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A PERFORMANCE - RELIABILITY BASED CRITERION FOR THE OPTIMUM DESIGN OF BRIDGES ISOLATORS

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ABSTRACT

This paper concerns a performance-reliability based criterion for the optimum design of bridges isolators. The meaning of "optimum design" concerns a structure designed in order to satisfy several performance requirements regarding the safety and the serviceability.

In order to carry out this procedure a stochastic approach is developed and a Gaussian, zero mean, filtered, non stationary stochastic process, is adopted in order to model the earthquake motion. The hysteretic Bouc-Wen model is employed in order to reproduce the non linear constitutive behaviour of isolators, whereas a linear law is assumed in order to represent the piers. Covariance response is attained by means of the approximate stochastic linearization method and the system reliability, required in order to make explicit the safety and the serviceability performance objectives, is evaluated in the hypothesis of independent crossings. Finally, the optimum design, performed in two different phases, is carried out in a parametric form.

KEYWORDS*:* Bridges Isolation, Stochastic Process, Hysteretic Model, Equivalent Linearization, Performance-Based Seismic Design

INTRODUCTION

Seismic isolation is a modern design approach intended for reducing destructive effects on structures caused by strong earthquakes. In bridges this method has the most important objective to protect low mass elements such as the piers and the foundations, from the high inertia forces transmitted from the heavy mass of the deck. The technique is realized in a simple way by replacing with the isolation systems, conventional devices adopted to accommodate thermal movements. In this way, seismic isolation acts by reducing the seismic forces and as result, the bridge piers survive even under strong earthquakes.

Nowadays for the seismic protection of bridges various devices are available: Rubber Bearings (*RB*), Lead Rubber Bearings (*LRB*), High Damping Rubber Bearings (*HDRB*) and others, such as the Friction Pendulum (*FP*). The most important features of these isolators is to supply in a single combined element, vertical support, lateral flexibility, restoring force and dissipation energy capacity. An extensive review of developments and recent applications has been provided by Symans et al. (1999).

The main objective of using the isolation technique is to reduce the seismic forces to or near the elastic limit capacity of structural elements so as to avoid or limit inelastic deformations and related damage phenomena. In bridges, by using seismic isolation, shear forces transmitted from the superstructure to the piers are reduced by shifting the natural period of the bridge away from the frequency range where the energy content of earthquakes is high (this scope is achieved when *RB*, *HDRB* or *LRB* are used). As result of employing the isolation strategy, the superstructure motion is decoupled from the piers motion during the earthquake, producing an effect of the reduction of inertia forces. At the same time, the seismic energy demand to the bridge is also reduced as a consequence of dissipation energy concentrated in isolators that are suitably designed for this purpose.

The *RB* isolator is realized by alternating rubber layers and steel plates vulcanised; the main characteristic of this device, whose constitutive law is approximately of a linear kind, consists of increasing the natural period of the protected system. However the damping, mainly of a viscous kind, is small. Higher dissipative capacities can be attained using the *HDRB.* At present, this isolator is considered one of the most attractive tool in the passive seismic protection. The main characteristic of this device is to supply high dissipative capacity; furthermore, one more significant attribute of *HDRB* is represented by a shear stiffness related to shear deformation level. The stiffness, in fact, is high for low deformations (10%-20%), protecting the structure by undesirable displacements caused by low intensity earthquakes and wind loads. For higher deformations the shear stiffness decreases and therefore the device acts by disconnecting the bridge deck motion from the pier motion. Finally, for deformations larger then 120%-150%, shear stiffness increases, but this deformation level is undesirable and design criteria could prevent it.

By placing a lead core in a *RB* isolator it can obtain the *LRB*, also named *New Zealand* isolator (*NZ*); this is characterized by horizontal flexibility, energy dissipation and vertical load capacity. In fact, the rubber provides the lateral flexibility to elongate the structure natural period whereas steel plates provide a high vertical support and confine the lead core. The latter produces an elevate dissipative capacity and at the same time it is able to support wind loads and small or moderate earthquakes. The force - displacement relationship of *HDRB* and *NZ* can be well represented by using the differential Bou-Wen model which is described by means of five analytical parameters governing the force-deformation cycle shape.

In seismic design of isolated structures it is important to decide with regard to the most appropriate mechanical characteristics of devices. For this intended aim, firstly the design variables for the problem must be defined: these are the isolator initial elastic stiffness, the isolator damping, the isolator elastic limit displacement and the post elastic stiffness. The main goal of this study is the evaluation of optimum mechanical parameters of isolators utilized in seismic protection of bridges. Their mechanical characteristics will be designed in order to guarantee safety and serviceability objectives and to maximize their performances in protecting bridges against destructive effects caused by strong earthquakes.

The study presented here refers in particular to isolators whose mechanical behaviour could be well described by a smoothed bilinear law, as the *HDRB* and the *LRB* ones (Figure 1) which can be modelled using the hysteretic non linear Bouc-Wen model (Bouc, 1967 and Wen, 1976)

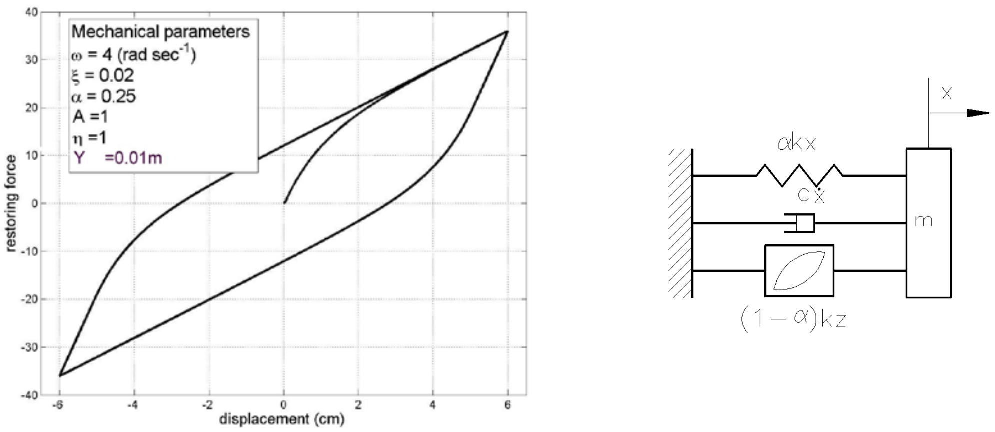


Fig. 1 Constitutive behaviour for a HDRB and its model

The proposed method, in agreement with current Technical Codes in the field, is based on the new seismic design philosophy *-performance based seismic design*. For this reason to two different excitation levels will be considered. The first one is a minor earthquake which can occur several times during the life time of the structure. For this load condition, the performance objective requires no damage and the bridge and the isolators only suffer small stresses, smaller than their elastic limit capacity. In this situation, both the bridge and the isolators will be modelled by means of a visco-elastic constitutive law.

The second excitation level is a severe earthquake which has a low occurrence probability during the life time of the structure. For bridges, which represent *essential facilities*, it is necessary that these remain operational and therefore even for a severe earthquake the damage should be limited or prevented (in relation to the bridge importance). Consequently, inelastic deformations of the piers should be avoided, whereas the hysteretic response of the isolators guarantees a ductile behaviour and an energy dissipation. For a severe earthquake, a non linear behaviour is adopted in order to reproduce the isolators constitutive law, whereas the pier is assumed linear as result of reduction of seismic forces produced by using the seismic isolation technique.

In order to carry out this procedure, a stochastic approach is implemented and a Gaussian, zero mean, filtered, non stationary stochastic process, is adopted in order to model the earthquake acting at the bridge foundation. The non linear hysteretic Bouc-Wen model is assumed to represent the constitutive behaviour of the isolation devices under the severe loading, whereas a linear law is always supposed to model the pier-deck structure. The non linear stochastic problem is solved by means of the stochastic linearization method in order to achieve the system covariance response. In order to make explicit the safety and the serviceability objectives, which are defined in terms of limit state crossing probability, it is necessary to define system reliability. Finally the optimum design, performed in two different phases, is carried out in a parametric form.

THE DYNAMIC MODEL OF THE ISOLATED BRIDGE

In this section the dynamic model of the isolated bridge, adopted in order to perform the main objective of the study, is discussed (Figure 2).

