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PERFORMANCES OF A CLASS OF ISOLATION SYSTEMS FOR THE PROTECTION OF BRIDGES AND VIADUCTS IN SEISMIC AREAS

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SUMMARY

Three different isolation systems for bridges and viaducts are examined. All of them exploit steel-PTFE sliding bearings to support the weight of the deck and a mechanism, based on different technologies and materials (i.e. rubber, steel and shape memory alloys), to provide re-centring and/or additional energy absorbing capability.

A series of numerical analyses have been carried out, in order to: (1) assess the reliability of different force-based and displacement-based design approaches, (2) compare the performances of different types of isolation systems, (3) evaluate the effect of friction variability on the structural response and (4) identify the variation of response indices with changes in the ground motion, bridge and isolation characteristics. Reference was made to a pier-deck model, in which the pier is modelled through an elastic beam element with distributed mass and elasticity, while the effective mass of the deck is lumped. An accurate mathematical model of the frictional behaviour of steel-PTFE sliding bearings has been developed and calibrated, based on the results of previous experimental studies carried out by the authors.

Six different configurations have been considered for each type of isolation system, in accordance with different performance objectives and design spectra. Each numerical simulation has been then carried out with reference to a specific value of bearing pressure (9.36MPa, 18.72MPa and 28.1MPa) and air temperature (-10°C, 20°C and 50°C). Thus, a total of nine analysis cases have been considered for each bridge configuration. The variability of the structural response with the state of lubrication of steel-PTFE interfaces and pier height has been also evaluated.

Both artificial and natural seismic excitations were used in the non-linear dynamic analysis of the isolated bridge. Near-fault records of three historical earthquakes have been selected as input ground motions. In addition, eight artificial acceleration profiles, compatible with the response spectra provided by the new Italian Seismic Code, for soil type A and D (PGA equal to 0.35g and 0.4725g, respectively), have been considered. In this paper the main results of this parametric study are discussed.

INTRODUCTION

There are several conditions that, alone or together, may lead to the decision of seismically isolating a bridge: (i) avoid brittle failure in some piers, (ii) avoid concentration of damage (hence ductility demands) in not-regular bridges (e.g. continuous bridges with piers of significantly different height), (iii) reduce spectral accelerations in very stiff piers, (iv) increase the energy dissipation capacity of the bridge in order

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to reduce strength and displacement demands, etc. Obviously, the first two conditions mainly apply in the retrofitting of existing bridges, while the others are better related to the design of new bridges.
In principle, the design of a seismically isolated bridge is simpler than the design of a conventional bridge, as all the structural members (with the obvious exception of the isolation system) can be assumed to behave elastically. On the other hand, this circumstance makes the q-factor approach [1], commonly applied for the design of conventional bridges, inadequate for the design of isolated bridges. Alternative design procedures are then needed.
When designing an isolated bridge, the bridge geometry and the pier/deck sections are usually known, as they result from non-seismic load conditions. Thus, the pier strength and the characteristics of the isolation system are the only variables. When dealing with the retrofitting of existing bridges, the pier reinforcement is also known and the characteristics of the isolation system become the only design variables.
Common practice for the design of isolated structures typically utilizes an equivalent visco-elastic model to account for the characteristics of the isolation system. The linearization of the isolation system is appealing, as it greatly simplifies the analysis. However, it has several limitations. First, the characteristics of the equivalent isolation system depend on the structural response. As a consequence, an iterative procedure should be used to get reliable results. Second, different linearization criteria exist [2]. Thus, a preliminary critical selection should be made, also taking into account the specific characteristics of the isolation system to be used. Finally, it should be observed that at the end of the design process, non-linear analyses are generally needed, in order to check the predicted response.
In this study a force-based and a displacement-based approach for the preliminary design of isolated bridges are proposed. They are based on the Capacity Spectrum Method, as defined in [3]. The proposed design method reduces the elastic response spectrum to intersect the Capacity Curve of the pier-isolator system and find the Performance Point (i.e. maximum force and displacement) of the isolation system. The performance objective of the design assumed in the force-based approach is to limit the expected maximum force transmitted by the isolation system to the pier under a proper fraction of the lowest strength (flexure, shear, footing) of the pier. The performance objective of the design assumed in the displacement-based approach is to limit the expected maximum displacement of the isolation system under a proper fraction of the ultimate displacement capacity of the device. Obviously, the force-based approach turns out to be particularly useful in the retrofitting of existing bridges, while the displacement-based approach better suits for the design of new bridges.
In the design phase, reference was made to the response spectra provided by the new Italian Seismic Code (ISC) [4] for soil type A and D, with intensity level corresponding to a return period of about 500 years (PGA equal to 0.35g and 0.4725g, respectively).
Three different isolation systems have been examined. All of them exploit steel-PTFE sliding bearings to support the weight of the deck and a mechanism, based on different technologies and materials (i.e. rubber, steel and shape memory alloys), to provide re-centring and/or additional energy absorbing capability.
A comprehensive parametric analysis has been carried out by SAP2000_NonLinear [5], in order to: (i) assess the reliability of the above mentioned performance-based design approaches and (ii) evaluate the variation of the bridge response with changes in the ground motion, bridge and isolation system characteristics. Particular attention in the numerical analysis has been devoted to the friction variability of the steel-PTFE sliding bearings with contact pressure, air temperature and state of lubrication of steel-Teflon interfaces. Both artificial and natural seismic excitations have been used in the non-linear analysis of the isolated bridge. Near-fault records of three historical earthquakes (Imperial Valley (18/5/1940), Michoacan (19/9/1985) and Great Hanshin-Awaji (17/1/1995)) have been selected as input ground motions. In addition, 8 artificial acceleration profiles, compatible with the two response spectra used in the design, have been considered. In this paper the main results of this parametric analysis are discussed.

ISOLATION SYSTEMS

Sliding isolation devices are particular attractive for the use in bridge applications. They reduce seismic force transmission to piers and abutments by limiting shear transfer across the sliders. Before slipping, the sliding isolation devices provide an effective restraint against movements due to non-seismic service loads (wind, traffic, etc.). Once their friction threshold is exceeded, the structural response is governed by the frictional characteristics of the sliding interfaces, which in turn are a function of velocity, contact pressure, air temperature and state of lubrication [6]. In any case, auxiliary recentering and/or dissipating devices are needed, in order to limit the maximum displacements experienced by the isolation systems during the earthquake and the residual displacements at the end of the seismic excitation.

In this study three different sliding isolation systems have been considered. All of them are based on the coupling between lubricated steel-teflon sliding bearings and an auxiliary device based on different technologies and materials, namely: (i) rubber, (ii) steel and (iii) shape memory alloys (SMA) [7].

