ddot{u}_s(t)$ is:

$$
\begin{align*}
    m_s \cdot \ddot{u}_s(t) + c_s \cdot \dot{u}_s(t) + k_s \cdot u_s(t) &= -m_s \cdot \left[ \ddot{u}_g(t) + \ddot{u}_b(t) \right] \\
    m_b \cdot \ddot{u}_b(t) + f_b(t) + c_b \cdot \dot{u}_b(t) - c_s \cdot \dot{u}_s(t) - k_s \cdot u_s(t) &= -m_b \cdot \ddot{u}_g(t)
\end{align*}
$$

(1)

where $u_s$ denotes the displacement of the superstructure relative to isolation bearing, $u_b$ the isolator displacement relative to the ground, $m_s$ and $m_b$ respectively the mass of the superstructure and of the base, $k_s$ and $c_s$ respectively the superstructure stiffness and inherent viscous damping constant, $c_b$ the bearing viscous damping constant, $\ddot{u}_g(t)$ the ground motion input, the dot differentiation over time, and $f_b(t)$ denotes the FPS bearing resisting force, that is expressed as:

$$
f_b(t) = k_b \cdot u_b(t) + \mu(\dot{u}_b)(m + m_b)gZ(t)
$$

(2)

where $k_b = (m_s + m_b)g/R$, $g$ is the gravity constant, $R$ is the radius of curvature of the FPS, $\mu(\dot{u}_b(t))$ the coefficient of sliding friction, which depends on the bearing slip velocity $\dot{u}_b(t)$, and $Z(t) = \text{sgn}(\dot{u}_b)$, where $\text{sgn}(\cdot)$ is the sign function. Experimental results [Constantinou et al., 1990; Constantinou et al., 2007; Mokha et al., 1990] suggest that the coefficient of sliding friction of Teflon-steel interfaces obeys to the following equation:

$$
\mu(\dot{u}_b) = f_{\text{max}} - Df \cdot \exp\left(-\alpha|\dot{u}_b|\right)
$$

(3)

in which $f_{\text{max}}$ represents the maximum value of friction coefficient attained at large velocities of sliding, $f_{\text{min}} = f_{\text{max}} - Df$ represents the value at zero velocity. By dividing Eqn. (1a) by $m_s$,
and Eqn. (1b) by $m_b$, and introducing the mass ratio $\gamma = m_s / (m_s + m_b)$ [Kelly, 1997], Eqn. (1) can be rewritten as:

$$
\ddot{u}_s(t) + 2\zeta_s \omega_s \dot{u}_s(t) + \omega_s^2 u_s(t) = -[a_s(t) + \ddot{u}_b(t)]
$$

$$
\ddot{u}_b(t) + \frac{1}{1-\gamma} [2\zeta_b \omega_b \dot{u}_b(t) + \omega_b^2 u_b(t) + \mu(\dot{u}_b) \text{sgn}(\dot{u}_b)] - 2\zeta_s \omega_s \frac{\gamma}{1-\gamma} \dot{u}(t) - \omega_s^2 \frac{\gamma}{1-\gamma} u(t) = -\ddot{u}_b(t)
$$

(4)

where the parameters $\omega_s$ and $\zeta_s$ denote the superstructure circular frequency and damping factor, whereas the parameters $\omega_b = \sqrt{k_b / (m_s + m_b)} = \sqrt{g / R}$ and $\zeta_b$ denote the fundamental circular frequency and damping factor for a rigid mass $(m_s + m_b)$ on a linear frictionless isolator of stiffness $k_b$ and viscous damping constant $c_b$.

### 3. Random Variables

Seismic reliability assessment of a steel building structure, according to the structural performance (SP) evaluation method [Bertero and Bertero, 2002], is based on the coupling between structural performance levels [Vision, 2000] and associated exceeding probabilities during its design life [Saito et al., 1998]. According to [Cornell and Krawinkler, 2000; Porter, 2003], the uncertainties related to the seismic input intensity are separated from those related to the characteristics of the record (record-to-record variability) by introducing a scale factor, i.e., an intensity measure (IM). The approach is based on calculating the probabilities of exceeding different limit state thresholds given different values of the intensity measure with the aim to define the fragility curves of the system. Afterward, the abovementioned fragility curves integrated with the seismic hazard curve, expressed in terms of the same IM, related to a reference site, lead to the mean annual rates of exceeding the limit states. Using a Poisson distribution, it is possible to transform the mean annual rates of exceeding the limit states into probabilities of exceedance in the time frame of interest (e.g., 50 years). The aim of this work consists of evaluating the seismic reliability of steel structural systems equipped with friction pendulum isolators (FPS) considering both the friction coefficient and earthquake characteristics as random variables.