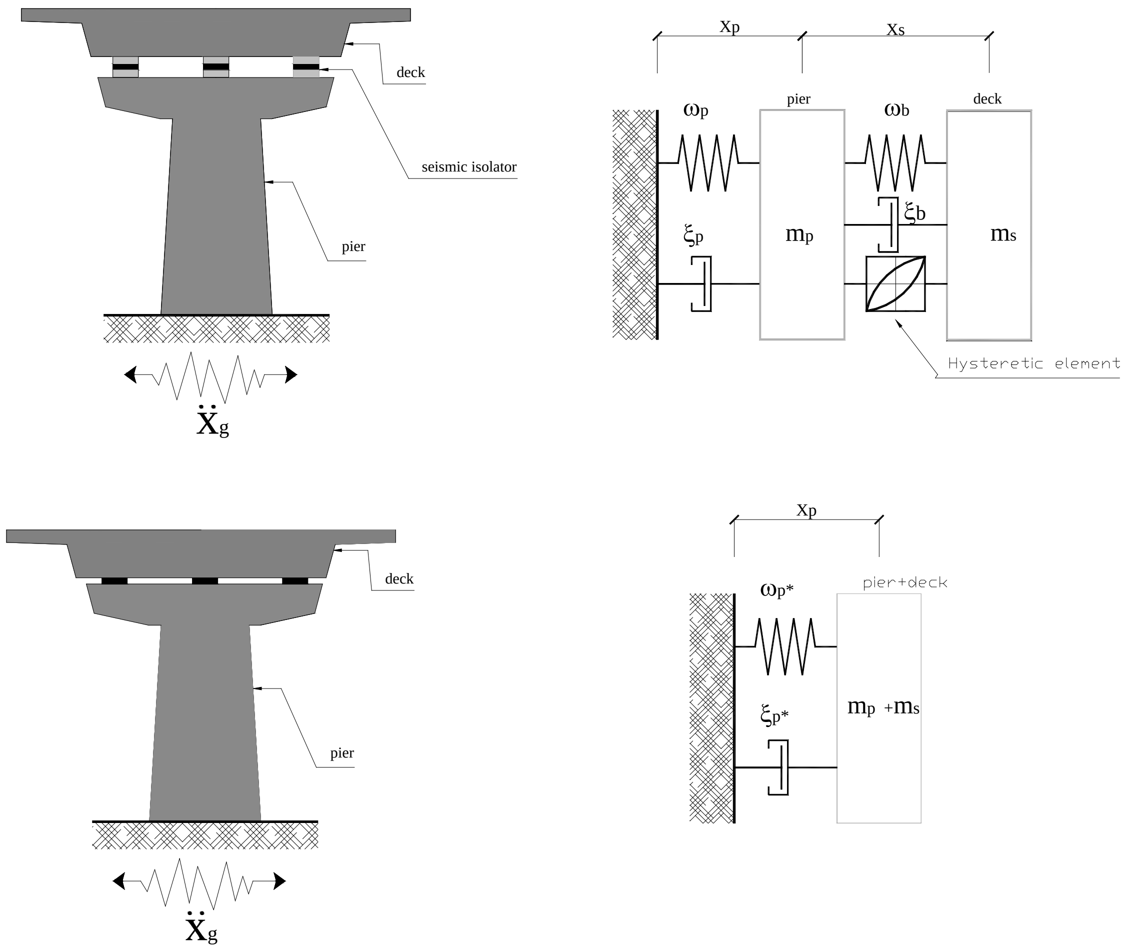


Fig. 2 Dynamic model of an isolated and a conventional bridge

Analysis of seismic behaviour of bridges is a very complicate problem involving not only structural modelling of pier-deck elements, but also several aspects, like soil-structure interaction (which considers the actual behaviour of soil foundation), multi –support seismic excitation (related to the difference of seismic input observed at different piers supports, especially in long bridges) and vertical oscillations (which are secondary occurrences observed in bridges under seismic excitations). All these phenomena, which are been extensively analysed (Monti, Pinto, 1998 – Takkar, Mahashwari, 1995) can significantly influence the bridge response both for conventional and isolated ones, but of course stochastic seismic analysis can considerably become complicated. Therefore, it is necessary to establish some assumptions in order to perform in a simple and reasonably accurate way the main objective of this study - a preliminary design of isolators mechanical characteristics on the basis of some performance requirements based on earthquake severity. Therefore, the analytical employed model should be simple and at the same time be able to capture the essential features of isolated bridges response. For this scope, a simplified 2 degrees of freedom system, below analysed in detail, is adopted in order to develop the stochastic analysis of the isolated bridges under the following assumptions:

- the deck superstructure is assumed to move as a rigid body;
- the pier is assumed to have a linear behaviour. This is a reasonable assumption, since the isolation technique attempts to reduce the earthquake response in such a way that the pier remains within the elastic range. As will be shown later, this is more than a simplified assumption because one of the performance objectives involved in the design proposed here will require that the pier should remain within its elastic limit;
- the pier is assumed to vibrate with its first mode. This assumption is quite rational for regular bridges;
- the effects of the incoherence of supports motion is ignored;
- the effects of soil – structure interaction is ignored and the vertical motion is not considered herein.

Since in the isolation of bridges, the devices are located between the piers and the deck it is possible, under the previous assumptions, to represent the structural system by means of a 2 degrees of freedom system, having masses m_p and m_s, that are respectively the mass of the pier and of the deck. The pier, as established in the hypothesis, is assumed linear as a consequence of adopting the isolation technique, and can be well represented in its first vibration mode by means of the first natural frequency $\omega_p = \sqrt{k_p / m_p}$ and the damping coefficient $\xi_p = c_p / 2\omega_p m_p$, where k_p and c_p are, respectively, the pier stiffness and viscous damping. The deck, whose behaviour is assumed rigid, instead, can be well modelled through a concentrate mass m_s positioned on the devices. The isolator elastic frequency ω_b and the damping coefficient ξ_b are defined respectively as:

$$\omega_b = \sqrt{\frac{k_b}{m_s}}\,, \qquad \xi_b = \frac{c_b}{2\omega_b m_s}$$

where k_b and c_b are the isolators elastic stiffness and the damping, governing the elastic phase behaviour.

The conventional bridge, whose response is necessary in order to evaluate the isolation performance, is represented by means of a concentrated mass m_p+m_s, a frequency ω^*_p and a damping coefficient ξ^*_p, related to the previous cited quantities through the relations:

$$\omega_p^* = \omega_p \sqrt{\frac{1}{1+\mu}} \qquad \xi_p^* = \xi_p \sqrt{\frac{1}{1+\mu}}$$

where $\mu = m_s/m_p$ is the mass ratio.

The non linear dissipative Bouc–Wen model (*BWM*) is adopted in order to represent the dynamic behaviour of isolators under the severe earthquake. It is a 1-dof non linear system having a mass m, whose non linear restoring force is expressed as:

$$Q(x,\dot{x},z) = c\,\dot{x} + \alpha\, k\, x + (1-\alpha) k\, z \tag{1}$$

where x is the non linear oscillator displacement, c and k are, respectively, the system damping and the elastic-initial stiffness, z is an internal variable governing the hysteretic behavior, and satisfying the differential equation:

$$\dot{z} = -\gamma |\dot{x}| |z|^{\eta-1} z - \beta\, \dot{x} |z|^{\eta} + A\dot{x} \tag{2}$$

The five parameters β, γ, η, α and A which appear in Equations (1) and (2) control the shape of the hysteretic force - deformation cycle and are well described by Cunha (1984). In detail, η is a natural number that controls the transition from the elastic to the post-elastic phase. When this parameter approaches infinity the constitutive law becomes bilinear. The parameter α is the ratio between the plastic phase stiffness k_f and the elastic initial one k_i (this particular result is true for $A=1$), whereas β controls the nature of the constitutive law (hardening or softening). Wong (1994) showed that β and γ determine the elastic limit displacement Y through the relation (for η=1 and A=1):

$$Y = \frac{1}{\beta + \gamma}$$

RESPONSE EVALUATION

In this section will be assessed the response of the isolated bridge subject to a seismic motion represented through a stochastic process. In detail, the time modulated Kanai-Tajimi process (1960) is adopted in order to represent the ground motion acting at the bridge foundation.