All the components above mentioned (i.e. steel-PTFE sliding bearings, rubber- steel- and SMA-based auxiliary devices) have been extensively tested at the laboratory of the University of Basilicata, in previous studies [8], [9], [10], [11]. Based on the experimental mechanical behaviour exhibited by each component, accurate mathematical models, for conditions of interest in seismic isolation, have been implemented. A brief description of these models is reported in the following paragraph.

Figure 1 compares the experimental and numerical force-displacement relationships of each device. All the experimental curves shown in figure 1 have been obtained from sinusoidal cyclic tests at frequency of loading ranging from 0.3Hz to 0.5Hz. As regards the sliding device, the test shown in figure 1 has been carried out on pure steel-PTFE interfaces, under a normal load of 20 kN (i.e. 18.72 MPa contact pressure) and air temperature of 20 °C. The friction coefficient at the peak velocity is equal to about 12.5%. It reduces to less than 3% for lubricated interfaces.

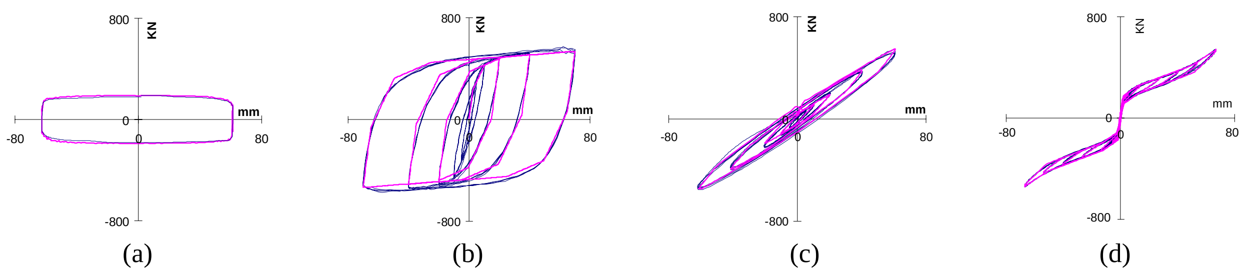


Figure 1. Comparison between experimental (thin line) and numerical (thick line) force-displacement relationships for (a) pure steel-PTFE sliding bearings, (b) rubber-, (c) steel- and (d) SMA-based auxiliary devices.

The great accuracy of the numerical models in capturing the actual behaviour of each device is apparent in figure 1. To be also noted the differences in the mechanical behaviour of the three auxiliary devices, in particular with regard to their recentering and energy dissipating capability.

Another major concern is the sensitivity of each device to temperature variations. The friction coefficient of PTFE-steel sliding interfaces decreases while increasing temperature [8]. Steel is practically insensitive to temperature variations. Rubber experiences a reduction in stiffness and energy dissipation capacity while increasing temperature [12]. SMA, instead, shows an opposite trend [11]. All that has been properly taken into account in the construction of each numerical model, as explained below.

NUMERICAL MODEL

Only bridges constituted by vertical piers connected by a horizontal deck are considered in this paper. Moreover, the attention is focused on the transverse response of the bridge, which is in most cases more

critical than the longitudinal response. The effects of different soil conditions under the piers, as well as of non-synchronous motion, are neglected.
If the coupling offered by the deck is small (e.g. for regular bridge geometry), each pier-isolator system can be regarded as an independent system and analysed separately, by taking into account the right amount of deck mass. Figure 2(a) shows the reference structure selected for the analysis. It is perfectly symmetric, with piers having same height and cross section. In the basic configuration, the bridge is characterised by piers of 10m height and hollow circular cross section with 4m external diameter and 0.35m thickness. The weight of the deck supported by each pier (see figure 2(a)) has been taken equal to 1145 ton.

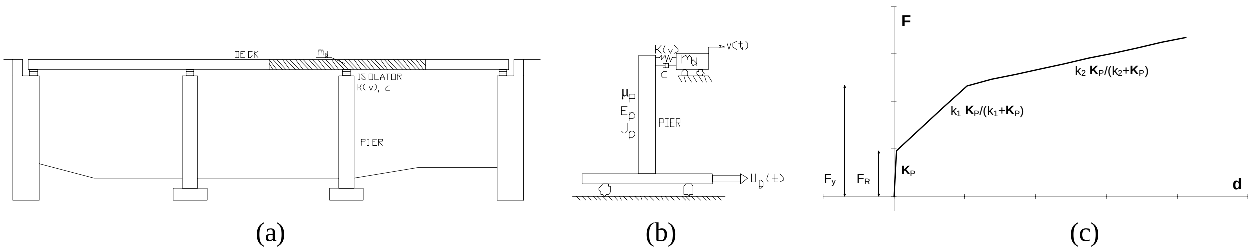


Figure 2. (a) Reference structure selected for the analysis, (b) simplified analytical model and (c) idealization of the monotonic response of the pier-isolator system.

In the numerical analysis, reference was made to the simplified model shown in figure 2(b), in which the pier is modeled through an elastic beam element with distributed mass and elasticity, while the effective mass of the deck (calculated according to a tributary principle) is lumped. To account for concrete cracking, the flexural rigidity of the pier has been reduced by 30%. A viscous damping ratio equal to 5% was adopted for the soil-foundation-pier system. Figure 2(c) shows the idealized monotonic behaviour of the pier-isolator system. Before slipping in the sliding bearings occurs, the response of the system is elastic with lateral stiffness equal to the non-isolated pier K_p. Once the sliders are activated, the effective tangent stiffness of the overall system reduces to $(k_1 \cdot K_p)/(k_1 + K_p)$, being k_1 the initial stiffness of the auxiliary device. When the auxiliary device activates, the effective tangent stiffness of the overall system reduces to $(k_2 \cdot K_p)/(k_2 + K_p)$, being k2 the post-yield stiffness of the auxiliary device.
As said before, great care has been used in modelling the mechanical behaviour of each isolation system. The frictional behaviour of steel-PTFE sliding bearings, in particular, has been described by means of the modified viscoplastic model by Constatinou et al. [13], as it was available in the structural analysis program SAP-2000 Nonlinear [5]. According to Constantinou et al., the coefficient of friction μ at sliding velocity v, can be approximated by the following equation:

$$\mu = \mu_{max} - \left(\mu_{max} - \mu_{min}\right) \cdot e^{-\alpha \cdot v} \tag{1}$$

in which μ_{max} is the coefficient of friction at high velocities, μ_{min} is the coefficient of friction at very low velocities and α is a function of bearing pressure and air temperature. The frictional resistance is then assumed equal to:

$$F = \mu \cdot W \cdot Z \tag{2}$$

where W is the vertical load, while Z is a dimensionless hysteretic quantity which can be calculated by solving the well-known differential equation proposed by Wen in random vibration studies of hysteretic systems [14]. In this study, μ_{max} , μ_{min} and α have been analytically expressed as a function of pressure and temperature, through a second-order polynomial model, whose coefficients have been obtained by a multivariate nonlinear regression with SPSS [15], based on the results of previous experimental studies carried out by the authors on pure and lubricated steel-PTFE interfaces [8]. During these tests, three different values of contact pressure (i.e. 9.36MPa, 18.72MPa and 28.1MPa) and air temperature (i.e. -10°C,

