Seismic protection of bridges using viscous dampers

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Abstract

The proposed paper deals with the seismic protection of bridges by means of passive energy dissipation viscous devices installed between the continuous beam deck and the supporting piers. An analytical approach is used to study the problem and a simple design procedure, using the frequency response analysis, is provided to determine the optimal value of the viscous damping parameter characterizing the dampers. Finally, the adequacy of the procedure is verified through time history analyses on an existing bridge.
Keywords: bridges, retrofit of bridges, viscous devices, passive control.

1 Introduction

The most recent bridges' seismic design procedures, which are essentially related to the concepts of performance-based and damage-controlled design, have made clear that the only increase of the design force levels does not improve the earthquake bridge performance. The preferential innovative strategy for controlling excessive vibrations induced by earthquake loads in new bridge structures (design perspective) as well as in existing ones (retrofit perspective), is the use of supplemental energy dissipation systems. They generally require a relative movement in the structure to activate the dissipation devices. The aim of incorporating supplemental damping devices in the design or retrofit of a bridge structure is the reduction of the dynamic response amplification. This is obtained by increasing the structural equivalent viscous damping according to the response spectra of the earthquake record at the site. In fact, the addition of a supplemental damping to a structure is more beneficial if the fundamental period of the construction including the damping devices falls within the range where dynamic amplification occurs for the expected earthquake motion. In Italy, with the recently approved seismic code for earthquake resistant structures, few simple code requirements are provided for the use of supplemental energy dissipation devices in bridges [1].

Several different types of energy dissipation devices have been proposed for this purpose. For bridges, the most effective and extensively applied are the viscous fluid dampers operating on the principle of fluid flow through orifices. During the last years, significant research efforts have been made in the development of the manufactured energy absorbing devices, and in their analytical and experimental characterization, while the progresses in the definition of specific design procedures have been slower. However, a few authors have tackled the problem and proposed some procedures referred to different energy dissipation devices introduced in bridges' isolation systems [2, 3, 4, 5].

For a particular class of bridges as specified in the next paragraph, a completely new design procedure based on minimization of response in the frequency domain has been developed and is presented in this paper.

2 Optimal design of protection system

Several existing bridges are characterized by a continuous deck connected to some piers through fixed bearings, and to others through bearings allowing sliding in the longitudinal direction (Figure 1a). In this way, the earthquake load on the whole deck is transferred to the fixed piers only. Limitation of deck displacements and accelerations and a significant reduction of the stresses on the fixed piers can be achieved through the insertion of passive viscous energy dissipation devices between the bridge deck and the longitudinally free piers. In the case of piers characterized by the same height and cross-section, a simplified SDOF model of such a kind of bridge is shown in Figure 1b, where k_f is the global stiffness of the fixed piers, k_b is the global stiffness of the other piers, m_f is the total mass of the deck, and C_d is the viscous damping parameter characterizing the passive linear viscous dampers.

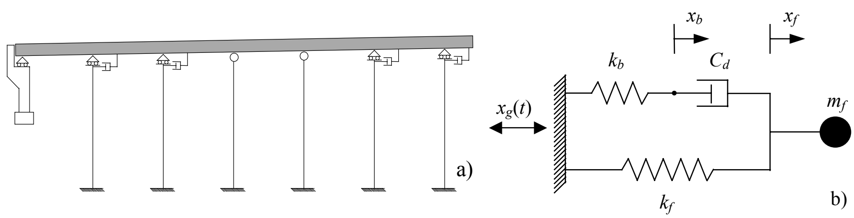
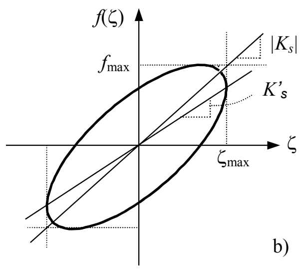


Figure 1: (a) Scheme of existing bridge, (b) SDOF model.

To the aim of singling out the optimal viscous damping constant C_d able to optimize the bridge response in terms of deck displacement, a frequency domain approach has been used to analytically examine the system in Figure 1b subjected to a harmonic base displacement $x_g(t) = x_{g,\max} \cdot e^{i\varpi t}$ [6].



