Seismic Reliability-Based Design of Elastic and Inelastic Structural Systems Equipped with Seismic Devices

CHAPTER 3

Chapter 3: Seismic Reliability-Based Design of Elastic and Inelastic Structural Systems Equipped with Seismic Devices

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Chapter Abstract
The scope of this chapter is to illustrate the seismic reliability-based design (SRBD) as an effective methodology to design both linear and non-linear structures isolated with seismic devices such as FPS (Friction Pendulum System) isolators. In order to model the isolated structures, an equivalent two-degree-of-freedom-model has been used taking into account the superstructure elastic or non-linear response whereas a widespread non-linear velocity-dependent model has been adopted for the friction FP device. By considering natural or artificial seismic records and assuming the seismic input characteristics together with the friction coefficient as the main aleatory uncertainties, incremental dynamic analyses are developed to derive the seismic fragility curves. The convolution integral permits to derive the seismic reliability curves by integrating the seismic hazard with the seismic fragility curves. Particularly, seismic reliability-based design expressions between the displacement ductility demand and the strength reduction factors for the inelastic superstructure as well as abacuses useful for the FPS device design are derived.

3.1 Introduction

Over the years, friction pendulum system (FPS) has experienced a great increase becoming a very effective seismic isolation technique (Mokha, Constantinou & Reinhorn, 1990; Constantinou, Whittaker, Kalpakidis, Fenz & Warn, 2007; Jangid, 2005). Structural reliability methods and reliability-based studies, including reliability-based optimization of base-isolated structures have been presented by (Chen, Liu, Peng & Li, 2007; Alhan & Gavin, 2005; Castaldo, Palazzo & Della Vecchia, 2015). The seismic performance of base-isolated buildings has been analyzed as a function of the FPS isolator characteristics by adopting a two-degree-of-freedom (2dof) model able to simulate the superstructure flexibility with a velocity-dependent model (Mokha, Constantinou & Reinhorn, 1990) for the FPS behavior. Particularly, nondimensional motion equations describing the seismic response of the 2dof system have been proposed by (Castaldo & Tubaldi, 2015) along with the description of the nondimensional statistics of the relevant response parameters as a function of both isolator and system properties. The abovementioned results can be useful to derive and assess fragility and seismic risk curves.

The present chapter assesses focuses on the seismic reliability of both elastic and inelastic superstructures isolated by FPS devices with the objectives to propose SRBD results. As for the FPS isolators, SRBD expressions are presented to design these devices considering either an elastic or inelastic response of the superstructure. As for yielding superstructures, designed in line with (Structural Engineering institute, 2010; European Committee for Standardization, Eurocode 8, 2004; Italian Building Codes, NTC2018, 2018; Japanese Ministry of Land, Infrastructure and Transport, 2000) and assuming a perfectly elastoplastic law or a hardening and softening post-yielding rule, relationships between the ductility-dependent strength reduction factors and the displacement ductility demand are proposed. All the proposed results are achieved for several inelastic and elastic building properties, increasing seismic intensities and selecting the value of 50 years as reference life and L'Aquila (Italy) as reference site (geographic coordinates: 42°38'49''N, 13°42'25''E). In addition, aleatory uncertainties are explicitly considered on both the friction coefficient and the seismic input, so as to treat that as the main aleatory uncertainties. As far as the uncertainty in the seismic inputs is concerned, either artificial records defined through Monte Carlo simulations in line to the power spectral density (PSD) method (Shinozuka & Deodatis, 1991) or natural records have been adopted. As for the sliding friction coefficient, the value at large velocity is modeled as random variable through a Gaussian probability density function (PDF) and the corresponding sampled values are defined in accordance to the Latin Hypercube Sampling (LHS) method (Celarec & Dolsek, 2013). Afterwards, incremental dynamic analyses (IDAs) (Vamvatsikos & Cornell, 2002) are developed to assess the superstructure and bearing response of the elastic and inelastic systems. These results are processed for computing seismic fragility curves of both the inelastic and elastic superstructure and the FP bearings, selecting specific values of the corresponding limit states (*LSs*). Successively, the fragility curves are integrated with the site-related seismic hazard ones to derive the seismic reliability curves of the investigated structures. Finally, SRBD regressions where the ductility-dependent strength reduction factors are related to the displacement ductility demand together with SRBD abacuses for the single-concave FP devices are proposed to ensure a reliable design of structures with FPS isolators.

