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SEISMIC RESPONSE OF ISOLATED BRIDGES BY HDRB

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SUMMARY

Bridges structures are key elements of transportation network and their efficiency is very much essential during the emergency phase after a seismic events. Owing to catastrophic damage levels observed on bridges as result of strong earthquakes, investigation studies in the last decades have been focused on methods able to improve their stability and reliability. Therefore, as reply of weakness inherent on conventional seismic design methods, non traditional ones have been developed intended for preventing or reducing inelastic deformations, and consequent destructive effects, in bridges structures. For this reason a great effort has been made in this field in order to introduce passive vibration control devices with the aim to improving bridges behavior against seismic events, and to avoiding, simultaneously, significant cost increments. In this study the performance of *HDRB* utilized for bridges seismic protection is assessed; in detail, the reliability, evaluated in terms of first passage probability, is calculated and a sensitivity analysis is carried out in order to recognize the most suitable isolator mechanical parameters which allow to attain high performance of this technique.

INTRODUCTION

In the last decades the use of seismic isolation is getting increased attention from designers for seismic bridges hazard mitigation. This technique, having the main aim to protect relatively low mass elements as piers and foundations, makes use of seismic isolators located between bridge deck and piers. The use of aseismic devices is quite easy in bridges, because these can be located by replacing conventional devices adopted to accommodate thermal movements, with isolation systems. For the seismic protection of bridges, Rubber Bearings (*RB*), Lead Rubber Bearing (*LRB*) and High Damping Rubber Bearing (*HDRB*) are extensively adopted. The most important characteristic of these devices is the opportunity to provide into a single element, vertical support, lateral flexibility, restoring force and dissipation energy. Symans et Al [1] supplied an extensive review of current applications in this field.

Seismic isolation is a strategy intended for reducing seismic forces to or near the elastic limit capacity of structures, curtailing at the same time inelastic deformations and related damage phenomena. With the use of seismic isolation in bridges, reduction of shear forces transmitted from the superstructure to the piers is achieved thanks to the combination of two effects: first, the isolator works by shifting the natural period of the bridge away from the frequency range where is high the energy content of earthquakes. Also, the

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energy demand required to the structure from the earthquake is also reduced thanks to the dissipation energy concentrated is these devices, properly designed for this purpose. In this study performance of *HDRB* used for bridges seismic isolation is assessed; in detail, the reliability evaluated in terms of first passage probability is calculated. In order to take into account of random nature of earthquakes, the analysis will be performed using a stochastic approach. More precisely, a gaussian, zero mean, non-stationary, filtered stochastic process is used in order to realistically model the seismic action. Moreover, as consequence of using the isolation technique, the constitutive behavior of the bridge is assumed linear, whereas a non linear model is adopted for the isolation system. After the covariance analysis is performed by using the approximate technique of stochastic linearization, the reliability of the isolated bridge is evaluated in the hypothesis of Poisson assumption. The reliability analysis is adopted in this study with the aim to performing serviceability assessment for the isolated bridge. For this scope two different criteria of serviceability have be introduced. The first one regards the displacement of the top of the pier: more precisely, the requirement that the pier remains in the elastic range is attained if this displacement does not exceed a specific value x_p^{max}, that assumes the meaning of the critic threshold level for the reliability assessment. The second serviceability criterion considers the relative displacement of the superstructure respect to the pier; this is related to acceptable isolator deformations and to requirements for the superstructure response.

STRUCTURAL MODEL AND MOTION EQUATIONS

The dynamic analysis of bridges structures is a quite complex problem and therefore it needs to introduce some simplifications in order to make easy the formulation of a mathematical model able to represent in a suitable manner their dynamic behavior under seismic actions. In bridges structures the *HDRB* are installed between the deck and the columns; moreover, the deck is assumed rigid and the piers are supposed linear as result of the reduction of seismic forces due to devices installation. When seismic devices are located between the deck and the piers it is possible to model the isolated bridge by means of a 2 degrees of freedom system, having masses m_s and m_p. The piers can be well represented in their first vibration mode by means of the first natural frequency ω_p and the damping coefficient ξ_p. The rigid deck, instead, can be well modeled by means of a concentrate mass m_s located on the isolator. The elastic frequency ω_b and the damping coefficient ξ_b of the isolator are defined, respectively, as:

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

where k_b and c_b are the isolator elastic stiffness and the damping.