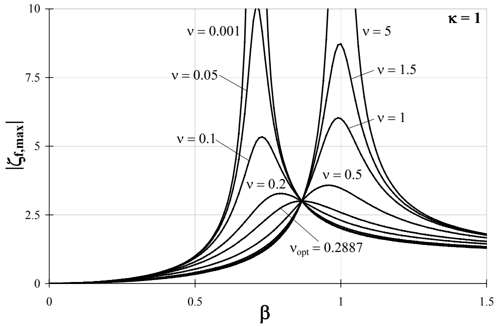


Figure 2: (a) $f(\zeta)-\zeta$ cycle, (b) transfer functions $\left|\zeta_{f,\max}(\beta)\right|$ for $\kappa = 1.0$.

The general expression of the equation of motion is the following:

$$m_f \cdot \ddot{x}_f + k_f \cdot x_f + F_d = m_f \varpi^2 x_{g,\max} \cdot e^{i\varpi t} \tag{1}$$

where $F_d = k_b \cdot x_b = C_d \cdot (\dot{x}_f - \dot{x}_b)$. The force – displacement relationship for the Maxwell element composed by the spring k_b and the viscous dashpot C_d in series is given by the well known expression $F_d(x_f) = K_d(\varpi) \cdot x_f$, where the complex stiffness $K_d(\varpi)$ is obtained as follows:

$$x_f = x_b + \frac{\dot{x}_f - \dot{x}_b}{i\varpi} = F_d\left(\frac{1}{k_b} + \frac{1}{i\varpi C_d}\right) \quad \Rightarrow \quad F_d = \frac{k_b C_d^2 \varpi^2 + i k_b^2 C_d \varpi}{k_b^2 + \varpi^2 C_d^2} x_f \quad \Rightarrow$$
$$\Rightarrow \quad K_d(\varpi) = \frac{k_b C_d^2 \varpi^2}{k_b^2 + \varpi^2 C_d^2} + i \frac{k_b^2 C_d \varpi}{k_b^2 + \varpi^2 C_d^2} = K'_d(\varpi) + i K''_d(\varpi) \tag{2}$$

and $K'_d(\varpi)$ and $K''_d(\varpi)$ are respectively the storage and the loss modulus of the Maxwell element. Therefore Eq (1) can be written as:

$$m_f \cdot \ddot{x}_f + \left[k_f + K'_d(\varpi) + i K''_d(\varpi)\right] \cdot x_f = m_f \varpi^2 x_{g,\max} \cdot e^{i\varpi t} \tag{3}$$

The above equation can be usefully written in non-dimensional form, by introducing two non-dimensional parameters. By considering the non-dimensional time $\tau = t \cdot \omega_b$, where $\omega_b = \sqrt{(k_f + k_b)/m_f}$ represents the circular frequency in the case of infinitely stiff damper ($C_d \to +\infty$), and substituting the derived expressions $x_f(t) = x_f[\tau(t)] = x_f(\tau)$ and $\ddot{x}_f(t) = \omega_b^2 \cdot x''_f(\tau)$ in Eq 3, the following equation of motion can be obtained:

$$x_f'' + \frac{1}{k_f + k_b} \cdot \left[k_f + K_d'(\varpi) + iK_d''(\varpi)\right] \cdot x_f = \beta^2 x_{g,\max} \cdot e^{i\beta\tau} \tag{4}$$

with $\beta = \varpi/\omega_b$. By introducing again the non-dimensional displacement $\zeta_f(\tau) = x_f(\tau)/x_{g,\max}$ and substituting the derived expression $x_f''(\tau) = x_{g,\max} \cdot \zeta_f(\tau)$ in Eq 4, this becomes $\zeta_f''(\tau) + f(\zeta_f) = \beta^2 e^{i\beta\tau}$, where $f(\zeta_f) = \left[k_f \cdot x_f + F_d(x_f)\right] / \left[(k_f + k_b) \cdot x_{g,\max}\right] = K_s(\beta) \cdot \zeta_f(\tau)$ represents the normalized restoring force (Figure 2a). The force – displacement relationship is completely defined once the complex stiffness $K_s(\beta)$ of the controlled structure is known:

$$K_s(\beta) = \frac{\kappa}{1+\kappa} + \frac{4\beta^2\nu^2\kappa(1+\kappa)}{(1+\kappa)\left[1+4\beta^2\nu^2\kappa(1+\kappa)\right]} + i\frac{2\beta\nu\sqrt{\kappa(1+\kappa)}}{(1+\kappa)\left[1+4\beta^2\nu^2\kappa(1+\kappa)\right]} = \tag{5}$$
$$= K_s'(\beta) + iK_s''(\beta)$$

where $K_s'(\beta)$ and $K_s''(\beta)$ are the overall storage and the loss modulus of the controlled structure, and the non-dimensional parameters $\kappa = k_f/k_b$ and $\nu = C_d / 2\sqrt{k_f m_f}$ represent the fixed piers/free piers relative global stiffness and the viscous damping ratio in the limit case $k_b \to +\infty$. As the system is linear, an exact solution can be easily evaluated. The steady-state response $\zeta_f(\tau) = \zeta_{f,\max} e^{i\beta\tau}$ is periodic with frequency β and the amplitude of motion is the modulus of the complex number $\zeta_{f,\max}(\beta)$:

$$\zeta_{f,\max}(\beta) = \frac{\beta^2}{\dfrac{\kappa}{1+\kappa} + \dfrac{4\beta^2\nu^2\kappa}{1+4\beta^2\nu^2\kappa(1+\kappa)} - \beta^2 + i\dfrac{2\beta\nu\sqrt{\kappa(1+\kappa)}}{(1+\kappa)\left[1+4\beta^2\nu^2\kappa(1+\kappa)\right]^2}} \Rightarrow$$
$$\Rightarrow \quad \left|\zeta_{f,\max}(\beta)\right| = \frac{\beta^2}{\sqrt{\left(\dfrac{\kappa}{1+\kappa} + \dfrac{4\beta^2\nu^2\kappa}{1+4\beta^2\nu^2\kappa(1+\kappa)} - \beta^2\right)^2 + \dfrac{4\beta^2\nu^2\kappa}{(1+\kappa)\left[1+4\beta^2\nu^2\kappa(1+\kappa)\right]^2}}} \tag{6}$$

The following limit cases can be considered:

$$\nu = 0 \quad \Rightarrow \quad C_d = 0 \text{ (no damper)} \Rightarrow \quad \left|\zeta_{f,\max}(\beta)\right| = \frac{\beta^2}{\left|\kappa/(1+\kappa) - \beta^2\right|};$$

$$\nu \to +\infty \quad \Rightarrow \quad C_d \to +\infty \text{ (infinitely stiff damper)} \Rightarrow \quad \left|\zeta_{f,\max}(\beta)\right| = \frac{\beta^2}{\left|1-\beta^2\right|}.$$

The quantity $\left|\zeta_{f,\max}(\beta)\right|$ as a function of β is plotted in Figure 2b for $\kappa = 1.0$ ($k_b = k_f$) and for several different values of ν: the strong influence of the device damping coefficient on the system response comes out, i.e. a deep modification in the dynamic behaviour of the structure can be produced by a change in the viscous constant of the device. All the curves have a common point, corresponding to the intersection of the two limit curves for $\nu = 0$ and $\nu \to +\infty$, whose co-ordinates are $\overline{\beta} = \sqrt{(1+2\kappa)/[2(1+\kappa)]}$ and $\left|\zeta_{f,\max}\left(\overline{\beta}\right)\right| = 1 + 2\kappa$. The belonging of this point to all the curves has been verified by checking that $\left|\zeta_{f,\max}\left(\overline{\beta}\right)\right| = 1 + 2\kappa$. Among such curves, that one, for which the aforesaid point represents a maximum, i.e. the minimum resonance peak in the range $\nu = [0,+\infty]$, is obtained by equating to zero the derivative of Eq 6 with respect to β:

$$\frac{128\beta^6\left(\beta^2-1\right)\kappa^3(1+\kappa)^5\nu^6 + 16\left[2\beta^6\kappa^2(1+\kappa)^3(1+3\kappa) - 3\beta^4\kappa^2(1+\kappa)^2\left(2\kappa^2+2\kappa+1\right)\right]\nu^4}{(1+\kappa)\left[1+4\beta^2\nu^2\kappa(1+\kappa)\right]^3} + \\ + \frac{4\left[6\beta^4\kappa^2(1+\kappa)^2 - \beta^2\kappa(1+\kappa)\left(6\kappa^2+2\kappa+1\right)\right]\nu^2 + 2\beta^2\kappa(1+\kappa) - 2\kappa^2}{(1+\kappa)\left[1+4\beta^2\nu^2\kappa(1+\kappa)\right]^3} = 0 \tag{7}$$

and computing it for $\beta = \overline{\beta}$. It corresponds to the physically acceptable real root of the 3rd degree equation in the unknown ν^2:

$$8\kappa^2(1+\kappa)(1+2\kappa)^3\left(\nu^2\right)^3 + 4\kappa(1+2\kappa)^2(\kappa+2)\left(\nu^2\right)^2 - 2(1+2\kappa)(\kappa-1)\nu^2 - 1 = 0 \tag{8}$$

Such real solution, provided by the well known formula of Tartaglia-Cardano, represents the optimal (in the sense that it minimizes the amplitude of the resonance peak) value ν_{opt} of the parameter ν, for each assumed value of the parameter κ:

$$\nu^2_{opt,j+1} = \varepsilon^j \sqrt[3]{-\frac{q}{2} + \sqrt{\Delta}} + \varepsilon^{3-j}\sqrt[3]{-\frac{q}{2} - \sqrt{\Delta}} - a \qquad j \in \{0,1,2\} \tag{9}$$

where:

$$-\frac{q}{2} = \frac{1}{216\kappa^3(1+\kappa)^3}, \qquad \Delta = 0 \quad \forall\kappa, \qquad a = \frac{\kappa+2}{6\kappa(1+\kappa)(1+2\kappa)} \tag{10}$$

and $\varepsilon^0 = \varepsilon^3 = 1$, $\varepsilon^1 = -1/2 + i\sqrt{3}/2$ and $\varepsilon^2 = -1/2 - i\sqrt{3}/2$ are the cubic roots of the unity. We can state that, as $\Delta = 0$, a simple root $2\sqrt[3]{-q/2} - a$ and a double root $-\sqrt[3]{-q/2} - a$ are obtained, both real and of opposite sign: the first one is always positive, the second one is always negative. Therefore ν_{opt} is given by the square root of the first one:

$$\nu_{opt} = \sqrt{2\sqrt[3]{-\frac{q}{2}} - a} = \sqrt{\frac{1}{2(1+\kappa)(1+2\kappa)}} \tag{11}$$

The design spectrum in Figure 3 reports the optimal value of the damper parameter ν_{opt} as function of the relative stiffness κ.

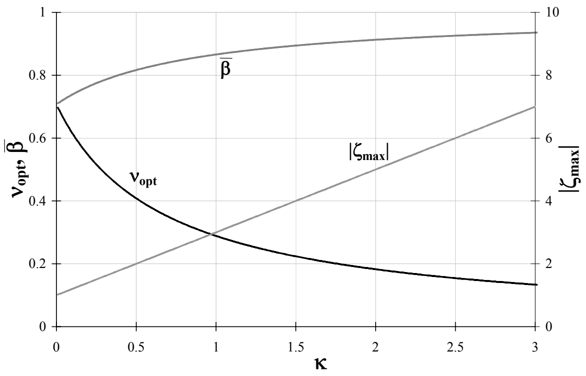


Figure 3: Design spectra $\nu_{opt} - \kappa$.