3.2 Equations of motion for linear and nonlinear structures with FPS devices

The 2dof system, adopted by (Naeim & Kelly, 1999) and correctly modified in relation to the inelastic behaviour of both the superstructure and the device as depicted in Fig. 3.1, is used. Under the assumption of neglecting the vertical component (i.e., large radii of curvature *R* of the FP devices), the device displacement can be computed only in accordance to the horizontal component and so the isolator restoring force applies (Zayas, Low & Mahin, 1990):

$$f_b = \frac{W}{R} u_b + \mu_d W \,\mathrm{sgn}(\dot{u}_b) \qquad (3.1)$$

where $W = (m_b + m_s)g$ denotes the weight on the isolator, g the gravity constant, u_b the bearing displacement with respect to the ground, μ_d the sliding friction coefficient, sgn the signum function applied to the sliding velocity, $\dot{u}_b$. The non-linear relationship between the friction coefficient and the sliding velocity $\dot{u}_b$ applies (Mokha, Constantinou & Reinhorn, 1990; Constantinou, Whittaker, Kalpakidis, Fenz & Warn, 2007):

$$\mu_d = f_{\max} - (f_{\max} - f_{\min})\exp(-\alpha \dot{u}_b) \qquad (3.2)$$

where $f_{\min}$ and $f_{\max}$ denote, respectively, the friction coefficients at very low and high sliding velocities, α and the ratio $f_{\max}/f_{\min}$ are herein assumed as deterministic parameters equal, respectively, to 30 and 3 (Castaldo & Tubaldi, 2015; Castaldo, Palazzo & Della Vecchia, 2015; Castaldo, Amendola & Palazzo, 2017). In this way, the equations describing the non-linear response of the 2dof system with FP devices, to the seismic input $\ddot{u}_g(t)$, apply:

$$\begin{aligned} &(m_b + m_s)\ddot{u}_b + m_s\ddot{u}_s + c_b\dot{u}_b + \frac{W}{R}u_b + \mu_d W \operatorname{sgn}\dot{u}_b = -(m_b + m_s)\ddot{u}_g \\ &m_s\ddot{u}_b + m_s\ddot{u}_s + c_s\dot{u}_s + f_s(u_s) = -m_s\ddot{u}_g \end{aligned} \qquad (3.3a,b)$$

where m_b and m_s denote respectively the mass of the device and of the superstructure, c_b and c_s respectively the viscous damping factor of the isolator and of the superstructure. Dividing Eq.(3.3a) by $m_b + m_s$ as well as Eq.(3.3b) by m_s, and introducing the mass ratio $\gamma = m_s/(m_s + m_b)$ (Naeim & Kelly, 1999), the structural $\omega_s = \sqrt{k_s/m_s}$ and isolation $\omega_b = \sqrt{k_b/(m_s + m_b)} = \sqrt{g/R}$ circular frequency, the structural $\xi_s = c_s/2m_s\omega_s$ and isolation $\xi_b = c_b/2(m_b + m_s)\omega_b$ inherent damping ratio, the equations in non-dimensional form derive:

$$\begin{aligned} &\ddot{u}_b + \gamma\ddot{u}_s + 2\xi_b\omega_b\dot{u}_b + \frac{g}{R}u_b + \mu_d g \operatorname{sgn}\dot{u}_b = -\ddot{u}_g \\ &\ddot{u}_b + \ddot{u}_s + 2\xi_s\omega_s\dot{u}_s + a_s(u_s) = -\ddot{u}_g \end{aligned} \qquad (3.4a,b)$$

where $a_s(u_s) = f_s(u_s)/m_s$ is the non-dimensional superstructure force which is a function, respectively, of the stiffness k_s in the elastic response and of the yielding condition in the plastic one. Specifically, the superstructure responses in elastic phase if Eq.(3.5) is satisfied and Eq.(3.6) is useful to assess the corresponding restoring force.

$$\left|u_{s,i} - u_{0,i-1}\right| < y(u_{s,i}) \qquad (3.5)$$

$$f_{s,i}(u_{s,i}) = k_s(u_{s,i} - u_{0,i-1}) \qquad (3.6)$$

In equations (3.5) and (3.6), $f_{s,i}$ represents the superstructure restoring force at time instant i, $u_{s,i}$ the superstructure displacement with respect to the base at time instant i, $u_{0,i-1}$ the peak plastic excursion at time instant ($i-1$), k_s the superstructure elastic stiffness. The function $y(u_{s,i})$ represents the yielding condition which is not univocally determined due to the translation of the elastic domain, so the yielding limits depend on the displacement direction. In detail, the yielding condition (Hong & Liu, 1999) is a function of the yield displacement u_y, being f_y the yield force, and of the hardening or softening post-yield divided by the elastic stiffness (Hatzigeorgiou, Papagiannopoulos & Beskos, 2011). This latter ratio is denoted as *H* or *S* and expressed as follows:

$$H = S = k_y / k_s \tag{3.7}$$

The term (*H* or *S*) distinguishes the hardening to the softening response. The superstructure responses inelastically if Eq.(3.8) is respected and the restoring force applies (Eq.(3.9)):

$$\left| u_{s,i} - u_{0,i-1} \right| \geq y(u_{s,i}) \tag{3.8}$$

$$f_{s,i}\left(u_s\right) = k_s(u_{s,i} - y(u_{s,i}))\,\mathrm{sgn}(u_{s,i} - u_{0,i-1}) \tag{3.9}$$

From Eq.(3.4a), it derives that the isolation period depends only on the curvature radius of the FP isolators. In addition, the isolation period $T_b = 2\pi / \omega_b$ divided by the structural one $T_s = 2\pi / \omega_s$ provides the seismic isolation degree $I_d = T_b / T_s$ (Palazzo, 1991).

3.2.1 Inelastic properties of the non-linear superstructures

When it comes to the inelastic superstructure response (Fig. 3.1), identified by a perfectly elastoplastic or hardening/softening behavior representative of the global response of multi-story building frames (Fanaiea & Asfar, 2014; Abdollahzadeh & Banihashemi, 2013), the strength reduction factor or behavior factor, q, associated only to the ductility-dependent component (Fanaiea & Asfar, 2014; Abdollahzadeh & Banihashemi, 2013), is defined as:

$$q = \frac{f_{s,el}}{f_y} = \frac{u_{s,el}}{u_y} \tag{3.10}$$

where $u_{s,el}$ and $f_{s,el}$ denote, respectively, the yield deformation and minimum yield strength necessary for the superstructure to behave elastically.

The displacement ductility, μ, of the superstructure applies:

$$\mu = \frac{u_{\mathrm{s,max}}}{u_y} \tag{3.11}$$

where $u_{\mathrm{s,max}} = \left|u_s\left(t\right)\right|_{\mathrm{max}}$ means the peak response during the inelastic behavior.

Note that referring to perfectly elastoplastic law, the abovementioned behaviour factor q is different from the one codified by (Structural Engineering institute, 2010; European Committee for Standardization, Eurocode 8, 2004; Italian Building Codes, NTC2018, 2018; Japanese Ministry of Land, Infrastructure and Transport, 2000; FEMA P695, 2009) because the overstrength capacities are not taken into account explicitly. Contrarily, as for a hardening response, the behaviour factor is in accordance with the code provisions. In addition, as for softening systems due to the $P-\Delta$ effects, q is also consistent with the codes for the absence of the overstrength capacities.

3.3 Aleatory uncertainties in the seismic reliability evaluation

Seismic reliability definition of a structure, in accordance with the structural performance (SP) method (Collins & Stojadinovic, 2000; Bertero & Bertero, 2002), is developed by coupling SP levels (SEAOC Vision 2000 Committee, 1995) and related exceeding probabilities within its reference life (CEN-European Committee for Standardization, Eurocode, 2006; Saito, Kanda & Kani, 1998). In accordance with the PEER-like modular approach (Cornell & Krawinkler, 2000) and performance-based earthquake engineering (PBEE) approach (Aslani & Miranda, 2005), let us introduce an intensity measure (*IM*) to separate the uncertainties associated to the seismic input intensity from the ones related to the record characteristics. Particularly, the randomness on the seismic intensity can be represented by a hazard curve specific for the site, whereas the record characteristics uncertainty for a specific intensity level can be represented through a group of ground motions selected with different characteristics (e.g., duration and frequency content) and scaled to the same *IM* value. It is important to put in evidence that, as suggested by (Shome, Cornell, Bazzurro & Carballo, 1998; Luco & Cornell, 2007; Pinto, Giannini & Franchin, 2003), the *IM*'s choice should respect the hazard computability and criteria of both efficiency and sufficiency. In this study, the *IM* coincides with the spectral displacement, $S_D\left(\xi_b, T_b\right)$, at the period of the isolated system, $T_b = 2\pi / \omega_b$ and for the damping ratio ξ_b taken equal to zero (Castaldo & Tubaldi, 2015; Ryan & Chopra, 2004). Either artificial records, achieved in line to the PSD method (Shinozuka & Deodatis, 1991; Castaldo, Palazzo & Amendola, 2017) or natural records (Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018) are considered to consider the record-to-record variability in the responses.