The non stationary time modulated Kanai-Tajimi process is obtained by multiplying the stationary Kanai-Tajimi process with a time modulating function $V(t)$:

$$\ddot{x}_g = \bar{\ddot{x}}_g V(t)$$

where $\bar{\ddot{x}}_g$ is the stationary Kanai-Tajimi process.

In this study the exponential modulation function (Figure 3) is adopted: $V(t) = \alpha_v t e^{-\beta_v t}$

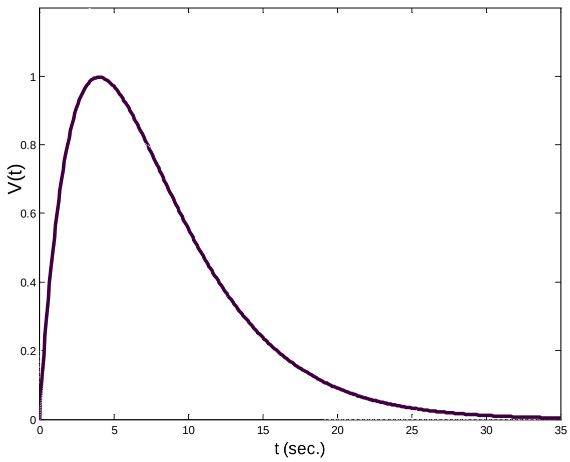


Fig. 3 Exponential modulation function $V(t)$

The motion equations of the isolated bridge subjected to a seismic motion represented by the time modulated Kanai-Tajimi process are:

$$\begin{cases} m_p\ddot{x}_p + c_p\dot{x}_p - c_b(\dot{x}_s - \dot{x}_p) + k_p x_p - \alpha_b k_b(x_s - x_p) - (1-\alpha_b)k_b z_b = -m_p\ddot{x}_g \\ m_s\ddot{x}_s + c_b(\dot{x}_s - \dot{x}_p) + \alpha_b k_b(x_s - x_p) + (1-\alpha_b)k_b z_b = -m_s\ddot{x}_g \\ \dot{z}_b = -\gamma_b\left|(\dot{x}_s - \dot{x}_p)\right|\left|z_b\right|^{\eta_b - 1} z_b - \beta_b(\dot{x}_s - \dot{x}_p)\left|z_b\right|^{\eta_b} + A_b(\dot{x}_s - \dot{x}_p) \\ \ddot{x}_g = \ddot{x}_f + V(t)w = -2\xi_g\omega_g\dot{x}_f - \omega_g^2 x_f \end{cases} \quad (3)$$

where x_p and x_s are, respectively, the displacements of the top of the pier and the of deck relative to the ground.

The motion equations have been written with the consideration that the isolator has a non linear behaviour: this is the more general hypothesis that can be particularized when the isolator exhibits a linear behaviour under low earthquakes.

The parameters β_b, γ_b, η_b, α_b and A_b, which rule the shape of the isolator hysteretic cycle, have been introduced. Furthermore, in Equation (3) the filter motion equation appears, where x_f is the response of the filter representing the ground, characterised by a frequency ω_g and a damping coefficient ξ_g; finally, w is the white noise excitation process at the bed rock and $V(t)$ is the modulation function.

The dynamic problem formulated above is non linear, for the reason that the seismic isolators have a non linear behaviour. In this study, the *stochastic linearization method* is adopted in order to perform the analysis. The fundamental idea of this approximate technique is that the equation describing the non linear system can be substituted by a linear one, equivalent in stochastic terms; moreover, the hypothesis of a Gaussian response process should be assumed. The approximate linearized form of the original non linear equation is then achieved by minimising in a stochastic way the difference between the non linear equation and the linearized one (Roberts and Spanos, 1990).

Then, by adopting the equivalent stochastic linearization method, the non linear equation governing the internal variable z_b is replaced with the next linear one, equivalent in stochastic meaning:

$$\dot{z}_b = -c_b^{\,e}\,(\dot{x}_s - \dot{x}_p) - k_b^{\,e}\, z_b \tag{4}$$

For $A_b = 1$ and $\eta_b = 1$, in the hypothesis of variables z_b and $\dot{u}_s = \dot{x}_s - \dot{x}_p$ jointly Gaussian, Atalik and Utku (1976) provided the equivalent coefficients $c_b^{\,e}$ and $k_b^{\,e}$:

$$\begin{cases} c_b^{\,e} = \sqrt{\dfrac{2}{\pi}}\left[\beta_b\,\sigma_{z_b} + \gamma_b\,\dfrac{E[\dot{u}_s\, z_b]}{\sigma_{\dot{u}_s}}\right] - A_b \\ k_b^{\,e} = \sqrt{\dfrac{2}{\pi}}\left[\gamma_b\,\sigma_{\dot{u}_s} + \beta_b\,\dfrac{E[\dot{u}_s z_b]}{\sigma_{z_b}}\right] \end{cases} \tag{5}$$

where σ_{z_b} and $\sigma_{\dot{u}_s}$ are, respectively, the standard deviations of variables z_b and $\dot{u}_s$ and $E[\dot{u}_s z_b]$ is their covariance.

In a matrix form the linearized motion Equation (4) is converted into:

$$\begin{cases} \mathbf{M}\,\ddot{\mathbf{x}} + \mathbf{C}\,\dot{\mathbf{x}} + \mathbf{K}\,\mathbf{x} + \mathbf{q}(1-\alpha_b)k_b z_b = -\mathbf{M}\,\mathbf{r}\,\ddot{x}_g \\ \dot{z}_b = -c_b^e \dot{u}_s - k_b^e z_b \\ \ddot{x}_g = \ddot{x}_f + wV(t) = -2\,\xi_g\,\omega_g\,\dot{x}_f - \omega_g^2 x_f \end{cases} \tag{6}$$

where the linearization coefficients $k_e^{\,b}$ and $c_e^{\,b}$ appear, and where:

$$\mathbf{M} = \begin{bmatrix} m_p & 0 \\ 0 & m_s \end{bmatrix} \quad \mathbf{C} = \begin{bmatrix} c_p + c_b & -c_b \\ -c_b & c_b \end{bmatrix} \quad \mathbf{K} = \begin{bmatrix} k_p + \alpha k_b & -\alpha k_b \\ -\alpha k_b & \alpha k_b \end{bmatrix} \quad \mathbf{x} = \begin{Bmatrix} x_p \\ x_s \end{Bmatrix} \quad \mathbf{q} = \begin{Bmatrix} -1 \\ 1 \end{Bmatrix} \quad \mathbf{r} = \begin{Bmatrix} 1 \\ 1 \end{Bmatrix}$$

By introducing the following co-ordinate change:

$$\mathbf{x} = \mathbf{T}\mathbf{u}$$

where $\mathbf{u} = \{u_p \quad u_s\}^T$ is the vector containing the pier displacement relative to the ground and the superstructure displacement relative to the pier, which corresponds with the isolator displacement, and **T** is the co-ordinate change matrix:

$$\mathbf{T} = \begin{bmatrix} 1 & 0 \\ 1 & 1 \end{bmatrix}$$

Equation (6) can be written as:

$$\begin{cases} \mathrm{M}^*\,\ddot{\mathbf{u}} + \mathrm{C}^*\,\dot{\mathbf{u}} + \mathrm{K}^*\mathbf{u} + \mathbf{q}(1-\alpha_b)k_b z_b = -\mathrm{M}^*\,\mathrm{r}^*\,\ddot{x}_g \\ \dot{z}_b = -c_b^e\,\dot{u}_s - k_b^e z_b \\ \ddot{x}_g = \ddot{x}_f + wV(t) = -2\,\xi_g\,\omega_g\,\dot{x}_f - \omega_g^2 x_f \end{cases} \tag{7}$$

where the new matrices and the new vectors in the new co-ordinates appear:

$$\mathbf{M}^* = \mathbf{MT} = \begin{bmatrix} m_p & 0 \\ m_s & m_s \end{bmatrix} \quad \mathbf{C}^* = \mathbf{CT} = \begin{bmatrix} c_p & -c_b \\ 0 & c_b \end{bmatrix} \quad \mathbf{K}^* = \mathbf{KT} = \begin{bmatrix} k_p & -\alpha k_b \\ 0 & \alpha k_b \end{bmatrix} \quad \mathbf{r}^* = \mathbf{r}\mathbf{T}^{-1} = \begin{Bmatrix} 1 \\ 0 \end{Bmatrix}$$