20°C and 50°C) were considered. The same parameters have been selected for the numerical analysis described in this paper.
As far as the mechanical behavior of the auxiliary devices is concerned, reference was made to the experimental results of previous cyclic tests carried out at the University of Basilicata [9], [10], [11]. The experimental force-displacement relationships of each auxiliary device have then been simulated through a combination of elasto-plastic, linear elastic, nonlinear elastic and viscous elements, as shown in figure 1. Temperature was one of the main parameters considered in the numerical investigation. To account for the sensitivity of the mechanical behaviour of SMA-based devices to temperature variations, reference was made to the experimental evidence [11], which pointed out a linear dependence on temperature in the range 0°C ÷ 50 °C, with increases of force of the order of 10% every 10 °C, and decreases of equivalent damping the order of 13% every 10 °C. As far as rubber-based devices are concerned, reference was made to [12], which provides detailed information on the effects of extreme temperatures (-20°C and 40°C, specifically) on effective stiffness and effective damping of rubber isolators.

DESIGN OF ISOLATION SYSTEMS

In this study, two different procedures (a force-based and a displacement-based procedure, precisely) have been followed in the design of the isolation systems. Both are based on the Capacity Spectrum Method, as defined in [3].
The Capacity Spectrum Method is a nonlinear static procedure that compares the global force-displacement capacity curve of the structure with the response spectrum representation of the earthquake demand to obtain a performance point, i.e. an estimate of the maximum expected response of the structure during the ground motion.
For the structural system under examination, the capacity curve is given by the three-linear force-displacement relationship shown in figure 2(c). Two different demand curves were considered, corresponding to the response spectra provided by the new Italian Seismic Code (ISC) [4] for soil type A and D, with PGA equal to 0.35g and 0.4725g, respectively. Obviously, to use the Capacity Spectrum Method it was necessary to convert the capacity curve and the demand curve in the so called ADRS (Acceleration-Displacement-Response-Spectra) format, i.e. S_a versus S_d. The required equations to make this transformation are given in [3].
The location of the performance point must satisfy three conditions: it must (i) lie on the capacity spectrum curve, in order to represent the structural response at a given displacement, (ii) lie on a spectral demand curve, reduced from the 5%-damped spectrum, that represent the nonlinear demand at the same structural displacement and (iii) satisfy the performance objective of the design, which is expressed through a condition on the maximum force or maximum displacement of the isolation system.
The reduction factors of the demand curve are given in terms of equivalent damping (ξ_{eq}), which can be viewed as a combination of the inherent viscous damping of the soil-pier system (ξ_o = 0.05) and the effective hysteretic damping of the isolation system (ξ_{ef}). This latter is a function of the shape of the capacity curve (figure 2(c)) and of the estimated maximum response of the isolation system. It can be calculated through the well known equation:

$$\xi_{ef} = \frac{1}{4\pi} \cdot \frac{W_D}{W_S} \qquad (3)$$

where W_D is the energy dissipated by the isolation system in a single cycle of motion at the maximum expected displacement, while W_S is the associated maximum strain energy. The (global) equivalent damping can be estimated as follows [16]:

$$\xi_{eq} = \frac{D_P \cdot \xi_o + D_I \cdot \xi_{ef}}{D_P + D_I} \tag{4}$$

where D_P and D_I are the maximum expected displacements in the pier and in the isolation system, respectively.

The relation suggested in [4] for damping values different from 5% (i.e. $\eta = \sqrt{10/(5+\xi_{eq})}$), was adopted to calculate the reduction factors of the demand curve as a function of the equivalent damping of the soil-pier-isolator system. Obviously, being the equivalent damping a function of the maximum expected response of the isolation system, the determination of the performance point required a trial and error search, in order to satisfy the three criteria specified above. At this end, an iterative procedure was implemented into a spreadsheet.

The performance objectives assumed in the two different design approaches are as follows. In the force-based design approach, in particular, the performance objectives was to limit the maximum force transmitted by the isolation system to a fraction of the yielding strength of the pier. To estimate the yielding strength of the pier, reference was made to the maximum shear force obtained from spectral analysis of the response of the non-isolated bridge, by assuming a behaviour factor (q-factor, according to [1]) equal to 3.5. A further protection factor was then applied, in order to take into account possible over strength of the isolation system or larger than expected displacement demand. Practically, the strength of the device at the expected maximum displacement was taken equal to 55% of the design nominal strength of the pier. As regards the friction characteristics of the steel-PTFE sliding bearings, reference was made to the lower selected values of contact pressure and air temperature (i.e. 9.36 MPa and -10 °C, respectively), in order to maximize the force transmitted to the pier.

In the displacement-based design approach, the performance objective was to limit the expected maximum displacement of the isolation system to a fraction of the ultimate displacement capacity of the device. However, the selection of such a limit was also affected by the peak values of the ground displacement for high damping levels. As a matter of fact, the iterative procedure does not converge if the target displacement is greater than the peak ground displacement. Thus, the maximum allowable displacement of the isolation systems was directly taken equal to the peak value of the ground displacement (i.e. $d_g = 0.025 \cdot T_C \cdot T_D \cdot PGA$, according to [4]). As regards the friction characteristics of the steel-PTFE sliding bearings, reference was made to the higher selected values of contact pressure and air temperature (i.e. 28.1 MPa and 50 °C, respectively), in order to maximize the pier-deck displacement.

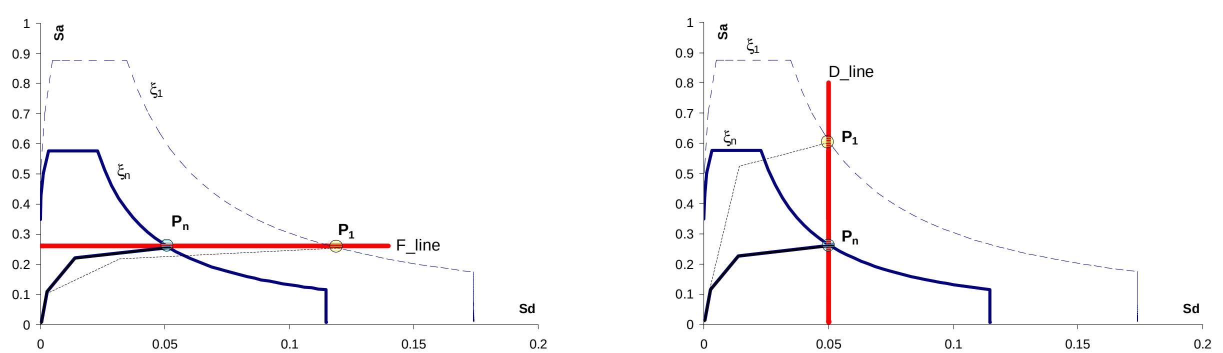


Figure 3. Schematization of the force-based (left) and displacement-based (right) design procedures used for the isolation systems, as derived from the Capacity Spectrum Method.