In order to evaluate the record-to-record variability of the structural system response, several artificial earthquake excitations have been considered. In particular, each earthquake excitation can be modeled as a Gaussian stationary process with mean value equal to zero and two-sided power spectral density (PSD) function $S_f(\omega)$. It follows that the stochastic process $f(t)$ can be simulated by the following series as $N \to \infty$:

$$
f(t) = \sqrt{2} \sum_{n=0}^{N-1} A_n \cos(\omega_n t + \Phi_n)
$$

(5)

where $A_n = (2S_f(\omega_n)\Delta\omega)^{\frac{1}{2}}$, $\omega_n = n\Delta\omega$ for $n = 0 \ldots N-1$, $\Delta\omega = \omega_u / N$, having assumed $T = 2\pi / \Delta\omega = 31.25\text{s}$ (NTC08) and $\omega_u = 50\text{rad/s}$, $\Phi_0, \Phi_1, \Phi_2, \ldots, \Phi_{N-1}$ are independent random phase angles distributed uniformly over the interval $[0, 2\pi]$. A sample function $f^{\text{th}}(t)$ of the simulated stochastic process $f(t)$ can be obtained by replacing the sequence of random phase angles $\Phi_0, \Phi_1, \Phi_2, \ldots, \Phi_{N-1}$ with their respective $i$-th realizations $\Phi_{i,0}, \Phi_{i,1}, \Phi_{i,2}, \ldots, \Phi_{i,N-1}$, sampled through Monte Carlo simulations. In this study, 50 sequence of random phase angles are sampled through Monte Carlo simulations in order to generate 50 input accelerometric signals. The power spectral density function (PSD) of the embedded stationary process,
described by the widely-used Kanai and Tajimi [Kanai, 1957; Tajimi 1960] and modified according to Clough and Penzien [Clough and Penzien, 1993], applies:

\[
S_f(\omega) = \frac{\omega^4 + 4\xi^2\omega^2\omega^2}{(\omega^2 - \omega_0^2)^2 + 4\xi^2\omega^2\omega^2} S_0
\]

In the parametric study, with the aim to assume the uncertainty related to earthquake characteristics in terms of soil dynamics parameters corresponding to site class B (according to EC8 [CEN, 2004]), \(\omega_g\) and \(\xi_g\) are modeled as random variables uniformly distributed, respectively, in the intervals \([3\pi, 5\pi]\) (rad/sec) and \([40\%, 60\%]\) [Pinto et al., 2004; Talaslidis et al., 2004], and sampled through Monte Carlo simulations. In order to obtain non-stationary stochastic processes, a time-modulating function proposed by [Shinozuka and Sato, 1967] is adopted. As regards the friction coefficient, the experimental data [Constantinou et al., 1990; Constantinou et al., 2007; Mokha et al., 1990] on sheet type Teflon bearings, have pointed out that friction is a complex phenomenon, not complying with the Coulomb friction law and that several mechanisms contribute to its variability. In this study, a uniform density probability function (PDF), ranging from 3% to 12%, has been assumed to model the sliding friction at large velocity as random variable \(f_{\text{max}}\). For the generation of the sampled values of the friction coefficient \(f_{\text{max}}\), within the stratified sampling techniques the Latin Hypercube Sampling (LHS) method [Mckey et al., 1979] has been used. In particular, in the following parametric study, 15 sampled values of the random variable \(f_{\text{max}}\) are employed and a ratio \(f_{\text{max}}/f_{\text{min}}\) equal to 3, based on regression of experimental results, whereas the exponent \(\alpha\) of Eqn. (3) equal to 30 [Constantinou et al., 1990; Constantinou et al., 2007; Mokha et al., 1990] are assumed.

4. Incremental Dynamic Analyses (IDA) Results of Base-Isolated Steel Structures

Seismic reliability assessment of equivalent base-isolated systems is based on developing incremental dynamic analyses (IDA) [Vamvatsikos and Cornell, 2002]. In this study, the spectral displacement, \(S_D(T_b, \xi_b)\) is assumed as intensity measure [Luco and Cornell, 2007].

![Figure 1. IDA curves of the isolation level with \(\gamma=0.7\), for \(R=4m\)](image)
Figure 2. IDA curves of the superstructure with $\gamma=0.7$, for $R=4m$