After the s*tate vector* **Y** is introduced:

$$\mathrm{Y} = \left\{ \mathrm{u} \quad x_f \quad z_b \quad \dot{\mathrm{u}} \quad \dot{x}_f \right\}^T$$

it is possible to write the *state equation* as:

$$\dot{\mathbf{Y}}(t) = \mathbf{A^e} \; \mathbf{Y}(t) + \mathbf{B}(t) \tag{8}$$

where $\mathbf{B}(t)$ has all elements equal to zero except $B_7(t) = -wV(t)$, and:

$\mathbf{A^e}$ is the *equivalent linearized state matrix* of order 7:

$$\mathbf{A}^e = \begin{bmatrix} \mathbf{0}_{2x2} & \mathbf{0}_{2x1} & \mathbf{0}_{2x1} & \mathbf{I}_{2x2} & \mathbf{0}_{2x1} \\ \mathbf{0}_{1x2} & 0 & 0 & \mathbf{0}_{1x2} & 1 \\ \mathbf{0}_{1x2} & 0 & -k_b^e & \mathbf{C}^e{}_{1x2} & 0 \\ -\mathbf{M}^{*-1}\,\mathbf{K}^* & \mathbf{r}^*\omega_g^2 & -\mathbf{M}^{*-1}\,\mathbf{q}(1-\alpha_b)k_b & -\mathbf{M}^{*-1}\,\mathbf{C}^* & \mathbf{r}^*2\xi_g\omega_g \\ \mathbf{0}_{1x2} & -\omega_g^2 & 0 & \mathbf{0}_{1x2} & -2\xi_g\omega_g \end{bmatrix} \tag{9}$$

In the matrix (9) the zero vectors and the zero matrices and the identity matrices appear.
Finally:

$$\mathbf{C}^e{}_{1x2} = \left\{ -c_b^e \quad 0 \right\}$$

Now, the stochastic response can be obtained by solving the differential equation of the covariance matrix $\mathbf{Q_{YY}}(t)$, whose time varying elements are the second order moments $E\lfloor Y_i\, Y_j \rfloor$ relative to the state vector **Y**:

$$\dot{\mathbf{Q}}_{\mathbf{YY}}(t) = \mathbf{A^e}\mathbf{Q_{YY}}(t) + \mathbf{Q_{YY}}(t)\mathbf{A^{e^T}} + \mathbf{G}(t) \tag{10}$$

$\mathbf{G}(t)$ is a matrix of order 7 having all elements equal zero except $G_{77}(t) = 2\pi S_0 V(t)^2$, where S_0 is the white noise intensity.

Figures 4 and 5 show the variance of the pier and the deck displacement, performed by using both the approximate linearization technique and the Monte Carlo simulation, developed by generating 1500 synthetic acceleration time histories. The following parameters for the stochastic ground shaking modelling and for the isolated bridge system are adopted: ω_g= 20rad/sec, ξ_g= 0.6 and S_0=0.0620m^2/sec^3, ω_b=6.28rad/sec, ω_p=12,56rad/sec, ξ_p=5%, ξ_b=10%, μ=4, Y=10cm, β=0.05 and γ=0.05, whereas α assumes the values 0.3, 0.6 and 0.9. The results show a relatively good agreement between the stochastic response of the linearized system and that of the original non linear one performed by means of simulation. However, in some conditions the linearized response is underestimated: this occurs especially in the isolator displacement when low values of the parameter α are adopted.

PROBABILISTIC PERFORMANCE - RELIABILITY OPTIMUM DESIGN OF BRIDGE ISOLATORS

The stochastic nature of natural hazards and accidental loads and the non deterministic character of material properties request to perform probabilistic methodologies in order to provide in a rational way the assessment of structural performance and safety.

In this paper a probabilistic performance-reliability based criterion for the optimum design of isolators used for seismic protection of bridges is proposed. The meaning of optimum design involves a structure designed in order to satisfy a set of requirements concerning performance objectives which are related to system *safety* and *serviceability***.** In fact, structural seismic design should be applied not only in order to guarantee the life safety and to prevent structural collapse, but also in order to control the level of damage and the behaviour of components and systems.

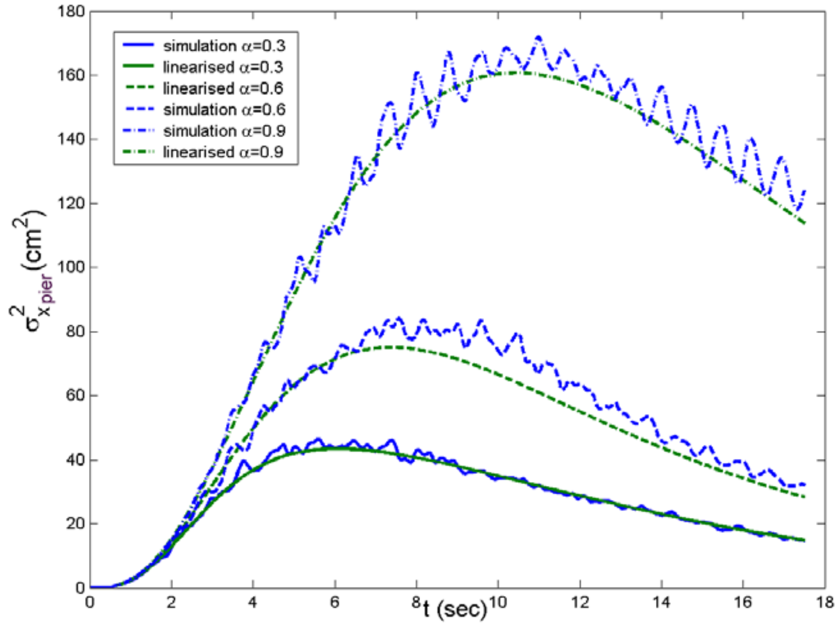


Fig. 4 Simulated and linearized pier variance response

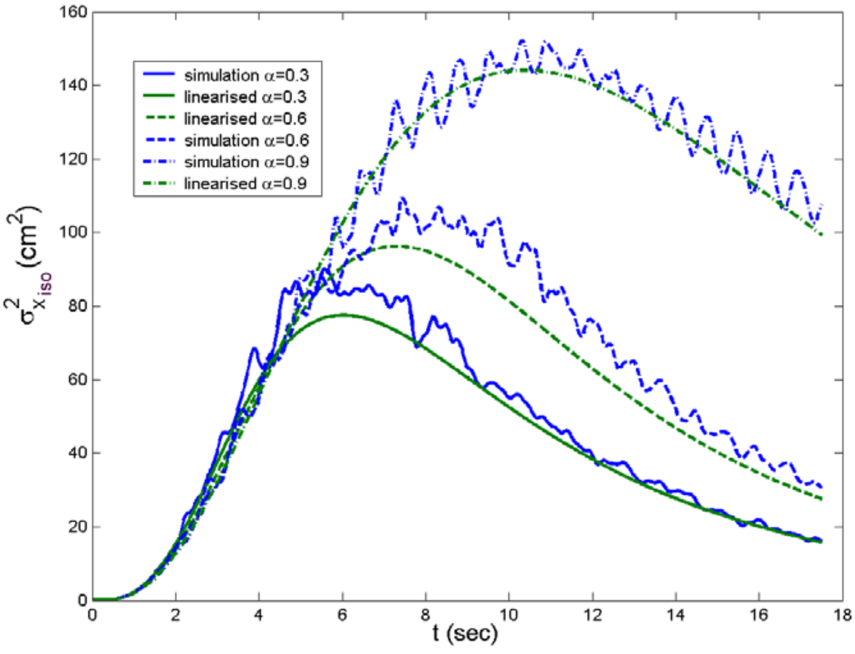


Fig. 5 Simulated and linearized isolator variance response.