Figure 3 summarises the capacity spectrum procedure used in this study to design the isolation systems. F-line and D-line point out the target force and displacement, respectively. The demand spectra reported with dashed line correspond to the first step of the procedure (ξ = 5%). The demand spectrum reported with continuous thick line correspond to the final step of the procedure. P_n is the performance point, i.e. the expected maximum response of the system.

In table 1 and 2 there are summarised some results of the two design procedures, for each isolation system and soil condition, but for two temperature cases only. Besides the maximum expected force (F_{design}) and displacement (D_{design}), there are reported the effective period (T_{ef}) and effective damping (ξ_{ef}). The maximum response predicted in the design phase is compared to that obtained from nonlinear dynamic analysis, as better described in the following paragraphs.

Table 1. Force-based design approach: comparison of the results of preliminary design and nonlinear analysis for each isolation system and soil condition (H = 10m, T = -10 °C, p = 9.36 MPa).

system	soil	DESIGN				NON LINEAR ANALYSIS (average on 4 accelerograms)					
		F (KN)	*D (mm)*	*T_{ef} (sec)*	*ξ_{ef} (%)*	*F (KN)*	*RMS_F (%)*	*error_F (%)*	*D (mm)*	*RMS_D (%)*	error_D (%)
steel	*A*	**760**	44	1.14	46	734	1	-3	47	11	7
	D	**1027**	206	2.12	47	926	1	-10	144	6	-30
rubber	*A*	**760**	82	1.55	22	651	6	-14	65	9	-21
	D	**1027**	304	2.58	19	901	3	-12	234	5	-23
sma	*A*	**760**	104	1.75	17	668	3	-12	78	9	-25
	D	**1027**	348	2.76	13	1002	4	-2	331	10	-5

Table 2. Displacement-based design approach: comparison of the results of preliminary design and nonlinear analysis for each isolation system and soil condition (H = 10m, T = 50 °C, p = 28.1 MPa).

system	soil	DESIGN				NON LINEAR ANALYSIS (average on 4 accelerograms)					
		F (KN)	*D (mm)*	*T_{ef} (sec)*	*ξ_{ef} (%)*	*F (KN)*	*RMS_F (%)*	*error_F (%)*	*D (mm)*	*RMS_D (%)*	error_D (%)
steel	*A*	418	**80**	2.07	50	401	1	-4	84	10	5
	D	1187	**220**	2.04	45	1036	2	-13	140	9	-36
rubber	*A*	1159	**80**	1.25	13	988	5	-15	66	7	-18
	D	3710	**220**	1.16	10	2635	3	-29	134	3	-39
sma	*A*	1969	**80**	0.96	6	1867	5	-5	73	9	-9
	D	6088	**220**	0.9	4	4633	7	-24	117	20	-47

SEISMIC INPUTS

Both artificial and natural seismic excitations have been used in the non-linear dynamic analysis of the isolated bridge. Near-fault records of three historical earthquakes (see figure 4) have been selected as input ground motions. In addition, eight artificial acceleration profiles, compatible with the 5%-damped response spectra used in the design procedure, have been considered.

In order to make possible a comparison between results relevant to different input motions, the natural records have been normalised according to two different criteria: (i) same PGA and (ii) same spectral acceleration at the effective period of vibration of each isolation system (i.e. T_{ef}). In both the cases, reference was made to the response spectrum provided by EC8 for seismic zone 1 with soil type A, hence PGA = 0.35g. Figure 5(a) shows the response spectra of the three natural records normalised with respect to PGA, while figure 5(b) shows an example of normalisation with respect to the spectral acceleration.

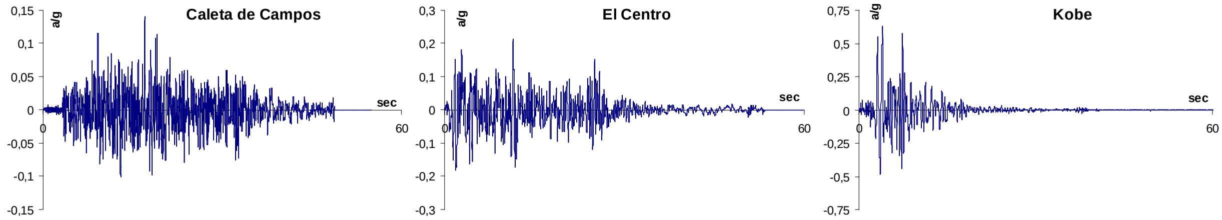
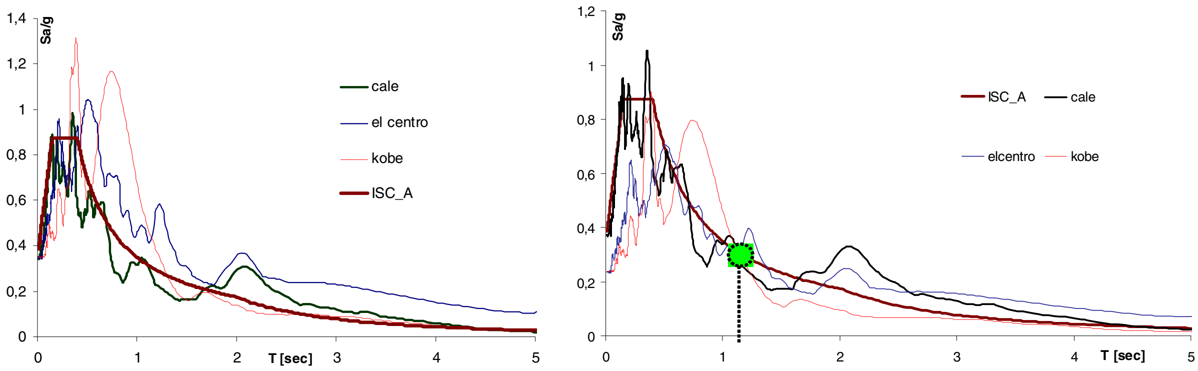


Figure 4. Acceleration profiles of the near-fault records of historical earthquakes used in numerical analyses.



(a) (b)

Figure 5. (a) Response spectra of the natural records normalised with respect to PGA, and (b) example of normalisation with respect to the spectral acceleration corresponding to the effective period of the isolation system.