3 Description of the case study structure

In the framework of the Project "Diagnostic and protection of architectural structures with particular reference to the effects deriving from seismic events and other natural disasters", financed by the Italian Ministry of Education, University and Research (2002-2005), an existing continuous bridge (Viadotto Grandi Luci) has been selected as case study in a seismic perspective. The bridge is a section of the highway of Napoli (Italy) connecting Ponticelli quarter to Malta Street. It is considered as a way of escape from Napoli in case of natural disaster (i.e. eruption of Vesuvio, earthquake, etc.) and its structural scheme is a 4-span continuous beam, having a maximum span length of 110 m. The photos in Figure 4 show the end piers. Piers 2 and 3 are connected to the deck through fixed bearings, while for the other piers (1, 4 and 5) sliding in the longitudinal direction is allowed using steel-teflon bearings. Photos in Figure 5 show the main girder from each of the two ends of the considered bridge section, while Figure 6 indicates the geometrical dimensions of the deck cross-section, made of a box-girder with cantilevers on both sides (steel Fe510 according to the Italian classification, i.e. SJ355 according to international classifications: characteristic ultimate strength f_{tk} = 510 MPa, characteristic yield strength f_{yk} = 355 MPa). The piers (Figure 4) have a variable height from approx. 26 m (pier 1) to approx. 36 m (pier 5), and are made of reinforced concrete C25/30 with 430 MPa yielding stress steel reinforcement (Figure 7). Regarding soil characteristics, the upper 15 m consist in altered pozzolana (N_{SPT} < 15 from Standard Penetration Test), i.e. a fine ashes matrix including pumices. Under this stratum, a less compressible pozzolana is found (15 < NSPT < 50) which includes strata of fine and coarse sand aggregate, as well as inserts of fine pumices and lapilli.

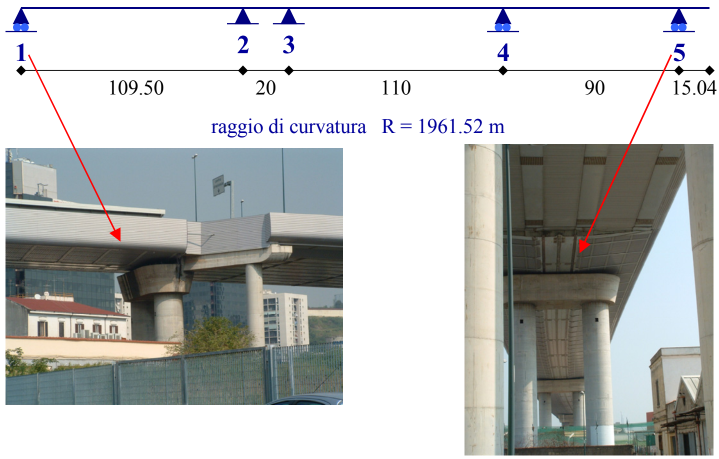


Figure 4: Structural scheme of the Long Span Bridge.

Figure 5: Views of the deck and the main girder.

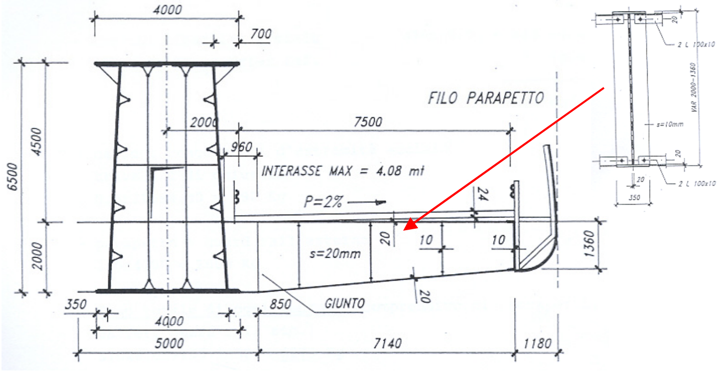


Figure 6: Cross section of the bridge deck.

A Finite Element complete bi-dimensional model of the bridge has been developed by using beam elements (Figure 8). Table 1 shows the first 10 periods and mode shapes of the structure. Figures 8 and 9 show the first global eigenform of the bridge in the longitudinal direction and the 1st local flexural mode of pier 4. It is worth to note that the fixed piers represent a kind of clamped restraint for the two parts in which they divide the deck, because of the small length of the span 2–3: the 4th and 5th periods correspond, respectively, to the first flexural modes of the 3rd + 4th span (Figure 10) and of the 1st span (Figure 11).