As for the aleatory randomness on the sliding friction coefficient at high velocity of the FPS isolators (Mokha, Constantinou & Reinhorn, 1990; Constantinou, Whittaker, Kalpakidis, Fenz & Warn, 2007), a specific Gaussian PDF is utilized and, successively, sampled through the LHS method. An in-depth discussion on the PDF and sampled values may be acknowledged in (Castaldo, Amendola & Palazzo, 2017; Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018).

3.4 Incremental dynamic analyses (IDAs) results and curves

Since the main goal of the present study concerns developing IDAs (Vamvatsikos & Cornell, 2002), the system responses for increasing *IM* levels and for different superstructure and isolator properties have been computed, in accordance with Eq. (3.4). These deterministic values are also combined with the sampled ones of the friction coefficient (Castaldo, Amendola & Palazzo, 2017; Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018) and with the artificial or natural ground motions scaled to the increasing levels of the $IM = S_D(T_b)$. In detail, the deterministic parameters (Castaldo, Palazzo & Amendola, 2017; Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018) are: the seismic isolation degree I_d, the isolation period of vibration T_b, the mass ratio γ and the (ductility-dependent) strength reduction factor q varying from 1.1 to 2 (Structural Engineering institute, 2010; European Committee for Standardization, Eurocode 8, 2004; Italian Building Codes, NTC2018, 2018; Japanese Ministry of Land, Infrastructure and Transport, 2000). In addition, different cases of post-yield stiffness are considered (both hardening and softening behaviours): 0.03 and 0.06 for H and for S (Hatzigeorgiou, Papagiannopoulos & Beskos, 2011). It is important to highlight that the yielding characteristics of the inelastic systems have been designed in line with (Structural Engineering institute, 2010; European Committee for Standardization, Eurocode 8, 2004; Italian Building Codes, NTC2018, 2018; Japanese Ministry of Land, Infrastructure and Transport, 2000). Particularly, scaling the records to the $S_D(T_b)$ value specific for the life safety limit state and L'Aquila site, the average yield displacement $u_{y,\text{average}}$ and strength $f_{y,\text{average}}$ of the superstructures have been calculated in Matlab-Simulink (Math Works Inc., 1997) for each value of q. More details may be found in (Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018).

The different structural systems, designed as described above, are then subjected to the seismic records, scaled to several intensity levels, consistent with the IDAs procedure. Moreover, different friction coefficient values have been sampled and the simulations have been performed in Matlab-Simulink (Math Works Inc., 1997) to solve Eq. (3.4). Observe that we have not adopted any limits on the response parameters in order to numerically calculate the statistics. The results of the IDAs are computed as displacement relative to the base or ductility demand μ, coherently with the superstructure behaviour (e.g., linear or nonlinear), and peak displacement of the bearings with respect to the ground $u_{b,\max} = \left|u_b(t)\right|_{\max}$. These response parameters $u_{s,\max}$, μ and $u_{b,\max}$, represent the engineering demand parameters (EDPs) for the investigated systems and the corresponding peak values have been processed through lognormal PDFs (Castaldo & Tubaldi, 2015; Castaldo, Palazzo & Della Vecchia, 2015; Castaldo, Amendola & Palazzo, 2017; Cornell & Krawinkler, 2000; Ryan & Chopra, 2004) by computing the statistics by means of the maximum likelihood estimation technique (MLET), for each parameter combination and *IM* level. Therefore, it has also been suitable to calculate the 50th, 84th and 16th percentile for each lognormal PDF (Castaldo & Tubaldi, 2015). Observe that, for softening structural systems, the maximum available ductility capacity coincides with the numerical failure condition that occurs if the softening strength is totally annulled.