NONLINEAR RESPONSE OF SEISMICALLY ISOLATED BRIDGES

Comprehensive numerical analyses have been carried out with SAP2000_Nonlinear [5] in order to: (1) assess the reliability of the proposed design procedure in predicting the maximum response of bridges equipped with strongly nonlinear isolation systems and (2) evaluate the variations of the structural response with changes in the ground motion, bridge and isolation system characteristics.
The fundamental structural parameters considered in this study are as follows: (i) the inherent characteristics of the auxiliary device, from highly dissipating to strongly recentering, (ii) the pier height, taken equal to 10m and 30m, (iii) the air temperature, taken equal to -10°C, 20°C and 50°C, (iv) the bearing pressure in the steel-PTFE sliding bearings, taken equal to 9.36MPa, 18.72MPa and 28.1MPa, and (v) the state of lubrication of the steel-PTFE interfaces. In the following, the structural response of the bridge is described through the force transmitted by the isolation system to the pier and the pier-deck displacement. Artificial and natural earthquakes are considered separately.

Structural response to artificial earthquakes

The main results of both preliminary design and non-linear analysis are compared in tables 1 and 2, for the bridge with piers of 10m height. In the force-based design approach (table 1), the air temperature and the contact pressure are taken equal to -10°C and 9.36 MPa, respectively. In the displacement-based approach (table 2), instead, the same two quantities are assumed equal to 50°C and 28.1 MPa, respectively. In both the cases, the PTFE-steel interfaces are supposed to be perfectly lubricated.
As can be seen, the bridge responds essentially as predicted in the design phase, with percent errors in the attainment of the performance objective of the same order of magnitude as the dispersion in the response, due to different input motions. The average errors reported in tables 1 and 2 are defined as percent differences between the average force/displacement obtained from the nonlinear analysis and the force/displacement predicted in the design. Similarly, the RMS (root mean square) of the response (over 4 accelerograms) is referred to the corresponding mean value and expressed in percent value. It can be observed that the proposed design procedure is more accurate in predicting maximum forces than maximum displacements. Moreover, it works better for stiff soils (type A) than for soft soils (type D). In any case, it is conservative, as it overestimates the maximum expected response. Actually, a certain overestimation is needed, to compensate the effects of temperature variations, as explained below. The accuracy of the proposed design procedure is substantially independent of the isolation type used.

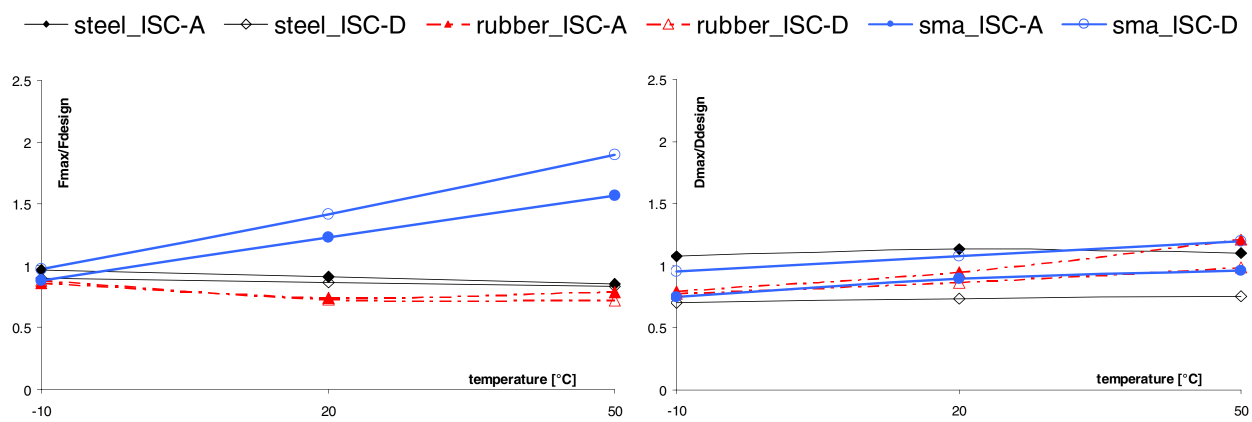


Figure 6. Bridge with piers of 10m height, equipped with isolation systems designed under force-control (T = -10°C, P = 9.36MPa). Sensitivity of the structural response to temperature variations (P = 9.36MPa): maximum force (left) and maximum displacement (right) normalised with respect to the corresponding design values.

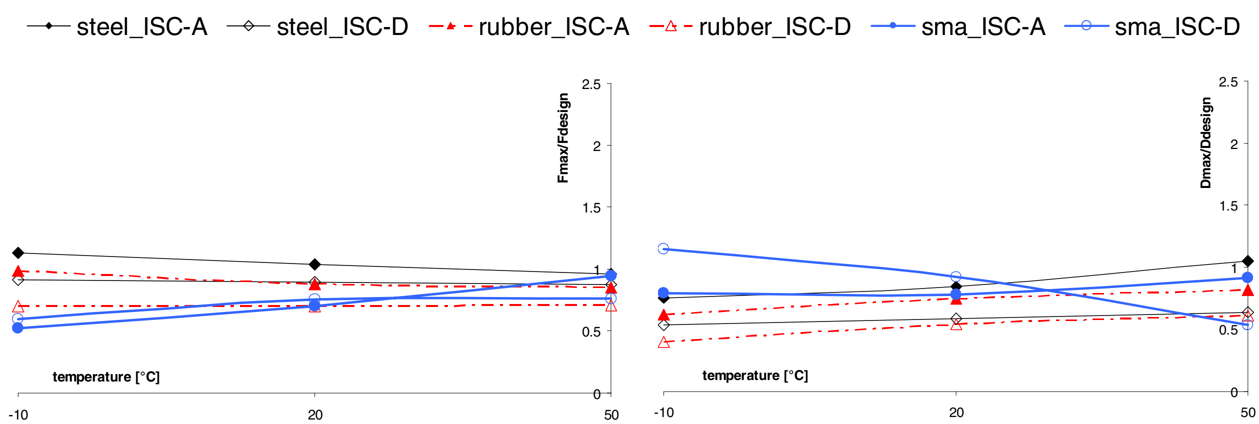


Figure 7. Bridge with piers of 10m height, equipped with isolation systems designed under displacement-control (T = 50°C, P = 28.1MPa). Sensitivity of the structural response to temperature variations (P = 28.1MPa): maximum force (left) and maximum displacement (right) normalised with respect to the corresponding design values.

Figures 6 and 7 show the sensitivity of the structural response to temperature variations, as obtained from nonlinear analyses. They refer to the same bridge configurations as table 1 and table 2, with the bearing pressure in the sliders at the design values (i.e. 9.36 and 28.1 MPa, respectively). The results relevant to different soil conditions are reported separately. In figures 6 and 7, the structural response obtained from nonlinear analyses (average over 4 accelerograms) is described through the maximum force transmitted by the isolation system to the pier (left) and the maximum pier-deck displacement (right). Both response indices are normalised with respect to the corresponding values obtained from the preliminary design.

As can be seen, the choice of a suitable design temperature is fundamental in order to obtain a structural response which satisfies the performance objective of the design over the whole operational range of temperature. This holds especially when also the auxiliary device (in addition to the sliding bearings) exhibits a mechanical behaviour which depends on temperature, like for rubber- and SMA-based device.