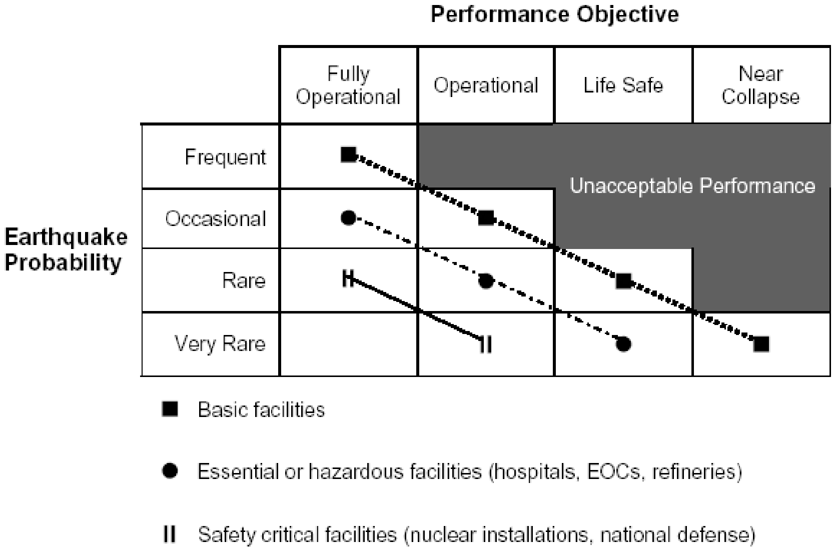
On the basis of these ideas a new design philosophy has been under development in recent years. This is the *performance-based design* (SEAOC Vision 2000, 1995) which can be defined as a design to reliably achieve targeted performance objectives. Performance-based design is a general design philosophy in which the design criteria are expressed in terms of achieving stated performance objectives when the structure is subjected to stated levels of seismic hazard. It is possible to summarize that this seismic design philosophy accomplishes the following objectives:

1. To prevent non structural damage in minor earthquake, which may occur frequently during the service life of the structure.
2. To prevent structural damage and minimize non structural damage during moderate earthquake ground shakings, which may occasionally occur.

3. To avoid collapse or serious damage during severe earthquake ground shakings, which may rarely occur.

Therefore the meaning of this design philosophy is firstly, the definition of the earthquake probability and secondly, the performance objectives. These are summarised in the performance matrix, reproduced below (table 1) (SEAOC Vision 2000, 1995) which considers four levels of earthquake hazard.

Table 1: Earthquake Probability and Performance Objective (SEAOC Vision 2000, 1995).



Also specific Seismic Codes for bridges developed around the world (TNZ -1995, JRA- 2002, AASHTO - 1996, ATC 32 -1996, EC8 - 1994) are based on *performance expectations*, which are related to different earthquake levels expressed by means of the occurrence probability.

Table 2 shows recommended *Seismic Performance Criteria* (ATC, 1996) for a two level design approach, which considers two service levels and three damage levels for bridge design, and these can be different for ordinary and important bridges.

Table 2: Recommended Seismic Performance Criteria (ATC 1996a).

Ground Motion at Site	Ordinary Bridges	Important Bridges
Functional-Evaluation Ground Motion	**Service Level**-Immediate **Damage Level**- Repairable Damage	**Service Level**-Immediate **Damage Level**- Minimal Damage
Safety-Evaluation Ground Motion	**Service Level**-Limited **Damage Level**- Significant Damage	**Service Level**-Immediate **Damage Level**- Repairable Damage

On the basis of this current philosophy used in seismic design practice, this section develops a method intended for carrying out the probabilistic optimum design criterion of isolation devices for seismic protection of bridges. The criterion is applied to two earthquake levels - the minor earthquake and the severe earthquake (very sever ground motion is not included in this analysis).

After the earthquake hazard levels are stated, the next step is the description of the required performance objectives that, as explained before, should be fixed on the basis of the earthquake severity. For each hazard level a limit state will be established, defined as a state where the structure attains an undesirable structural behaviour.

Commonly, isolated structures are designed in agreement with the following requirements: to withstand minor and moderate earthquakes without damage to structural elements, non structural components, and contents; to withstand severe earthquake ground motion without failure of the isolation system, without significant damage to structural elements (or without damage in relation to the structure importance), without extensive damage to non structural components, and without major disruption to facility function, and subsequently without loss of life.

In conformity with the requirements previously explained, the optimum design method is developed in the following way:

Minor earthquake

This is a frequent earthquake having a 50% in 50 years of probability of occurrence. For this, the following objectives are established:

- The displacement of the top of the pier must be smaller than the elastic limit displacement. The isolator must remain in the elastic range in order to avoid large relative displacements of the superstructure respect to the pier, as required for service loads, and to avoid plastic displacements that can reduce the isolator capacity under the severe earthquake.

The design variables in this phase are the initial elastic stiffness k_b, the isolator damping c_b and the isolator elastic limit displacement Y. More precisely, the first one is expressed by means of the frequency ratio $I = \omega_b / \omega_p$, that is the ratio between the elastic initial isolator frequency and pier frequency.

Concerning the damping coefficient ξ_b, previous studies (R. Greco et al., 2002) have verified that the optimum value ξ_{b_opt}, for the usual range ($5\% \le \xi_b \le 15\%$) which characterizes these isolators, always coincides with the highest value and, therefore, this parameter will not be designed but it will be recognised as a data problem.

Isolators performances are represented by means of two serviceability state limit probabilities: first, the probability that the displacement of the top of the pier crossings the pier elastic limit displacement X_{p_e}, secondly, the probability that the maximum isolator displacement crossings the elastic limit displacement Y.

The first requirement is expressed in terms of pier reliability, i.e. the probability that the displacement of the top of the pier X_p doesn't cross the threshold elastic level X_{p_e}:

$$P_S(t, X_{p\_e}) = P\left[\left|X_p(t)\right| \le X_{p\_e} \forall\, 0 \le t \le \tau\right] \ge \overline{P} \tag{11}$$

where τ is the earthquake duration and $\overline{P}$ is the reliability target fixed for this limit state. In this study the reliability is determined in the hypothesis of independent threshold crossings, with a Poisson distribution (Nigam, 1983):

$$P_S(t,\xi) = \exp\left\{-\int_0^t \alpha(t)dt\right\} = \exp\left\{-\int_0^t 2\nu_{\xi^+}(t)dt\right\} \tag{12}$$

where:

$$\nu_{\xi^+}(t) = \nu_{0^+}(t)\exp\left[-\frac{\xi^2}{2\sigma_x^2(t)}\right] \text{ and } \nu_{0^+}(t) = \frac{1}{2\pi}\frac{\sigma_{\dot{x}}(t)}{\sigma_x(t)} \tag{13}$$

and ξ is a generic threshold level.

The use of this approach for the reliability evaluation has some restrictions which are related both to the Poisson hypothesis and to the Gaussian joint probability distribution assumption for the displacement and the velocity processes, necessary in order to perform the stochastic linearization technique adopted here.

The supposition of independent barrier crossings of the random process $X(t)$ is quite adequate in the analysis developed here. Indeed it is well known that this hypothesis can be quite poor and excessively conservative when clumping effects occur in barrier crossings (for a more accurate analysis different and more complicated approaches can be used, as that proposed by Vanmarke (1972) based on the use of envelope process). Moreover, it has been verified that the Poisson hypothesis is a valid assumption for high threshold levels and so it could be used in the present analysis due to the fact that reliability levels investigated here are high (the minimum reliability target considered is $1\text{-}10^{-2}$) and therefore the barrier up crossing events can are effectively independent.

The second limitation in the reliability evaluation developed here is related to the actual non linear structural behaviour. In fact, the stochastic linearization technique, adopted here in order to supply the system covariance response is able to provide, with a good agreement, the mean and the variance of the original nonlinear process. However, it isn't able to give with the same accuracy information about the response probability distribution. Moreover, joint probability distribution function tails information, which is an essential element in the mean rate crossing evaluation of exact Rice formulation (1944) can be affected by severe mistakes that could induce incorrect reliability results.

In spite of this problem the comparison developed between the approximate linearized analysis and the Monte Carlo simulation shows that for rare crossing events (when the reliability is high) there is a good agreement between the two methodologies (Figure 6).