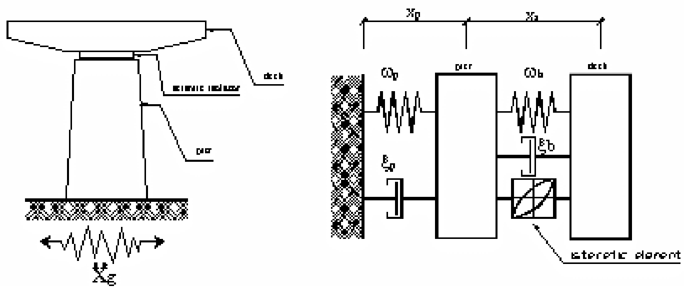


Figure 1: Schematic model of the isolated bridge.

By introducing the displacement vector $\mathbf{u} = \{u_p,\ u_s\}^T$, where u_p and u_s are, respectively, the displacement of the top of the pier respect to the ground and of the deck relative to the pier, the equations of the motion for the isolated bridge subjected to a seismic motion represented by the time modulated Kanai-Tajimi process [2] are:

$$\begin{cases} m_p\ddot{u}_p + c_p\dot{u}_p - c_b\dot{u}_s + k_p u_p - \alpha_b k_b u_s - (1-\alpha_b)k_b z_b = -m_p\ddot{x}_g \\ m_s\ddot{u}_s + m_s\ddot{u}_p + c_b\dot{u}_s + \alpha_b k_b u_s + (1-\alpha_b)k_b z_b = -m_s\ddot{x}_g \\ \dot{z}_b = -\gamma_b|\dot{u}_s||z_b|^{\eta_b-1}z_b - \beta_b\,\dot{u}_s|z_b|^{\eta_b} + A_b\dot{u}_s \\ \ddot{x}_f + 2\xi_g\omega_g\dot{x}_f + x_f\omega_g^2 = -V(t)w \end{cases} \tag{1}$$

In equation (1) the Bouc-Wen model [3] has been adopted in order to describe the isolator constitutive law. This is characterized by a non linear behavior and by energy dissipation; this is a very versatile model able to reproduce several constitutive laws, both softening and hardening, with and without degradation, when the parameters defining the model are correctly selected. Complete information about the Bouc-Wen model can be found in the study of Baber and Wen [4]. More precisely, z_b is an internal variable related to hysteretic behavior; also the parameters β_b, γ_b, η_b, α_b and A_b govern the shape of the isolator hysteretic cycle and can be selected in order to reproduce the observed hysteretic cycle of isolators. Furthermore, in equation (1) the filter motion equation is present, where x_f is the response of the filter having frequency ω_g and damping coefficient ξ_g. Finally, w is the white noise excitation process at the bed rock and $V(t)$ is the exponential modulation function: $V(t) = \alpha_v t e^{-\beta_v t}$.

The problem just stated is non linear as consequence of the isolator hysteretic behavior. Many different mathematical methods can be used in order to solve this stochastic problem. In the research here presented, the equivalent stochastic linearization is adopted. This technique is the most practical approximate method for the analysis of stochastic non linear structural dynamic problems. The basic idea of the method is that the non linear equation can be well replaced by an appropriate linear one; moreover, the hypothesis of a gaussian response process must be assumed. The approximate linearized equation is then achieved by making minimum, in a stochastic way, the difference between the non linear equation and the linearized one. As result, with the equivalent stochastic linearization method, the non linear equation governing the internal variable z_b is replaced with the next linear one:

$$\dot{z}_b = -c_b{}^e\,\dot{u}_s - k_b{}^e\,z_b \tag{2}$$

Atalik and Utku [5] provided the equivalent coefficients $c_b{}^e$ and $k_b{}^e$ for $A_b = 1$ and $\eta_b = 1$, in the hypothesis of variables z_b and $\dot{u}_s$ jointly gaussian:

$$\begin{cases} c_b{}^e = \sqrt{\dfrac{2}{\pi}}\left[\beta_b\,\sigma_{z_b} + \gamma_b\dfrac{E[\dot{u}_s\,z_b]}{\sigma_{\dot{u}_s}}\right] - A_b \\ k_b{}^e = \sqrt{\dfrac{2}{\pi}}\left[\gamma_b\,\sigma_{\dot{u}_s} + \beta_b\dfrac{E[\dot{u}_s z_b]}{\sigma_{z_b}}\right] \end{cases} \tag{3}$$

In equation (3) the terms σ_{z_b} and $\sigma_{\dot{u}_s}$ are, respectively, the standard deviations of variables z_b and $\dot{u}_s$; moreover, $E[\dot{u}_s z_b]$ is the covariance of the mentioned variables. By adopting the approximate method of the stochastic linearization, the motion equations can be written in the following way:

$$\begin{cases} m_p \ddot{u}_p + c_p \dot{u}_p - c_b \dot{u}_s + k_p u_p - \alpha_b k_b u_s - (1-\alpha_b) k_b z_b = -m_p \ddot{x}_g \\ m_s \ddot{u}_s + m_s \ddot{u}_p + c_b \dot{u}_s + \alpha_b k_b u_s + (1-\alpha_b) k_b z_b = -m_s \ddot{x}_g \\ \dot{z}_b = -c_e^b \dot{u}_s - k_e^b z_b \\ \ddot{x}_f + 2\xi_g \omega_g \dot{x}_f + x_f \omega_g^2 = -V(t) w \end{cases} \tag{4}$$

where the linearization coefficients $k_e{}^b$ and $c_e{}^b$ appear.
After one introduces the 7 elements *state vector* **Y**:

$$\mathbf{Y} = \{\mathbf{u} \quad x_f \quad z_b \quad \dot{\mathbf{u}} \quad \dot{x}_f\}^T$$

the state equation for the system can be written as:

$$\dot{\mathbf{Y}}(t) = \mathbf{A}^{\mathbf{e}} \; \mathbf{Y}(t) + \mathbf{B}(t) \tag{5}$$

where **B**(*t*) is a vector of order 7, called *transfer input vector*, having all elements equal zero except $B_7(t) = -wV(t)$ and where $\mathbf{A}^{\mathbf{e}}$ is a square matrix of order 7, representing the *equivalent linearized state matrix*. After the state equation is written, the stochastic analysis can be performed by solving the differential equation of the covariance matrix $\mathbf{Q}_{\mathbf{YY}}(t)$, whose elements, variable in the time, are the second order moments $E[Y_i \, Y_j]$ relative to the state vector **Y**:

$$\dot{\mathbf{Q}}_{\mathbf{YY}}(t) = \mathbf{A}^{\mathbf{e}} \mathbf{Q}_{\mathbf{YY}}(t) + \mathbf{Q}_{\mathbf{YY}}(t) \mathbf{A}^{\mathbf{e}^{\mathbf{T}}} + \mathbf{G}(t) \tag{6}$$

In equation (6) $\mathbf{G}(t)$ 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.

RELIABILITY ASSESSMENT

One of the most crucial problem in the structural design process is the reliability assessment. An ordinary measure of the reliability is to define this as the complement of the failure probability. One of the main reason for studying the response of a system to a random input is the possibility to assess this failure probability. In fact, for a structure subjected to a random input the failure probability is usually formulated as the first passage problem, i.e. the probability that a specific response will exceed a given threshold level during a fixed time range. The observation time may be, for example, the lifetime of the structure or the strong motion duration of an earthquake.
Therefore, if the failure criterion for a structure is reached when the absolute value of a general response $|X(t)|$ exceeds a given critic threshold level during an observation time [0-*t*] (life time), then the reliability of the structure is obtained when there is not failure in the range [0-*t*], that is when the threshold limit is not reached. Then, if $X(t)$ is the system response process, the complement of the failure probability, i.e. the structural reliability, indicates the probability $P_s(t,\xi)$ that the response process $|X(t)|$ does not exhibit any crossing of the threshold limit $\pm\xi$ (double barrier) when $0 < \tau < t$. Exact solutions have not get been developed for the crucial problem of the first excursion failure probability, but a variety of methods with different degree of approximation are available in literature. In this study the reliability function $P_s(t,\xi)$ is evaluated in the hypothesis of independent threshold crossings, with a Poisson distribution [6]:

$$P_S(t,\xi) = \exp\left\{-\int_0^t \alpha(t)dt\right\} = \exp\left\{-\int_0^t 2\nu_{\xi^+}(t)dt\right\} = \exp\left\{-\int_0^t 2\nu_{0^+}(t) e^{-\frac{1}{2}\frac{\xi^2}{\sigma_x^2(t)}} dt\right\}$$

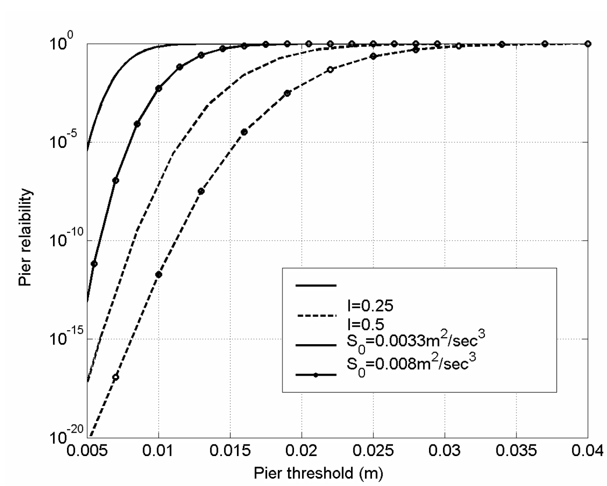
where:

$$\nu_{\xi^+}(t) = \nu_{0^+}(t)\exp\left[-\frac{\xi^2}{2\sigma_x^2(t)}\right]$$

and:

$$\nu_{0^+}(t) = \frac{1}{2\pi}\frac{\sigma_{\dot{x}}(t)}{\sigma_x(t)}$$

are, respectively, the mean number of crossings of the thresholds ξ_+ and 0_+ in the unit time.
The reliability analysis is adopted in this study with the aim to performing the assessment on the serviceability requirements for the isolated bridge: for this scope two different criteria of serviceability have be introduced. The first one regards the displacement of the top of the pier: more precisely, the requirement that the pier remains in the elastic range is attained if this displacement does not exceed a specific value x_p^{max}, that assumes the meaning of the critic threshold level for this reliability assessment.
The second serviceability criterion considers the relative displacement of the superstructure respect to the pier; this is associated to acceptable isolator deformations and to serviceability requirements for the superstructure.
In figures 2, 3 the reliability of the pier and the deck, evaluated by adopting the Poisson assumption, is discussed. The most significant parameters controlling the isolator performances are the ratio $I = \omega_b / \omega_p$ that is the ratio between the elastic isolator frequency and pier one, and the parameter α_b from whose the inelastic device behavior depends. In order to achieve the reliability assessment the stochastic model for a typical earthquake expected on a ground having a moderate flexibility, as the El Centro 1940 one, is considered. This earthquake is characterized by an energy content concentrate in the range 1-4 Hz with a peak acceleration around 0.34g. Therefore, in the KT model the values ω_g=15rad/sec, ξ_g=0.42 are adopted, whereas two intensities: S_0=0.0033m^2/sec^3 and S_0=0.0080m^2/sec^3 (that is a more severe earthquake), are considered. Finally for the modulation function $V(t)$, α_v=0.33 and β_v =0.125 are assumed, which correspond to $V_{\max}$=1 and $t_{\max}$=8sec, where $t_{\max}$ is the time where $V(t)$ attains $V_{\max}$.
Figures 2 and 3 corresponds to α_b=0.4 and α_b=0.2, respectively. Different lines are referred to I=0.25 and I=0.5. First of all, the results point out that the pier reliability improves is a lower isolation ratio is adopted. Concerning the pier serviceability requirement it needs, as previous specified, the this remains in the elastic range. The limit x_p^{max} =0.025m at the top of the pier is then fixed. In probabilistic terms, the serviceability requirement is satisfied if the failure probability, that is the probability that this limit is exceeded is very small. The fixed failure probability target is 10^{-4}, then $1\text{-}10^{-4}$ =0.999 is obtained for the reliability. Concerning the deck, a displacement x_s^{max} =0.15m is fixed as limit displacement, and the same reliability target 0.999 is assumed. Figures 4 and 5, where the horizontal line corresponds to the fixed reliability target, represent two details of figures 2 and 3, respectively.