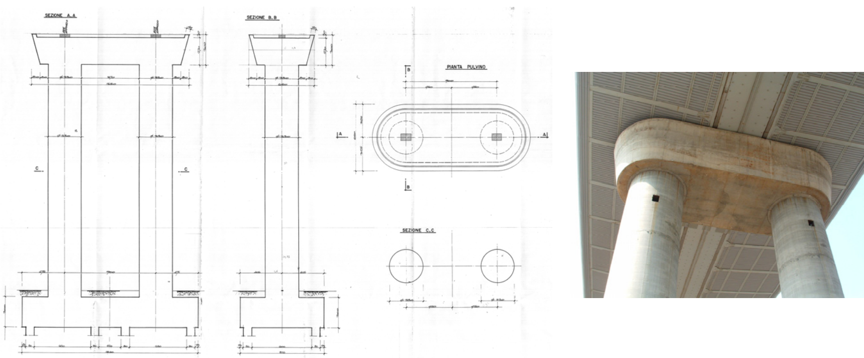


Figure 7: Characteristics of the piers.

Table 1: Dynamic characteristics of the bridge.

MODE	PERIOD [s]	MODE SHAPE
1	2.30	1st global mode in longitudinal direction
2	1.38	1st local flexural mode of pier 5
3	1.18	1st local flexural mode of pier 4
4	0.797	1st local flexural mode of 3rd + 4th span
5	0.787	1st local flexural mode of 1st span
6	0.738	1st local flexural mode of pier 1
7	0.537	2nd local flexural mode of 3rd + 4th span
8	0.308	2nd global mode in longitudinal direction
9	0.266	2nd local flexural mode of 1st span
10	0.262	3rd local flexural mode of 3rd + 4th span

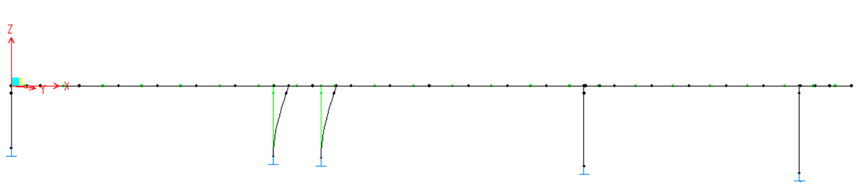


Figure 8: Finite element complete model and 1st mode – 2.30 s.

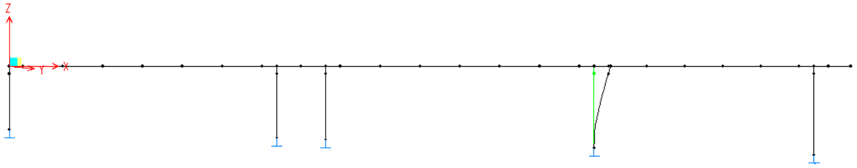


Figure 9: 3rd eigenform – 1.18 s.

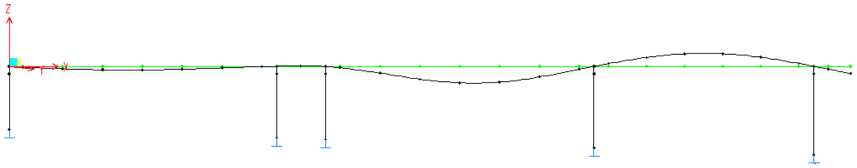


Figure 10: 4th eigenform – 0.80 s.

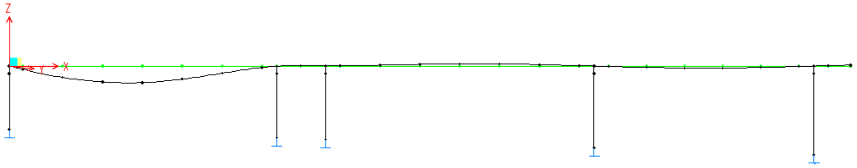


Figure 11: 5th eigenform – 0.79 s.