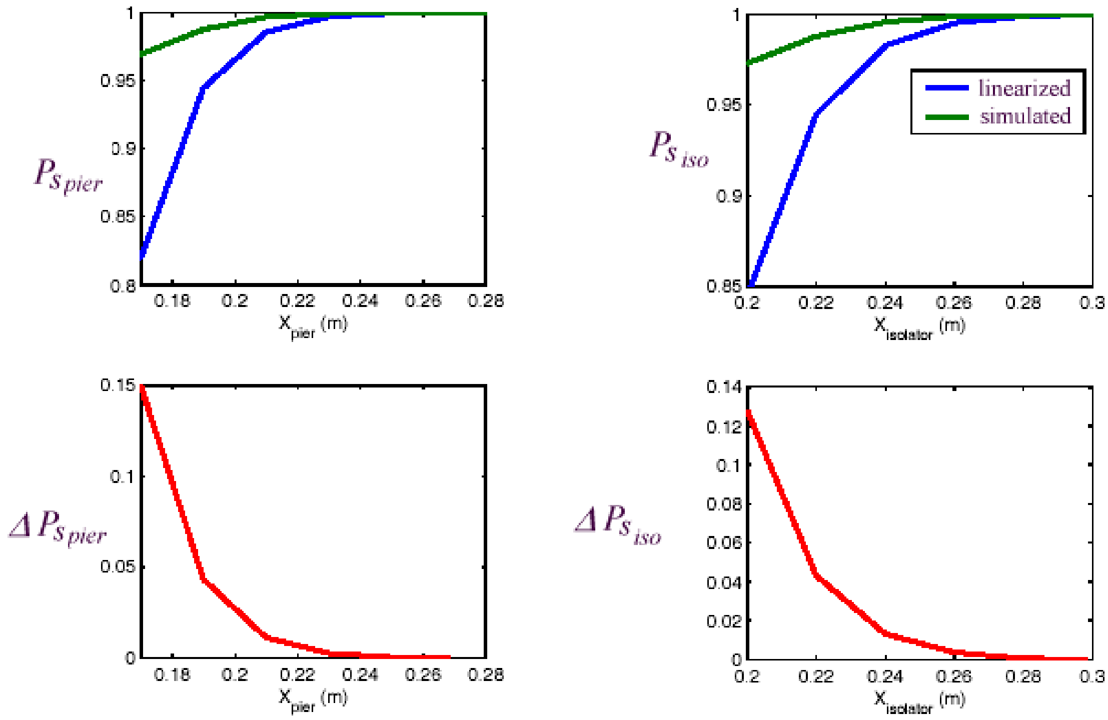


Fig. 6 Simulated and linearized pier and superstructure reliability, and corresponding error ΔP_s

In order to perform the optimum isolator design, relation (11) is replaced with its inverse formulation by representing in explicit form the maximum pier displacement $X_{p\_\max}$ that has a $\overline{P}$ probability not to be exceeded during the earthquake duration.

Hence, the relation (11) is replaced by:

$$X_{p\_\max}(t, \overline{P}) \leq X_{p\_e} \tag{14}$$

where $X_{p\_\max}(t, \overline{P})$ is the maximum pier displacement having a probability $\overline{P}$ not to be exceeded.

Similarly, for the isolator displacement it is required that:

$$X_{b\_\max}(t, \overline{P}) \leq Y$$

where $X_{b\_\max}(t, \overline{P})$ is the maximum isolator displacement having a probability $\overline{P}$ not to be exceeded.

Severe earthquake

This is rare earthquake having a 10% in 50 years of probability of occurrence. For this the following objectives are established:

- The displacement of the top of the pier must be smaller than the elastic limit displacement.

- This is expressed in terms of the pier reliability formula (14).
- The performance of the seismic isolation, i.e. the reduction of the seismic pier response, must be the best possible and also it needs to reduce the isolators displacement.
- Moreover, it is necessary that the isolator displacement should be smaller than the ultimate limit.

In an analogous manner this last requirement is expressed as:

$$X_{b\_\max}(t, \overline{P}) \leq X_{b\_u} \tag{15}$$

where $X_{b\_\max}(t, \overline{P})$ is the maximum isolator displacement having probability $\overline{P}$ not to be exceeded and X_{b_u} is the ultimate isolator displacement.

For the severe earthquake a suitable measure of the isolator performance is defined in probabilistic term as the ratio - $\max \sigma_{x_p} / \max \sigma_{x_{p\_0}}$ - of the maximum of the variance of the top of the pier displacement - $\max \sigma_{x_p}$ - of the isolated bridge, on the equivalent response - $\max \sigma_{x_{p\_0}}$ - for the conventional bridge.

The parameter characterizing the isolator behaviour under the severe earthquake loading, i.e. the post-elastic stiffness, is designed in this phase.

The requirement that the pier during the severe earthquake should remain in the elastic range, without showing damage consequent to inelastic deformations, is more essential for important bridges (to which it is required to remain functional) than for ordinary ones, for which a limited amount of damage can be accepted. One remember that current Technical Codes (see for example EC8, 1994) for bridges located in seismic areas authorize two different kinds of seismic behaviour and on the basis of this the performance requirements should be satisfied. More specifically the bridges should be designed in order to show under the design earthquake a ductile behaviour or an "elastic behaviour with a limited ductility". In sites where the seismic intensity is low this last behaviour is acceptable and the bridges can be designed in order to have an elastic behaviour without particular ductility requirements. In seismic area with moderate or high intensity, bridges with a ductile behaviour, having a capacity to dissipate an amount of seismic energy, are preferable. This main objective can be achievable both through the realization of plastic hinges, in proper locations, or by using isolation devices, that are in this circumstance the sacrificial elements (ATC 32, 1996), where it is possible to concentrate energy dissipation and then the damage.

Of course, a different performance objective may be required for the severe ground motion according to specificity of the analysed structure but, if performance objectives authorize some damage level for the pier, a more suitable model for it can be adopted in order to reproduce its real non linear behaviour under severe earthquakes.

DESIGN CRITERIA

In this section, the isolators optimum parameters (I_{opt} , Y_{opt} , α_{opt}) are attained by using the proposed method. The procedure is developed in two phases. In the first spep, a range of optimum values for the isolator parameters governing the elastic response (I_{opt} , Y_{opt}) is reached by using performance requirements concerning the minor earthquake. The performance objective on the pier response, that should remain elastic, is utilized together with a limit Y_{max}, which is the higher value to be assigned in the design to the isolator elastic limit displacement.

In the second step, firstly the requirement concerning the pier response for the severe earthquake are imposed for each pair of the optimum values (I_{opt} , Y_{opt}) which are obtained in the first phase. In this way the optimum value α_{opt} relative to this pair is reached. Then, by means of the safety condition for the isolator, the final optimum design parameters are obtained. This procedure is described below in detail and results obtained are represented in several graphs. The proposed procedure is also briefly shown in a scheme in Figure 7.

1. Design Criteria for the Minor Earthquake

In the first step of the proposed method, the seismic response and performance requirements to the minor earthquake are considered (this is the elastic phase design). In Figure 8 the displacements $X_{b\_\max}(t,\overline{P})$ and $X_{p\_\max}(t,\overline{P})$ having a probability $\overline{P}$ not to be exceeded are plotted against the frequency ratio *I*. The following parameters are adopted in this first phase of the optimum design: ω_g= 22rad/sec, ξ_g= 0.42, ω_p= 15rad/sec, ξ_p=0.05, and S_0 =0.0034 m^2/sec^3, corresponding to a maximum ground acceleration $\ddot{x}_{g\_\max}$ = 0.15*g*, where *g* is the gravity acceleration, obtained with the formula (Kanai and Tajimi, 1960):