The corresponding IM, hereinafter denoted as $S_b(T_b)$, in the IDA is considered ranging from 0m to 0.5m. The parameters $\xi_b$ and $\xi_s$ are assumed respectively equal to 0% and 2%, $\gamma$ is considered equal to 0.7, the radius $R$ of the FPS is assumed to be equal 4m, the fixed-base system period $T_s$ is considered equal to 0.3s, 0.9s and 1.5s, respectively. The response parameters $u_s$ and $u_b$ are adopted as the engineering demand parameters (EDPs), assumed to follow a lognormal distribution according to [Castaldo et al., 2015]. A lognormal distribution can be fitted to both response parameters (i.e., the extreme values of the EDPs), by estimating the sample lognormal mean, $\mu_{ln}(EDP)$, and the sample lognormal standard deviation, $\sigma_{ln}(EDP)$, through the maximum likelihood estimation method. Figs. 1-2 illustrate the IDA results regarding both the isolation level response $u_b$ and the superstructure response $u_s$. Each figure contains some surface plots, corresponding to different values of percentile (50th, 84th and 16th). Fig. 1 shows the IDA results regarding the isolation bearings response $u_b$. The lognormal mean and dispersion decrease for lower values of $T_s$ (high value of the isolation degree [Palazzo, 1991]). Fig. 2 shows the IDA results regarding the superstructure response $u_s$. The lognormal mean and dispersion slightly decrease for lower values of $T_s$. The trend of the IDA curves is consistent with the results in terms of the median and response percentiles obtained by [Barroso and Winterstein, 2002].

5. Seismic Fragility of Base-Isolated Steel Structures

With reference to performance levels of the steel superstructure, four discrete performance levels or limit states ($LS1, LS2, LS3, LS4$), corresponding respectively to “fully operational”, “operational”, “life safety” and “collapse prevention” are provided from Vision 2000 [Vision, 2000]. In accordance to [FEMA274, 1997; FEMA356, 2000; Castaldo et al., 2015], in Table 1, the $LS1$ and $LS2$ thresholds of fixed-base steel buildings as well as the corresponding failure probabilities in a design life of 50 years are reported. The performance limit states for the seismic fragility of base-isolated steel buildings, in accordance to NTC08 [NTC08, 2008] provisions, have been defined by limiting the response of the lateral-load-resisting superstructure system, IDI limits, to a fraction of the limits provided for designing comparable fixed-base buildings. In particular, with regard to NTC08 [NTC08, 2008]
provisions the limit response of the lateral-load-resisting superstructure system of a base-isolated building is equal to about two-thirds of the IDI limit of a comparable fixed-base building as also reported in Table 1. In the hypothesis of regular buildings, the Eqn. (7), according to NTC08, is considered as relationship between the fixed-base building period and its height \( H \), and is employed to estimate the height \( H \) as the integer multiple of the interstorey height assumed equal to \( h = 3 \) m and, so, the corresponding total number of floors \( N_f \):

\[
T_s = 0.085 H^{\frac{3}{4}}
\]  

(7)

Table 1. Limit state thresholds for the superstructure [FEMA274, 1997; FEMA356, 2000; Castaldo et al., 2015]

<table>
<thead>
<tr>
<th>Inter-story drift (ISD) index of fixed-base steel buildings</th>
<th>LS1 fully operational</th>
<th>LS2 operational</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.7%</td>
<td>1.5%</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Inter-story drift (ISD) index of base-isolated steel buildings [NTC08, 2008]</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0.47%</td>
<td>1.0%</td>
<td></td>
</tr>
</tbody>
</table>


\[
\gamma = \frac{\Gamma_1 M_{s1}}{\sum_{i=1}^{N_f} m_{s,i} + m_b}
\]  

(8)

For each \( T_s = \frac{2\pi}{\omega_s} \), assuming the building floor mass equal to \( m_{s,i} = 1000 \) kN \( s^2/\)m, for \( i=1...N_f \), it is possible to determine the floor stiffness and vector \( \Phi_1 \) containing the floor displacements of the first mode of the fixed-base structure normalized to the top floor displacement. The base mass \( m_b \) is assigned in order to respect the mass ratio \( \gamma \) [Kelly, 1997]:

where \( \Gamma_1 \) and \( M_{s1} \) represent respectively the participation factor and modal mass of the fundamental mode of the fixed-base structure. It follows that the maximum absolute interstorey drift of the 1st floor can be evaluated as \( u_{s,1,max} = \Gamma_1 \phi_1 u_{s,max} \), and this response parameter, divided by the interstorey height assumed equal to \( h = 3 \) m, corresponds to the overall maximum interstorey drift index experienced over the different stories that controls the performance of the superstructure.

Figure 3. Seismic fragility curves of the superstructure 1st floor with \( \gamma = 0.7 \), for \( R = 4 \) m
At each value of the intensity measure \( IM \), the probabilities \( p_f \) exceeding different limit states related to the superstructure have been numerically and fitted by a lognormal distribution. Fig. 3 shows the fragility curves regarding the superstructure 1st floor for three values of \( T_s \) (0.3s, 0.9s and 1.5s). The seismic fragility of the superstructure increases for higher values of \( T_s \) (lower values of the isolation degree).