Fig. 3.2 shows the IDA curves in relation to the EDP $u_{b,\max}$ for elastic superstructure (Fig. 3.2a) perfectly elastoplastic (Fig. 3.2b) system and both hardening (Fig. 3.2c) and softening superstructure (Fig. 3.2d) in order to outline the main differences among the abovementioned systems and together with the influence of both hardening and softening post-yield stiffness. Particularly, when an inelastic (hardening or softening) system is considered, higher response values are in general required to the FP bearings differently from the elastic or perfectly elastoplastic case. (Fig. 3.2a) illustrates the IDA results for the FPS response $u_{b,\max}$ considering an isolation period $T_b \cong 3s$ with the elastic superstructure period ranging from $T_s = 0.3s$ to $T_s = 1.5s$ whereby the isolation degree I_d is considered ranging between 2 and

10. The lognormal mean values of u_b, slightly increases for decreasing T_s. As for the inelastic system, an isolation degree $I_d = 2$ and an isolation period of vibration $T_b = 3s$ have been assumed in (Fig. 3.2b,c,d). As is evidenced in (Fig. 3.2), the increase of q has a little influence on the isolator response showing a slight decrease of $u_{b,\max}$. Finally, (Fig. 3.2b) shows how γ affects the statistics: the isolation displacement goes down for increasing γ. Note that (Fig. 3.2,c,d) are considered with $\gamma = 0.6$; a value of $\gamma = 0.7$ is assumed for the elastic superstructure in (Fig. 3.2a).

Fig. 3.3 depicts the IDA curves for the superstructure response ($T_b \cong 3s$, $\gamma = 0.7$) in terms of u_s valid for elastic superstructure (Fig. 3.3a) and of the EDP μ (Fig. 3.3b,c,d). As depicted in (Fig. 3.3b,c,d), the statistics of the EDP μ are highly influenced by q since an its increase leads to a noticeable displacement ductility demand μ, particularly for the softening behaviour (Fig. 3.3d). Fig. 3.3b puts in evidence an increase of the structural damage as the mass ratio increases. The parameter H (Fig. 3.3c) allows to achieve a strong reduction of the statistics of the inelastic superstructure response because the displacement ductility demand is reduced for all structural parameter combinations, differently from the perfectly elastoplastic systems. Note that an isolation degree $I_d = 2$ and an isolation period of vibration $T_b = 3s$ have been assumed in (Fig. 3.3b,c) whereas $I_d = 8$ and $T_b = 3s$ for the softening behaviour (Fig. 3.3d). In Fig. 3.3c,d the mass ratio is $\gamma = 0.6$. Regarding the elastic system (Fig. 3.3a), the lognormal mean decreases for lower values of T_s .

An in-depth description of the IDAs results is presented in (Castaldo, Amendola & Palazzo, 2017; Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018).

3.5 Seismic fragility of elastic and inelastic base-isolated structures with FPS bearings

In the present section, it is described the computation of the seismic fragility representative of the probabilities P_f exceeding different *LS* thresholds at each level of the *IM,* related to both the FPS bearings and the elastic as well as inelastic superstructure. The *LS* thresholds are herein expressed as radius in plan, *r [m]*, for the FPS isolators, while, referring to performance of elastic superstructure, four *LS*s (i.e., *LS1,LS2,LS3,LS4*), corresponding respectively to "fully operational", "operational", "life safety" and "collapse prevention" are considered (SEAOC Vision 2000, 1995). Within the displacement-based seismic design, each *LS* is expressed as IDI, Interstory Drift Index, defined as a fraction of the limits provided for designing comparable fixed-base buildings (Bertero & Bertero, 2000; FEMA 274, 1997). In Table 3.1, the *LS1* and *LS2* thresholds useful to assess the seismic fragility of the elastic superstructures together with the corresponding failure probabilities in the period of 50 years are reported.

As for the inelastic superstructure, available displacement ductility μ *[-]* is considered. In Tables 3.2 and 3.3 are listed the different *LS* thresholds in relation to both *r* and μ. Besides this, Tables 3.2 and 3.3 report the related reference failure probabilities in timeframe of 50 years (Bertero & Bertero, 2000; Aoki, Ohashi, Fujitani et al., 2000; Castaldo, Palazzo & Della Vecchia, 2015; Bertero & Bertero, 2000; European Committee for Standardization, Eurocode 8, 2004): this latter corresponds to the collapse *LS* for the FP isolators, and to the life safety *LS* for the superstructure (also in line with the design).

The exceedance probabilities P_f, at each *IM* level and for each parameter combination, are numerically calculated using complementary cumulative distribution functions (CCDF) and then fitted through lognormal PDFs (Castaldo, Amendola & Palazzo, 2017). Particularly, for softening behaviour, within the computation of the exceeding probabilities P_f, the numerical results deriving from both the collapse and not-collapse cases have been accounted for through the total probability theorem (Bazzurro, Cornell, Shome & Carballo, 1998), as follows:

$$P_{SL}(IM = im) = (1 - F_{EDP|IM=im}(LS_{EDP})) \cdot \frac{N_{not-collapse}}{N} + 1 \cdot \left(1 - \frac{N_{not-collapse}}{N}\right) \qquad (3.12)$$

where N denotes the total number of numerical analyses for the specific structural system at the specific IM level, and $N_{not-collapse}$ represents the number of the successfully performed simulations (i.e., without any collapse).