$$S_0 = \frac{0.141\xi_g \ddot{x}^2_{g\max}}{\omega_g(1+4\xi_g^2)}$$

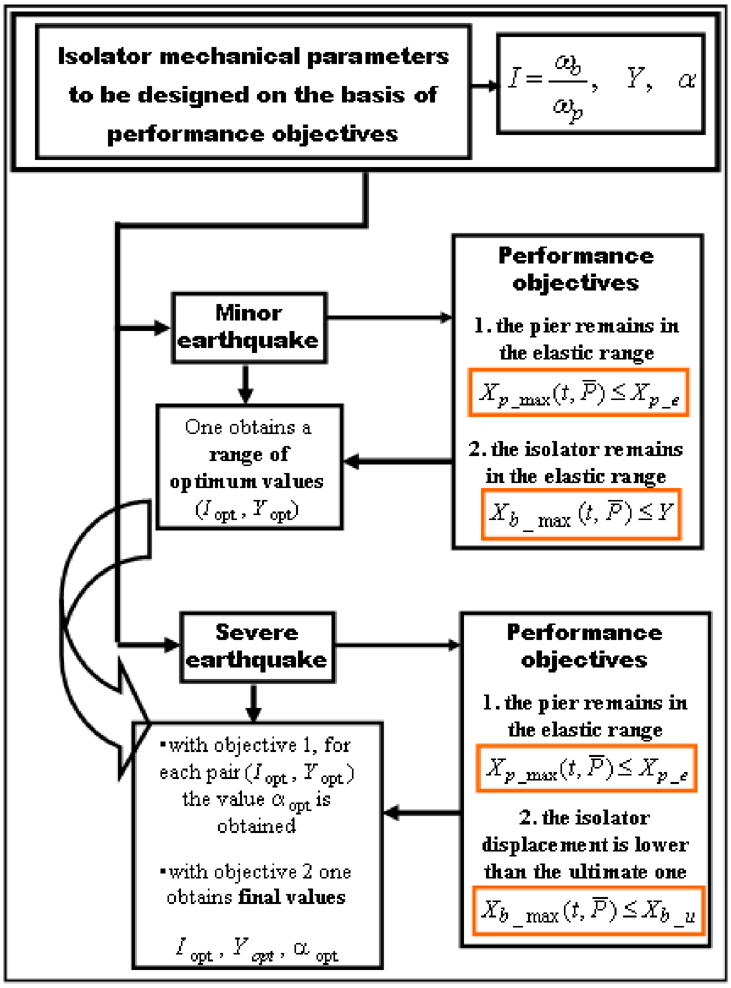


Fig. 7 Scheme of the procedure implemented in order to evaluate optimum isolator mechanical parameters

The following values: α_v=0.33 and β_v =0.125 are adopted, which correspond to $V_{\max}$=1 and $t_{\max}$=8sec, where $t_{\max}$ is the time where $V(t)$ reaches $V_{\max}$=1. The value $\overline{P}$ = 1-10^{-2} is adopted as probability target for the service earthquake. The performed optimum design method is described below.

Firstly the pier elastic limit displacement $X_{p\text{-}e}$ is established. For example, if the value 10 cm is fixed (this can be assigned in relation to specific pier characteristics) from Figure 8 one can evaluate the point A', for which the pier elastic limit displacement $X_{p\text{-}e}$ is attained with the fixed target probability $\overline{P}$.

Therefore, the corresponding value of I (point B') identifies the optimum frequency ratio I_{opt} and point C' the corresponding isolator elastic limit Y_{opt} .

If another value X_{p-e}=15 cm is fixed, one can notice form Figure 8 that this is always larger than the pier displacement. In this case one identifies the points A, B and C where I_{opt} is equal to the unit. At this point the problem in non unequivocal completed, because all values smaller than I_{opt} (point B), now named *max*I_{opt} (with the corresponding Y_{opt} (point C) - now named minY_{opt}), are admissible, and for each value smaller than *max*I_{opt} it can define a pair of values I_{opt} and Y_{opt}.

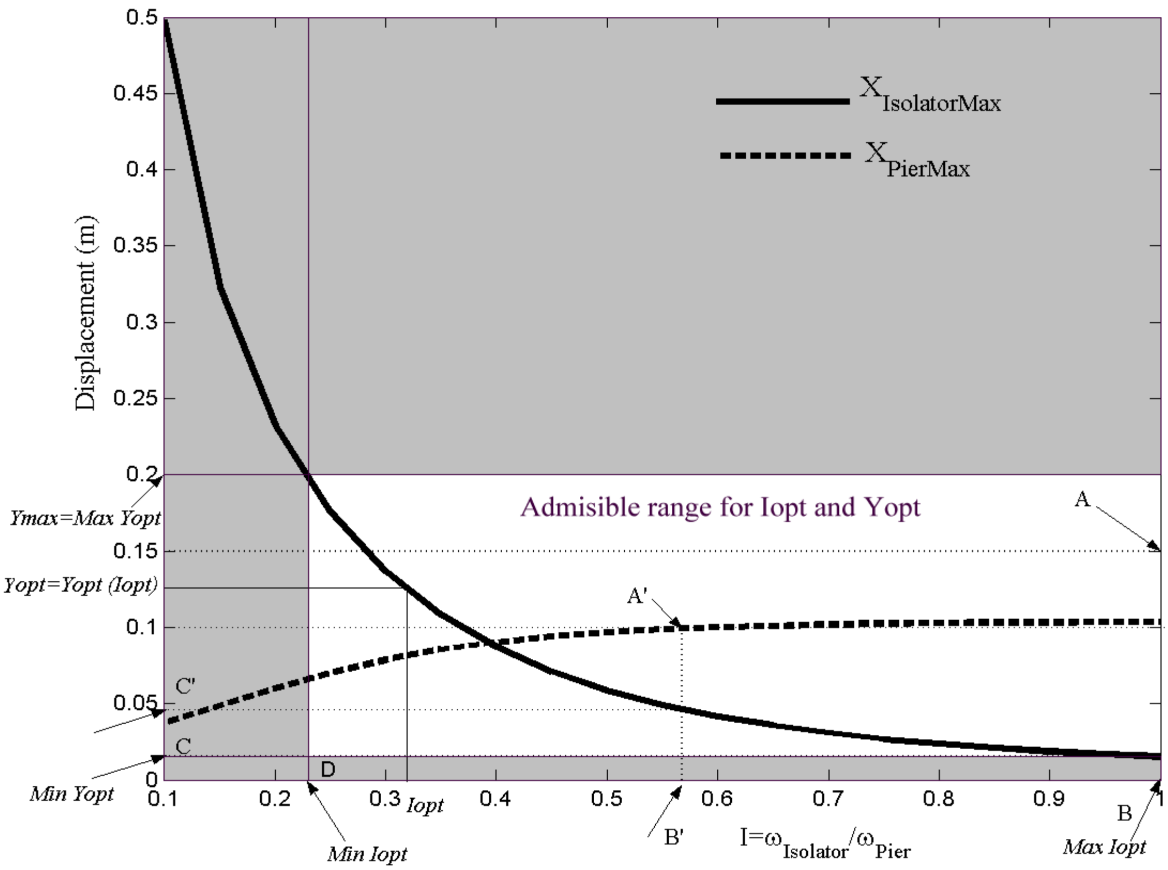


Fig. 8 Admissible range for the pair I_{opt} , Y_{opt}.

Now, it also needs to fix a limit Y_{max} for the isolator elastic displacement. In this way it can define the value *min*I_{opt} (point D) and, then a range of optimum values (I_{opt} , Y_{opt}), where $Y_{opt}= Y_{opt}(I_{opt})$, that for the service earthquake are optimally designed for the specified performance requirements. These pairs of optimum values will be utilised in the next step of the optimum design as shown as follows.

Design Criteria for the Severe Earthquake

In this section, by starting from the range of optimum pairs (I_{opt}, Y_{opt}) previously obtained, the optimum design in the post-elastic phase is carried out. The input for the system is the severe earthquake expected: the following parameters are adopted: ω_g= 22rad/sec, ξ_g= 0.42 and S_0 =0.0379 m^2/sec^3, corresponding to a maximum ground acceleration $\ddot{x}_{g\_\max}$ =0.50g.

For each pair (I_{opt} , Y_{opt}) the stochastic response to this earthquake is evaluated. In Figure 9 the displacements $X_{b\_\max}(t,\overline{P})$ and $X_{p\_\max}(t,\overline{P})$ having a probability $\overline{P}$ =1-10^{-3} not to be exceeded, are plotted against the parameter α.

The isolator elastic frequency is defined by I_{opt}, whereas in order to model the Bouc-Wen mechanical law, the parameters γ_{opt} and β_{opt} corresponding to Y_{opt}, where $Y_{opt}= Y_{opt}(I_{opt})$ are adopted. In the specific case represented in Figure 9 the pair I_{opt} = 0.331 and Y_{opt}= 8.5cm have been used. After that, because also for the severe earthquake it requires that the pier displacement doesn't overpass the elastic limit X_{p-e}, the line representing this value (15cm) is plotted in the graph. In this way it is possible to identify the point A and $\alpha_{(A)}$.