With reference to the performance levels of the isolation system, several different values for the plan dimension of the isolator are considered. In Table 2, the limit state thresholds assumed for the seismic fragility of the FPS isolation level are reported. Fig. 4 shows the fragility curves regarding the isolation level for three values of \( T_s \) (0.3s, 0.9s and 1.5s) and the different values of the limit state thresholds varying in the range \( LS1-LS9 \). The fragility curves are plotted in logarithmic scale in the range between \( 10^0-10^{-6} \). The seismic fragility of the isolation level increases for higher values of \( T_s \) (lower values of the isolation degree).

### Table 2. Limit state thresholds for the isolation level

<table>
<thead>
<tr>
<th>Maximum relative displacement [m]</th>
<th>( LS1 )</th>
<th>( LS2 )</th>
<th>( LS3 )</th>
<th>( LS4 )</th>
<th>( LS5 )</th>
<th>( LS6 )</th>
<th>( LS7 )</th>
<th>( LS8 )</th>
<th>( LS9 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.1</td>
<td>0.15</td>
<td>0.2</td>
<td>0.25</td>
<td>0.3</td>
<td>0.35</td>
<td>0.4</td>
<td>0.45</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Figure 4. Seismic fragility curves of the isolation level with \( \gamma = 0.7 \), for \( R = 4m \)

### 6. Seismic Reliability of Base-Isolated Steel Structures

Considering a site near Sant’Angelo dei Lombardi (Italy) as the reference site, in Figure 5 the seismic hazard curve, expressed in terms of the same \( IM = S_{D}(T_b) \), related to the isolated period (\( R = 4m \)), is plotted according to NTC08. The abovementioned curve represents the average values of the annual rate \( \lambda \) of exceeding the \( IM = S_{D}(T_b) \) level. Integrating the fragility curves related to both the superstructure and isolation level with the seismic hazard curve and using a Poisson distribution, it is possible to evaluate the seismic reliability of the overall steel system in the time frame of interest (50 years). The seismic reliability of the steel superstructure increases for low values of \( T_s \) (high values of the isolation degree), as shown in Figure 6.
Figure 5. Seismic hazard curves related to a site near Sant’Angelo dei Lombardi (Italy)

Figure 6. Seismic reliability curves of the superstructure 1st floor for $\gamma=0.7$ and $R=4m$

Figure 7. Seismic fragility curves of the isolation level for $\gamma=0.7$ and $R=4m$
The seismic reliability of the isolation level decreases slightly depends on the values of $T_s$ (Figure 7). An exceeding probability of $p_f=1.5\cdot10^{-3}$ (related to the collapse limit state, reliability index $\beta=3$ in 50 years) is achieved through a radius in plan $r$ ranging from about 0.35 m to about 0.45 m depending on system properties. The results are consistent with those obtained by Castaldo et al. [2016a] considering r.c. superstructures in ordinary conditions [Etse et al., 2013; Etse et al., 2014; Etse et al., 2015; Etse et al., 2016; Mroginski et al., 2015; Ripani et al., 2014; Vrech et al., 2015; Ripani et al. 2016] and L’Aquila as reference site.

7. CONCLUSIONS

This paper deals with the seismic reliability of steel systems equipped with friction pendulum isolators (FPS) considering both the friction coefficient and earthquake characteristics as random variables. The uncertainty in the seismic inputs is taken into account by considering a set of artificial records related to class B site. Incremental dynamic analyses are developed to evaluate the probabilities exceeding different limit states related to both superstructure and isolation level for different structural properties.

In the final part, considering the seismic hazard curve related to a site near to Sant’Angelo dei Lombardi (Italy), according to NTC08, and regarding steel structures isolated by FP bearings with a design life of 50 years, reliability curves are derived.

8. REFERENCES


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EFFETTI DI SUOLO DI CATEGORIA B SULL’AFFIDABILITA’ SISMICA DI SISTEMI IN ACCIAIO ISOLATI ALLA BASE

Paolo Castaldo*, Guglielmo Amendola, Bruno Palazzo

Department of Civil Engineering, University of Salerno, Italy

SOMMARIO: Il presente lavoro valuta l’affidabilità sismica di sistemi in acciaio isolati alla base con dispositivi attritivi a pendolo scorrevole e soggetti a registrazioni accelerometriche artificiali corrispondenti a suolo di categoria B. La sovrastruttura è modellata con un comportamento elastico mentre gli isolatori sono descritti con un modello che tiene conto della variazione del coefficiente di attrito con la velocità. L’incertezza relativa sia all’azione sismica che al coefficiente di attrito è considerata assumendo opportune funzioni di densità di probabilità. Analisi dinamiche non lineari incrementali sono sviluppate al fine di definire le curve di fragilità relative sia alla sovrastruttura che livello di isolamento. Infine, considerando come sito di riferimento Sant’Angelo dei Lombardi (Italia), l’affidabilità sismica dell’intero sistema è valutata.

*Corresponding author: Paolo Castaldo, Department of Civil Engineering, University of Salerno, Italy. Email: pcastaldo@unisa.it