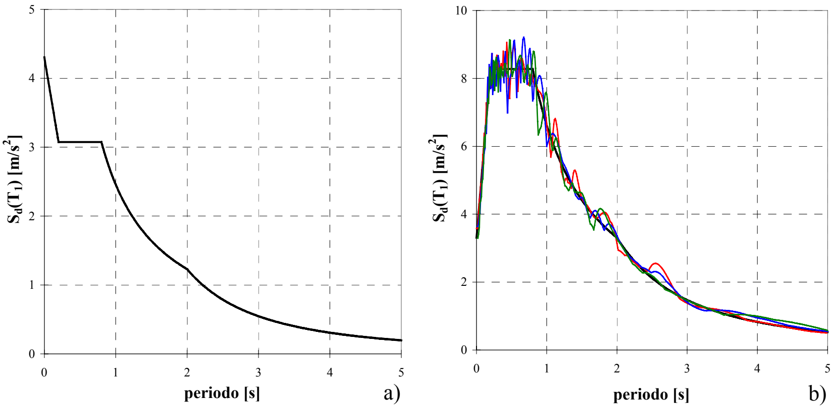


Figure 12: (a) Design response spectrum, (b) elastic response spectrum.

The structure was constructed approximately ten years ago, when an older Italian seismic classification and code was in force. In the proposed paper the recently approved new seismic classification and code for earthquake resistant structures [1] has been taken into account. A Response Spectrum Analysis on the existing bridge, considering the first 10 modes of the structure, has been performed with the design response spectrum provided by the new code (Figure 12a), obtained considering a design ground acceleration equal to $a_g = 0.25\,g = 2.45\,m/s^2$ (Napoli belongs to Zone 2 in the new Italian classification), a soil factor $S = 1.35$ which takes into account that the ground of the upper 15 m corresponds to the stratigraphic profile of type D (deposits of loose-to-medium cohesionless soil or of predominantly soft-to-firm cohesive soil), a behaviour factor q assumed equal to 3.5 (bridges having piers with flexural behaviour: H/L ≥ 3.5), and amplified by the importance factor $\gamma_I = 1.3$ corresponding to the bridges belonging to the I category (bridges of critical importance). The spectral acceleration corresponding to the fundamental mode of the bridge is equal to $0.93\mathrm{m/s^2}$ and the maximum values of shear forces and bending moments in the fixed piers are $T_{p,\max} = 7.19 \cdot 10^3$ kN and $M_{p,\max} = 2.08 \cdot 10^5\ \mathrm{kN \cdot m}$, respectively. The latter value is much higher than the flexural resistance at the base of the piers $M_{p,res} = 0.74 \cdot 10^5\ \mathrm{kN \cdot m}$ taking into account the interaction of the acting axial force $N_p = 360\,\mathrm{kN}$. The maximum horizontal displacement of the deck is of approximately 60 cm. In order to reduce bridge response in terms of displacements and stresses, it has been proposed to include linear viscous devices between the top of each free pier and the deck (a total of 3 dampers are introduced). A simplified SDOF model, like the one in Figure 1b, has been considered to determine the optimal viscous damping parameter: $k_f = 83941\,\mathrm{kN/m}$ is the global stiffness of 2 + 3 piers assembly, $k_b = 115304\,\mathrm{kN/m}$ is the global stiffness of 1 + 4 + 5 piers assembly, $m_f = 9177\,\mathrm{t}$ is the total mass of the deck, and C_d is the global viscous damping parameter of the 3 linear viscous dampers. For the relative stiffness $\kappa = 0.728$ of the considered bridge structure, the design procedure shown in the previous section provides an optimal value $\nu_{opt} = 0.3432$. This optimal ratio corresponds to the value $C_{di,opt} = C_{d,opt}/3 = 6351\,\mathrm{kN/(m/s)}$ to be assigned to the viscous damping coefficient of each control device.

4 Numerical investigation

In order to verify the adequacy of the behaviour factor q, non-linear time history analyses should be performed taking into account the non-linear behaviour of the critical structural cross-sections. However, in order only to determine the reduction of response caused by the insertion of linear viscous devices on top of the longitudinally free piers, linear time domain analyses have been carried out. An ensemble of three artificial accelerograms have been generated so as to match the elastic response spectrum at the site ($a_g = 0.25\,g$, soil type D) for 5% viscous damping (Figure 12b). As required by the code, it has been checked that, with a deficiency tolerance of 10%, the elastic response spectrum and the mean value of the spectral ordinates computed for the considered accelerograms on the interval of periods $0.15\text{s} \div 2T$ (T is the fundamental period of the bridge) and for an equivalent viscous damping coefficient of 5% are the same. A duration of 20 s has been assumed for the accelerograms, with a duration of the stationary part equal to 10 s.