As far as the force-based design approach is concerned (figure 6), the assumption of referring to the lowest values of the temperature range (i.e. -10 °C) is correct for the steel isolation system, while it appears too much conservative for the rubber-based isolation system, for which the choice of an intermediate values (e.g. 20 °C) seems more suitable. For the SMA isolation system, finally, the need of referring to the highest value of the temperature range (e.g. 50 °C) is apparent.

As far as the displacement-based design approach is concerned (figure 7), the assumption of referring to the highest values of temperature (i.e. 50 °C) appears to be appropriate for the SMA-based isolation system, while it results too much conservative for the rubber- and steel-based isolation systems, for which the choice of an intermediate reference temperature (e.g. 20 °C) appears to be more suitable.
Further numerical analyses, however, are needed to optimise the design temperature.

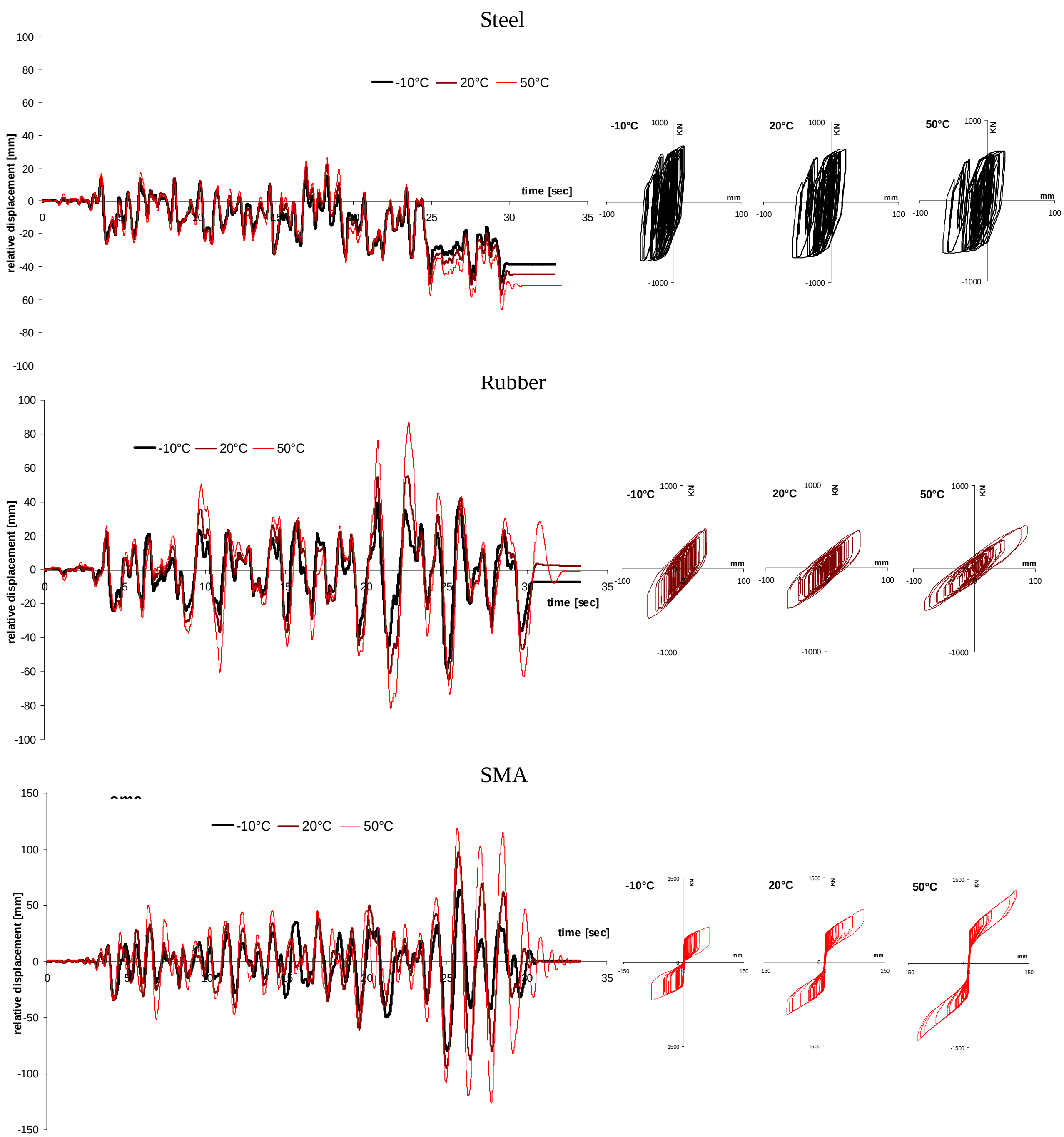


Figure 8. Sensitivity of the structural response to temperature variations: displacement-time histories and force-displacement relationships of the (a) steel-, (b) rubber- and (c) SMA-based isolation systems at -10 °C, 20 °C and 50 °C. Lubricated steel-PTFE sliding bearings, operating at 18.7MPa, are employed. Piers of 10m height.

As regards the variability of the structural response with temperature, the SMA-based isolation system performs worse, showing a higher sensitivity to temperature variations, especially in terms of maximum force transmitted to the piers. By assuming 20 °C as reference temperature and ±30 °C as thermal excursion, indeed, the maximum force variations range between 21% and 36% (depending on the design assumptions and soil conditions) for the SMA-based isolation system, between 2% and 20% for the rubber-based isolation system and, finally, between 3% and 9% only for the steel-based isolation system. Under the same conditions as before, the maximum displacement variations range between 12% and 42% for the SMA-based isolation system, between 14% and 28% for the rubber-based isolation system and, finally, between 4% and 21% for the steel-based isolation system.

Figure 8 clearly confirms that, by comparing the displacement-time histories and force-displacement relationships of the three isolation systems of table 1 (soil type A), at different air temperatures (namely -10°C, 20°C and 50°C). In figure 8 the contact pressure in the sliding bearings is taken equal to 18.72 MPa. The seismic input is represented by the first acceleration profile generated by the EC8 response spectrum for soil type A.