The fragility curves are illustrated in (Fig.s 3.4 and 3.5) with regard respectively to the isolators and the superstructure. In each plot, different curves in relation to the different values of γ, T_s and q are represented. Furthermore, only some results associated to few LS thresholds are reported because of space constraints, more details are available in (Castaldo, Amendola & Palazzo, 2017; Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018). Generally speaking, the seismic fragility of both the superstructure and FPS devices increases for decreasing LS thresholds and reaches peak values when the inelastic system presents a post-yield softening behavior.

Fig. 3.4 depicts the fragility curves of the FPS for the elastic system (Fig. 3.4a), the perfectly elastoplastic behaviour (Fig. 3.4b) along with hardening and softening systems, respectively (Fig. 3.4c,d). For the inelastic systems, the figures are related to $T_b = 3s$ and I_d=8. Fig. 3.4a illustrates that for high values of R (i.e., $T_b \cong 3s$), increasing T_s values lead to higher failure probabilities. This is due to both the sampled friction coefficient values and higher displacement demand. Fig. 3.4b shows that higher values of both γ and q cause a little decrease of the exceeding probabilities, in stark contrast, Fig. 3.4d depicts that high q correspond to increasing exceeding probabilities.

Fig. 3.5 depicts the fragility curves for the superstructure. Similarly to the previous results, the seismic fragility gets higher for decreasing *LS* thresholds. For the inelastic system ($T_b = 3s$ and I_d=8), higher values of γ and of q cause a high increase of the exceeding probabilities. For the inelastic limit states (μ>1), it is worthy highlighting how the superstructure seismic fragility decreases for higher T_b with a fixed value of the isolation degree since T_s and the corresponding yielding displacement increase. Differently, the superstructure seismic fragility increases for higher seismic isolation degree I_d with a fixed value of T_b since T_s and the corresponding yielding displacement decrease causing resonance effects. This dynamic amplification is reduced if the systems presents a post-yield hardening behavior (Fig. 3.5c), whereas is present in the case of the post-yield softening behavior (Fig. 3.5d). More details are available in (Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018). In relation to the elastic superstructure ($T_b \cong 3s$), the failure probabilities increase as higher values of T_s are considered because of the superstructure flexibility.

3.6 Seismic Reliability-Based-Design (SRBD) of base-isolated structures with FP devices

After the seismic fragility is assessed, the abovementioned CCDFs can be integrated with the site-related seismic hazard curves, defined in accordance to the same specific *IM*, $S_D(T_b)$, to compute the mean annual rates λ_{LS} (Eq.(3.13)) exceeding each *LS* threshold in relation to both the FP isolator and superstructure within each parameter combination.

$$\lambda_{LS}(ls) = \int_0^\infty G_{LS|IM}(ls|im)\left|\frac{d\lambda_{IM}(im)}{d(im)}\right| d(im) \quad (3.13)$$

Afterwards, by using homogeneous Poisson distribution, λ_{LS} can be used to compute the exceedance probabilities (Eq.(3.14)) within the reference life of 50 years and, so, the seismic reliability in 50 years of the all investigated structures.

$$P_f\left(50years\right) = 1 - e^{-\lambda_{LS}\cdot 50years} \qquad (3.14)$$

3.6.1 Seismic Reliability-Based Design of the FPS isolators

The seismic reliability computation on the FPS devices leads to the definition of the SRBD abacuses useful to select the corresponding radius in plan r. As for the elastic superstructure, Fig. 3.6 puts in evidence that the seismic reliability of the FPS isolators increases as R decreases, as demonstrated in (Castaldo, Amendola & Palazzo, 2017). Regarding the inelastic superstructure, as discussed and commented in (Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018), the seismic reliability of the FPS devices increases as T_b and I_d decrease and slightly depends on γ . The increase of q, specifically for low isolation degrees along with low isolation period T_b and for a perfectly elastoplastic behaviour, leads a reduction of the seismic reliability as illustrated in Fig. 3.7a, whereas, for high isolation degrees or high isolation periods (Fig. 3.7b), as q decreases the seismic reliability slightly decreases; Fig. 3.7c,d show the results in terms of seismic reliability respectively for hardening and softening post-yield stiffness, selecting an isolation degree $I_d = 2$, $T_b = 3$, for H=0.03 and S=0.03. What is worth outlining is that, differently from what observed for the perfectly elastoplastic case, the increase of q generally reduces the seismic reliability, independently on the isolation degree value, and its influence for high T_b with hardening superstructure is lower. In stark contrast, for softening superstructures, the parameter S strongly affects the displacement demand to the devices. Indeed, the respect of the (collapse) exceeding probability (i.e., P_f=$1.5\cdot 10^{-3}$) is ensured by a value of r lower than 1m, in the case of high γ and I_d, only for very low values of q. These results, compared to the outcomes derived from the perfectly elastoplastic model, demonstrate that considering either a hardening or softening behaviour always provides higher isolation displacement demand. This difference is more marked when the superstructure presents a post-yield softening behaviour for high q.