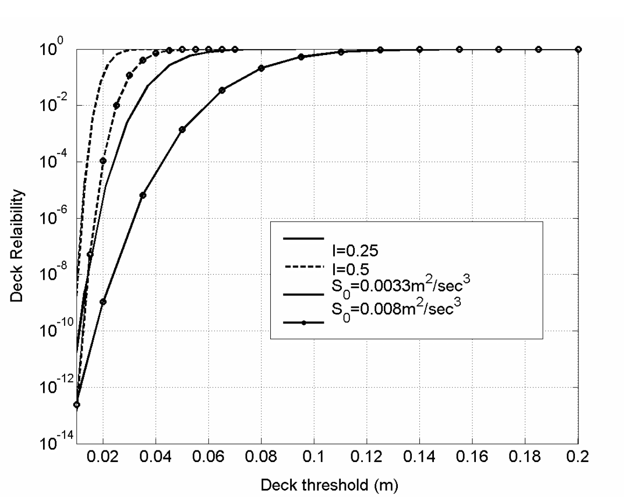


Figure 3. Pier and deck reliability vs. pier and deck threshold levels: ω_p=20rad/sec, ξ_b=10%, ξ_p=5%, α_b=0.2, μ=5, Y=0.01 m.

Now, it is possible to verify if the requirements previous established are satisfied. About the pier, the target is reached only for -I=0.25, S_0=0.0033m^2/sec^3, α_b=0.4-, for -I=0.25, S_0=0.0033m^2/sec^3 and S_0=0.0080m^2/sec^3 , α_b =0.2-, whereas if the value I=0.5 is adopted, the pier serviceability is never assured. About the deck displacement, the reliability is always guaranteed except that for -I=0.25, S_0=0.0080m^2/sec^3 , α_b =0.2.
By using these graphs, tanks to the sensitivity analysis carried out, therefore, one can assess both concerning the satisfaction of bridge serviceability requirements and, also regarding the most suitable mechanical isolator parameters, that are the isolation ratio I and the post-elastic stiffness ratio α_b, which allow to reach high performance of this technique.