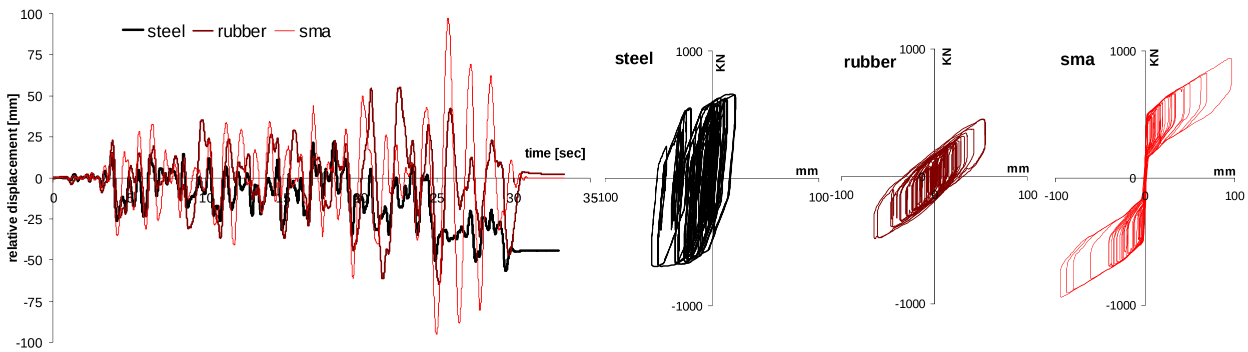


Figure 9. Comparison between the displacement-time histories and force-displacement relationships of the three types of isolation system, designed with reference to 760 kN threshold force (Pier height = 10m, bearing pressure = 18.72MPa, air temperature = 20°C, seismic input = acc No. 1 from EC8_A response spectrum).

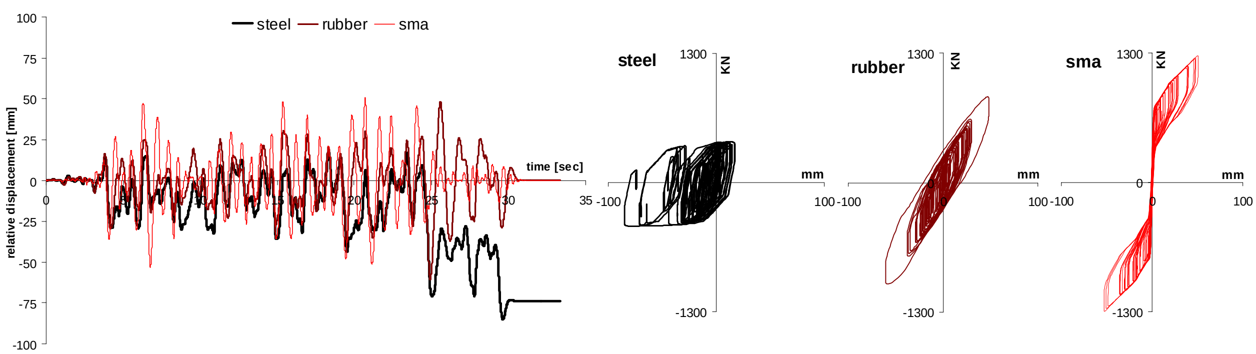


Figure 10. Comparison between the displacement-time histories and force-displacement relationships of the three types of isolation system, designed for a maximum displacement of 80 mm (Pier height = 10m, bearing pressure = 18.72MPa, air temperature = 20°C, seismic input = acc No. 1 from EC8_A response spectrum).

Figures 9 and 10 compare the displacement-time histories and force-displacement relationships of the three isolation systems, for the same bridge configuration (pier of 10m height), bearing pressure (18.72 MPa), air temperature (20 °C) and seismic input (the first artificial accelerogram generated from the EC8 response spectrum for soil type A). The isolation systems considered in figure 9 and 10 have been designed according to the force-based and the displacement-based design approach, respectively.

As can be seen, the choice of the best isolation system is not so easy, as each one presents some advantages and disadvantages with respect to the others.

The steel-based isolation system shows the best control of force when the performance objective of the design is to limit the maximum pier-deck displacement. On the other hand, it also exhibits a great scatter in the displacement response, with a large residual displacement at the end of the seismic action. This circumstance is absolutely undesirable because it should require very large expansion joints, which have maintenance problems, are costly, affect driving comfort and result in noise pollution.
The SMA-based isolation system suffer the large operational range of temperature, which implies high levels of force and low damping capacity. On the other hand, the SMA-based isolation system exhibits a strong supplemental recentering capability, which can be exploited to counteract possible increases in the friction resistance of the sliding bearings (e.g. due to grease deterioration) or to remedy installation imperfections (e.g. out of levels, etc.). It's worthwhile to observe that the purpose of the self-centering capability requirement for isolation systems is not so much that of limiting residual displacements at the end of the seismic action, as instead that of preventing cumulative displacements during the seismic events. Self-centering, therefore, assumes particular importance in structures located in proximity of a fault, where earthquakes characterised by highly asymmetric accelerograms are expected.
The rubber-based isolation system appears to be a good compromise between the previous two types of isolation system.
Figure 11 shows the maximum response of the bridge with piers of 30m height, equipped with isolation systems designed with reference to 760 kN threshold force. The bearing pressure in the sliding bearings is taken equal to 9.36 Mpa. As above, the maximum force transmitted to the piers and the maximum pier-deck displacement obtained from numerical analyses (average over 4 accelerograms) are normalised with respect to the corresponding values derived from the preliminary design at -10 °C and 9.36 MPa. Figure 11 confirms the great accuracy of the design procedure in predicting the maximum response of the isolated bridge, in terms of both forces and displacements, even when the deformability of the piers surely affects the dynamic response of the bridge. The only exception is represented by the steel-based isolation system, which shows an asymmetric response with (cumulative) maximum displacements much greater than expected.

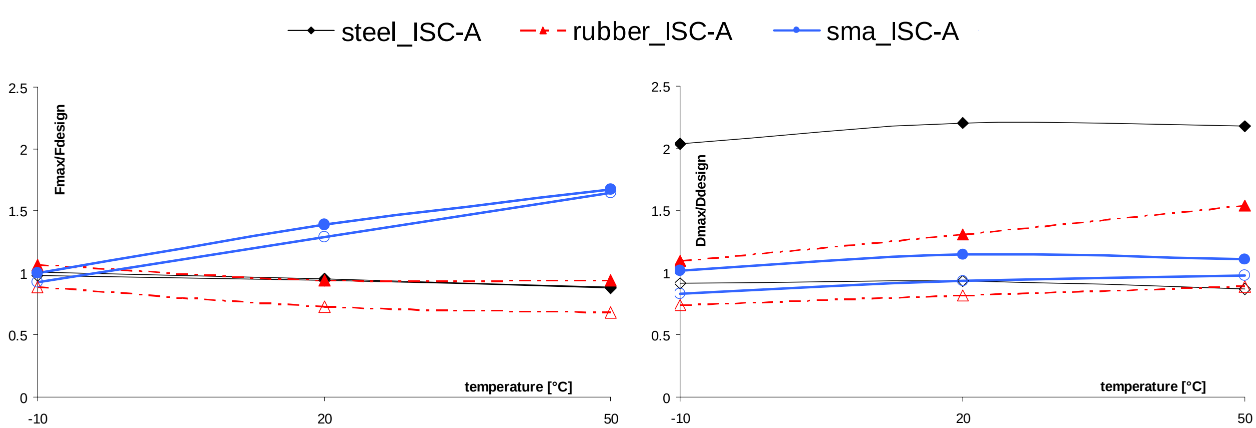


Figure 11. Bridge with piers of 30m height, equipped with isolation systems designed under force-control (T = -10°C, P =9.36 MPa). Sensitivity of the structural response to temperature variations (P = 9.36MPa): maximum force (left) and maximum displacement (right) normalised with respect to the corresponding design values.