3.7 Seismic Reliability-Based Design relationships for the superstructure

Fig. 3.8 shows the seismic reliability (SP) curves of the inelastic superstructure for the different properties and as a function of the displacement ductility. The seismic reliability rises for lower values of γ, I_d, q and for higher T_b. Higher values of H provide a seismic reliability increase, whereas higher values of S cause a strong drop of the seismic reliability. From these results, it is suitable to correlate the values of both μ and q which correspond to the life safety LS exceedance probability in 50 years (i.e., $2.2 \cdot 10^{-2}$) (Bertero & Bertero, 2000; Italian Building Codes, NTC2018, 2018). With this purpose, linear and multi-linear monovariate regressions, depicted in Fig. 3.9-Fig. 3.11, between μ and q have been achieved through the MLET (presenting a R-square value no lower than 0.93). The proposed SRBD regressions, suitable for regular base-isolated mdof (multi-degree-of-freedom) systems (Shome, Cornell, Bazzurro & Carballo, 1998; Fajfar, 2000), represent a reliable instruments to go over the displacement ductility demand for base-isolated systems in relation to the main properties (i.e., isolation degree, mass ratio and isolated period). The SRBD regressions also provide results consistent with the ones discussed by (Vassiliou, Tsiavos & Stojadinovic, 2013). Specifically, a slightly overestimated (ductility-dependent) strength reduction factor may produce a very large displacement ductility demand with a consequential collapse. It is also worthy picking out that, for perfectly elastoplastic systems (Fig. 3.9), lower values of I_d, for fixed T_b, produce a drop of the displacement ductility demand. It derives that if the isolation degree tends to a unitary value, the ductility demand meets the ‘equal displacement’ rule, $q = \mu$ (Newmark & Hall, 1973; Miranda & Bertero, 1994). Similarly, for fixed T_s, higher values of I_d are effective to reduce μ.

When it comes to hardening systems, as depicted in Fig. 3.10, the curves demonstrate the important role of the post-yield hardening stiffness H effective to make it possible a strong reduction of μ. In the assumption to assume a limit value for μ equal approximately to 5 (Paulay & Priestley, 1992), the upper limit for q equal to 1.5, as suggested by the international and national codes, could be a reliable value for high isolation degrees combined with low isolated periods and high mass ratios; whereas higher q values can be adopted to design systems with different structural properties. Note that, in accordance with the force-based approach to design regular base-isolated structures, these outcomes are coherent with the equivalent perfectly elastoplastic model because the behaviour factor is only influenced by the ductility-dependent term as previously commented.

When softening systems are examined, as illustrated in Fig. 3.11, the results demonstrate how the parameter S negatively influences the performance of base-isolated structures. For several structural properties, especially for high isolation degrees, slightly high strength reduction factors lead to a very large displacement ductility demand (Paulay & Priestley, 1992).

3.8 Conclusions

This chapter assesses the seismic performance of elastic and inelastic structures with FPS isolators with the so as to illustrate useful design recommendations for the FPS isolators as well as for the inelastic superstructure. With this purpose, the study describes the capabilities of the SRBD approach proposing SRBD relationships for the FPS isolators and for the isolated superstructures.

CHAPTER 3

References

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Figures

Figure 3.1: 2dof model of a perfectly elastoplastic, hardening or softening building isolated with FPS devices (modified from Castaldo, Amendola & Palazzo, 2017; Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018, with permissions).

Figure 3.2: IDA curves of the isolation level related to elastic (a), perfectly elastoplastic (b), nonlinear hardening (c) and softening (d) behaviour, with H=0.03, S=0.03, (modified from Castaldo, Amendola & Palazzo, 2017; Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018, with permissions).