The following figures summarize some relevant results of the numerical analyses performed on the complete bi-dimensional model of the structure, subjected to the generated artificial accelerograms amplified by the importance factor $\gamma_I = 1.3$. Figure 13 shows the displacement x_f and acceleration $\ddot{x}_f$ of the deck, when one of the artificial seismic input is acting: the response of the bridge in the actual configuration ($C_{di} = 0$) is compared with the one in the retrofitted configuration including passive linear viscous devices. The viscous damping parameter C_{di} is assumed equal to the optimal value $6351\,\text{kN}/(\text{m/s})$ corresponding to the case of a harmonic displacement base input. Figure 14 clearly indicates that this optimal value falls within a range of viscous damping values minimizing the response of the structure in terms of deck displacement x_f.

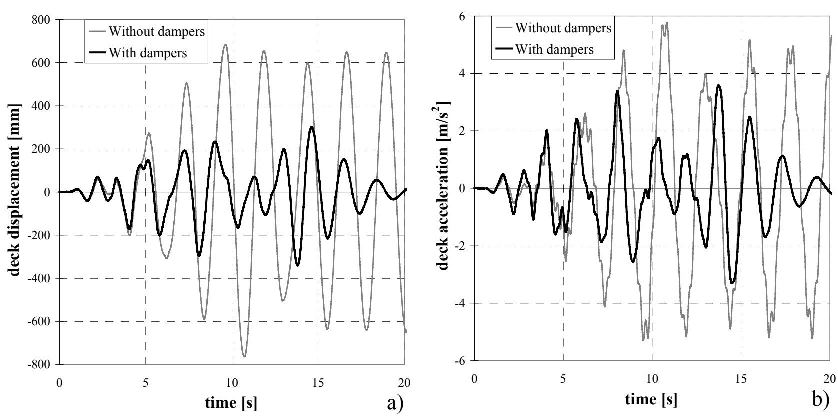


Figure 13: Comparison in terms of (a) deck displacement, (b) deck acceleration.

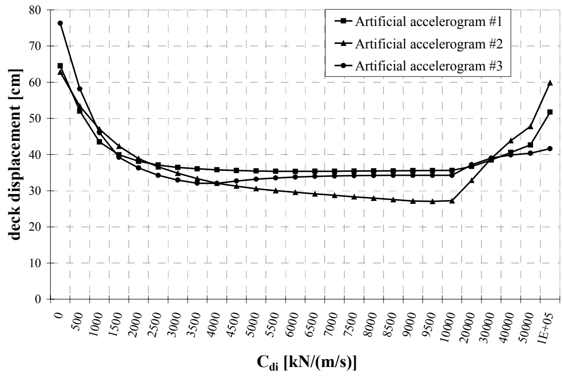


Figure 14: (a) Design response spectrum, (b) elastic response spectrum.

5 Conclusions

In several existing bridges the continuous deck is connected to some piers through fixed bearings, and to others through bearings which allow sliding in the longitudinal direction. In this way, a seismic action on the whole deck is transferred to the fixed piers only, so that the insertion of passive viscous energy dissipation devices between the bridge deck and the longitudinally free piers represents a solution to reduce deck displacements and fixed piers stresses. In this paper, for the above particular class of bridges, a completely new design procedure has been presented. The proposed method is based on the analytical determination of the response in the frequency domain to a harmonic base motion, and the computation of the optimal values of the viscous damping constant, varying the fixed piers/free piers relative stiffness. Linear time domain analyses have been carried out on an Italian existing bridge, in order to determine the reduction of seismic response caused by the insertion of linear viscous devices on top of the longitudinally free piers. Besides, it is demonstrated that the optimal value of the viscous damping parameter obtained by the above procedure falls within a range of viscous damping values minimizing the response of the bridge to a seismic input in terms of deck displacement

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