In order to supply an optimum design in a parametric form, which can be simple to use, the objective performance on the isolator displacement for the severe earthquake is not introduced at this time. For this reason, all values smaller than $\alpha_{(A)}$ are admissible and the optimum one is that minimizes the isolator displacement (point B). Finally one can identify $\alpha_{opt(B)}$ and the related pier displacement $X_{p\_\max}(\alpha_{opt})$ (point C).

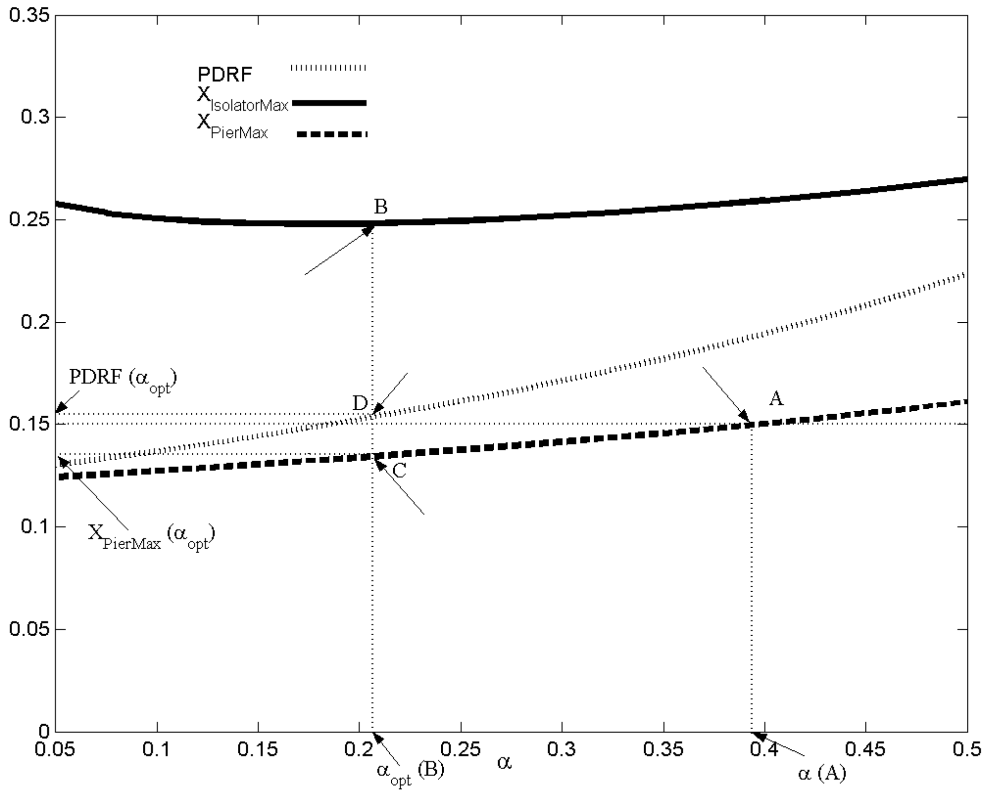


Fig. 9 Evaluation of α_{opt} for each pair I_{opt} , Y_{opt}.

In Figure 9 also the measure of the performance of the optimally designed isolator, i.e. the ratio $\max\sigma_{x_p} / \max\sigma_{x_{p\_0}}$ (briefly named PDRF- Pier Displacement Reduction Factor), is plotted. Then point D supplies the PDRF relative to the optimum isolator parameters.

This procedure, previously carried out for a pair of values (I_{opt}, Y_{opt}), is numerically implemented and extended to all pairs (I_{opt}, Y_{opt}) obtained in the elastic phase design. The results are plotted in Figure 10, where on the *x* axis there is the optimum isolation ratio I_{opt} and on the *y* axis there are α_{opt} (that is $\alpha_{opt(B)}$ in Figure 9), the corresponding isolator displacement $X_{b\_\max}(\alpha_{opt})$ and PDRF, resulting from these optimum values.

After all, when the ultimate isolator displacement X_{b_u} is fixed (in Figure 10 X_{b_u}=25 cm is assigned) it obtains the final I_{opt} (point A) and consequently Y_{opt}, α_{opt} (point B) and the corresponding PDRF (point C).

The results obtained through the proposed procedure can be generally adopted. This is because a different elastic pier displacement X_{p-e} and a different Y_{max}, only produce a variation of the admissible range $\min I_{opt}$ - $\max I_{opt}$, and then the same optimum plots can be used for the isolator design.

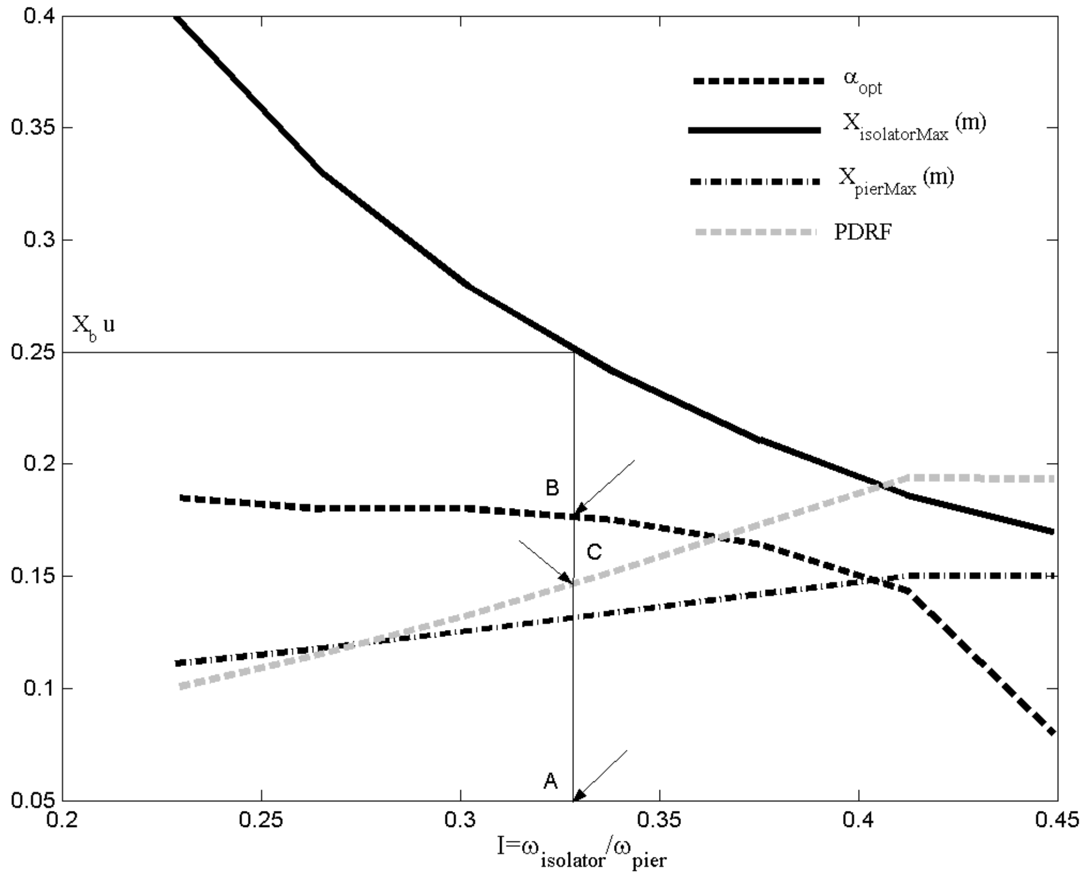


Fig. 10 Evaluation of α_{opt}

CONCLUSIONS

In this study a procedure for design isolator devices adopted for seismic isolation of bridges is developed. In order to obtain a structural model able to provide in a simple way the main objective of isolators design, some simplified hypotheses are assumed; then a system with two degree of freedom, representing respectively the pier and the rigid deck positioned on isolators is considered. Then, results reached in this study can be referred to bridges whose real configuration makes quite reasonable simplifying hypothesis here supposed.

The adopted design method is performance - reliability based, where the meaning of the design principle concerns a structure intended for satisfying requirements concerning the *safety* and the *serviceability*. The approach performed in a stochastic way provides in a simple way the isolator optimum mechanical parameters, starting from the serviceability and safety constrains required to structural elements as piers and devices, that are the elastic limit pier displacement, the elastic limit isolator displacement and the ultimate isolator displacement.

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