Figure 3.3: IDA curves of the superstructure related to elastic (a), perfectly elastoplastic (b), nonlinear hardening (c) and softening (d) behaviour, with H=0.03, S=0.03 (modified from Castaldo, Amendola & Palazzo, 2017; Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018, with permissions).

Figure 3.4: Seismic fragility curves of the isolation level related to elastic (a), perfectly elastoplastic (b), nonlinear hardening (c) and softening (d) behaviour, with H=0.03, S=0.03 (modified from Castaldo, Amendola & Palazzo, 2017; Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018, with permissions).

Figure 3.5: Seismic fragility curves of the superstructure related to elastic (a), perfectly elastoplastic (b), nonlinear hardening (c) and softening (d) behaviour, with H=0.03, S=0.03 (modified from Castaldo, Amendola & Palazzo, 2017; Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018, with permissions).

Figure 3.6: SRBD curves of the isolation level in the case of elastic superstructure for R=1m (a), R=2m (b), R=3m(c) and R=4m (d) (modified from Castaldo, Amendola & Palazzo, 2017, with permission).

Figure 3.7: SRBD curves of the isolation level in the case of perfectly elastoplastic system with I_d=2, for T_b=3 s (a), perfectly elastoplastic system with I_d=2, for T_b=6 s (b) nonlinear hardening system with I_d=2, for T_b=3 s, H=0.03 (c) and softening behaviour, with I_d=2, for T_b=3 s, S=0.03 (d) (modified from Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018, with permissions).

Figure 3.8: SRBD curves of the superstructure in the case of perfectly elastoplastic system with I_d=2, for T_b=3 s (a), perfectly elastoplastic system with I_d=2, for T_b=6 s (b) nonlinear hardening system with I_d=2, for T_b=3 s, H=0.03 (c) and softening behaviour, with I_d=2, for T_b=3 s, S=0.03 (d) (modified from Castaldo, Palazzo & Ferrentino, 2017; Castaldo, Palazzo, Alfano & Palumbo, 2018, with permissions).

Figure 3.9: Relationships between the displacement ductility demand and (ductility-dependent) strength reduction factors in the case of perfectly elastoplastic system for I_d=2 (a),

I_d=4 (b), I_d=6 (c) and I_d=8 (d), (modified from Castaldo, Palazzo & Ferrentino, 2017, with permission).

Figure 3.10: Relationships between the displacement ductility demand and (ductility-dependent) strength reduction factors for I_d=2, H=0.03 (a), I_d=2, H=0.06 (b), I_d=8, H=0.03(c) and I_d=8, H=0.06 (d), (modified from Castaldo, Palazzo, Alfano & Palumbo, 2018, with permission).

Figure 3.11: Relationships between the displacement ductility demand and (ductility-dependent) strength reduction factors for I_d=2, S=0.03 (a), I_d=2, S=0.06 (b), I_d=8, S=0.03(c) and I_d=8, S=0.06 (d), (modified from Castaldo, Palazzo, Alfano & Palumbo, 2018, with permission).

Tables

Table 3.1: Limit state thresholds for the elastic superstructure (Bertero & Bertero, 2000; CEN-European Committee for standardization, Eurocode 0, 2006; FEMA 274, 1997).

	LS1 - fully operational	*LS2 - operational*
Inter-story drift (ISD) index	0.1%	0.2%
p_f*(50 years)*	$5.0\cdot10^{-1}$	$1.6\cdot10^{-1}$

Table 3.2: LS thresholds for the FP isolators with the corresponding reference exceedance probability.

	$LS_{b,1}$	$LS_{b,2}$	$LS_{b,3}$	$LS_{b,4}$	$LS_{b,5}$	$LS_{b,6}$	$LS_{b,7}$	$LS_{b,8}$	$LS_{b,9}$	$LS_{b,10}$
r [m]	0.05	0.1	0.15	0.2	0.25	0.3	0.35	0.4	0.45	0.5
$p_f(50\ years)=1.5\cdot10^{-3}$										

Table 3.3: LS thresholds for the inelastic superstructure with the corresponding reference exceedance probability.

	$LS_{\mu,1}$	$LS_{\mu,2}$	$LS_{\mu,3}$	$LS_{\mu,4}$	$LS_{\mu,5}$	$LS_{\mu,6}$	$LS_{\mu,7}$	$LS_{\mu,8}$	$LS_{\mu,9}$	$LS_{\mu,10}$
μ *[-]*	1	2	3	4	5	6	7	8	9	10
$p_f(50\ years)=2.2\cdot10^{-2}$										