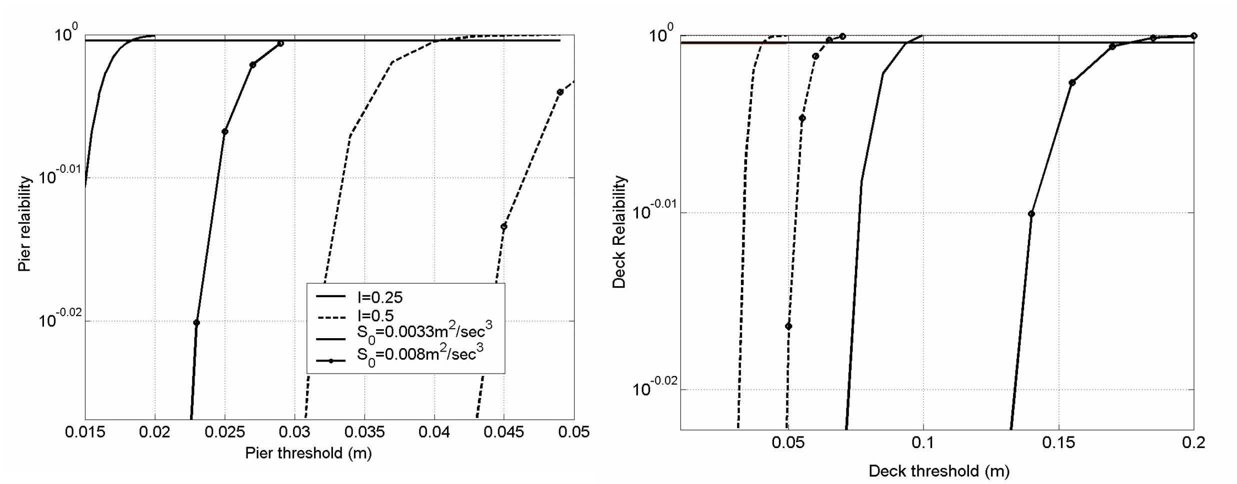


Figure 4: Pier and deck reliability vs. pier and deck threshold levels: ω_p=20rad/sec, ξ_b=10%, ξ_p=5%, α_b=0.4, μ=5, Y=0.01 m.

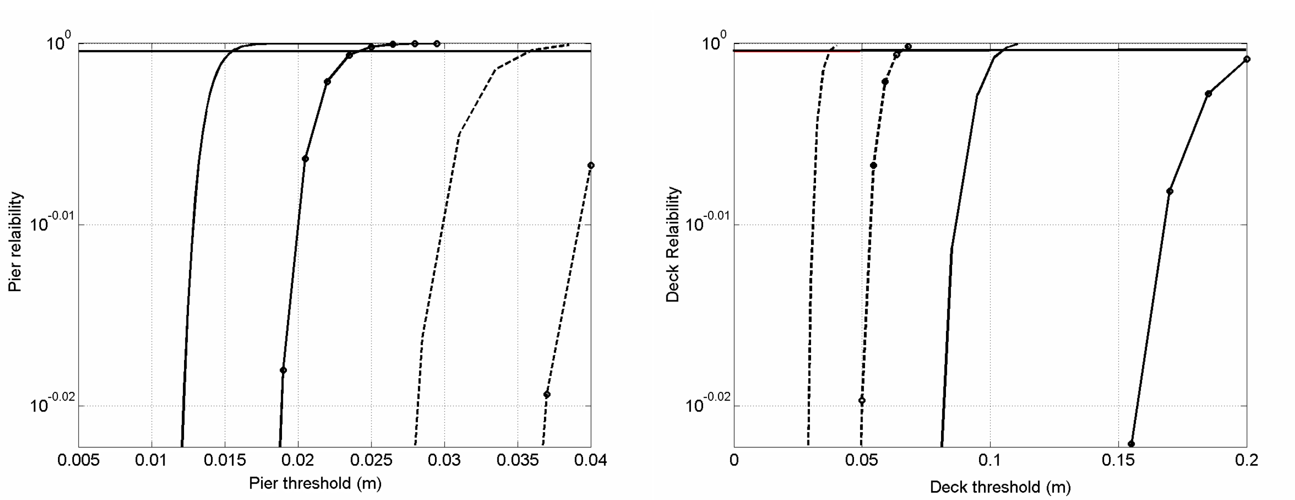


Figure 5: Pier and deck reliability vs. pier and deck threshold levels: ω_p=20rad/sec, ξ_b=10%, ξ_p=5%, α_b=0.2, μ=5, Y=0.01 m.

CONCLUSIONS

In this study the dynamic response of bridges isolated by *HDRB* to seismic actions modeled as non-stationary, stochastic, filtered processes, is assessed. The main goal of the study is to evaluate the devices performance in terms of reduction of pier and deck response and, also, in terms of the satisfaction of pier and deck serviceability requirements, with the final aim to select suitable mechanical parameters of devices, which guarantee high results of this technique. The analysis shows that the use of isolators can substantially protect low mass elements, such as the piers, from the high inertia forces transmitted from the deck. In this way the piers remain in the elastic range, as has been evaluated performing a reliability assessment; moreover, by selecting suitable isolator mechanical parameters, also the serviceability requirement for the deck is achieved.

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