Structural response to natural earthquakes

As said above, three different records of historical earthquakes have been considered in the numerical analyses, namely: (1) the record of El Centro (comp S90W) of the Imperial Valley Earthquake (18/5/1940), (2) the record of Caleta de Campos (comp N90E) of the Michoacan Earthquake (19/9/1985) and (3) the record of Kobe of the Great Hanshin-Awaji earthquake (17/1/1995). The selected acceleration profiles have been then normalised according to two different criteria: (i) same PGA (see figure 5(a)) and

(ii) same spectral acceleration at the effective period of vibration of each isolation system (see figure 5(b)). In both the cases, reference was made to the response spectrum provided by EC8 for seismic zone 1 and soil type A.

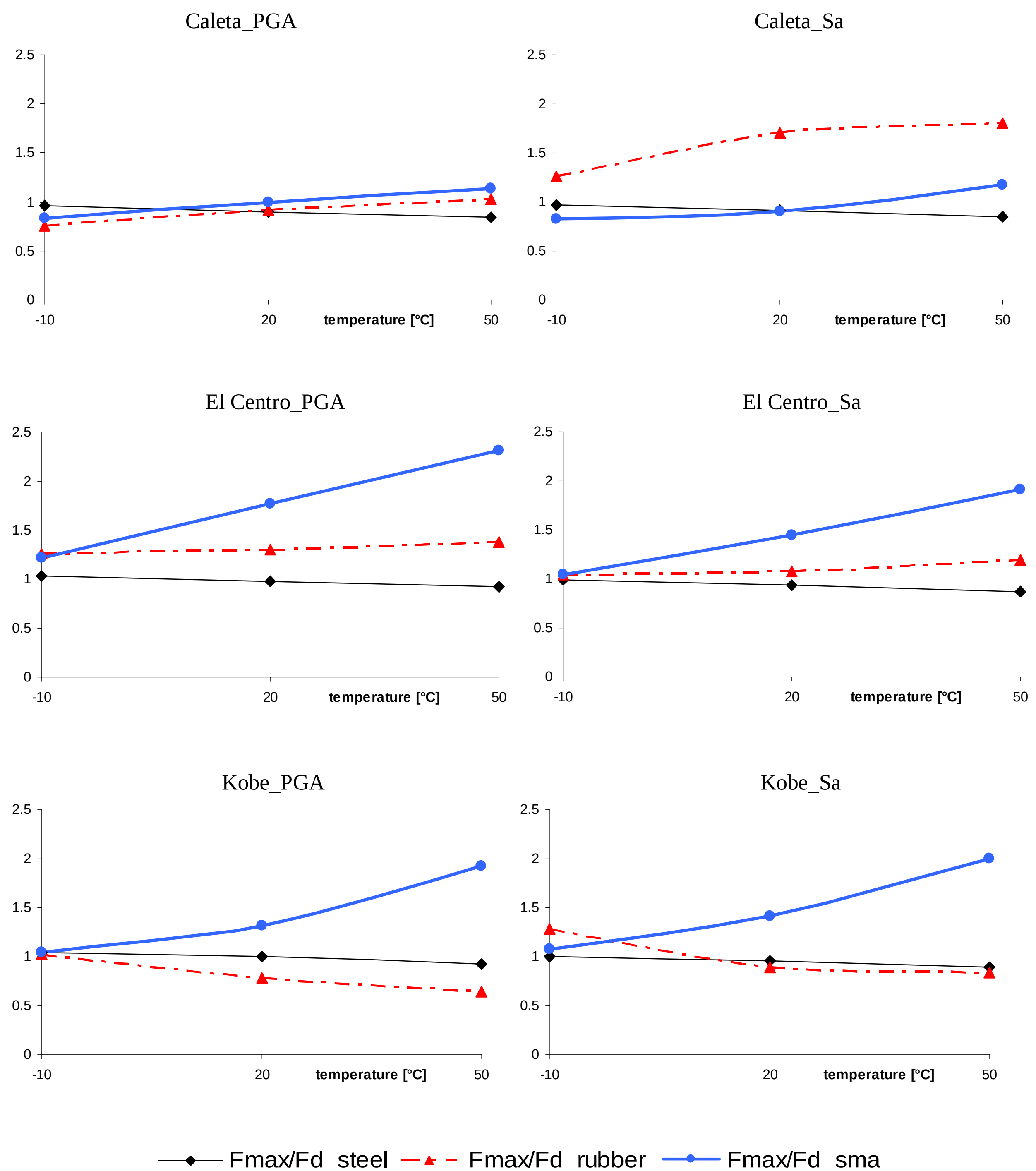


Figure 13. Isolation systems designed according to a force-based approach. Maximum force generated by the natural earthquakes in each isolation system, at different temperatures, normalised with respect to the threshold level assumed in the design phase.

The analyses with natural earthquakes were aimed at investigating the importance of the frequency content of the earthquake with reference to the structural response of bridges equipped with strongly non-linear isolation systems, designed according to both force-based and displacement-based performance approaches.
Figure 13, in particular, refers to the isolation systems designed according to the force-based approach. It shows, for each natural record and for both normalisation criteria, the ratio between the maximum force experienced by the three isolation systems, at different air temperatures, and the threshold force assumed in the design phase. Thus, values of the response index lower than or equal to 1 mean respect of the performance objective, while the contrary holds for values of the response index greater than 1.
As expected, the respect of the performance objective strongly depends on how the inherent nonlinear characteristics of the isolation system couples with the frequency content of the earthquake. Looking at the normalisation criterion with respect to PGA, it can be noted that for the Caleta de Campos earthquake all the isolation systems satisfy the performance objective, practically over the whole temperature range. For the Kobe earthquake, both steel- and rubber-based isolation system fully satisfy the performance objective over the whole temperature range, while the SMA-based isolation system does not. Finally, for the El Centro earthquake, only the steel-based isolation system always satisfies the performance objective. Looking at the normalisation criterion with respect to S_a, similar observations can be made. In any case, the respect (or not) of the performance objective largely depends on temperature. To this regard, the more penalised isolation system is that based on SMA, whose response, at the design temperature (i.e. -10 °C) lies on a valley of the response spectrum for both El Centro and Kobe earthquake. The same happens for the rubber-based isolation system with reference to the Caleta de Campos earthquake.

CONCLUSIONS

In this paper, bridge structures protected by strongly nonlinear isolation systems, based on the coupling between steel-PTFE sliding bearings and auxiliary re-centering and/or dissipating devices, are considered. A procedure for the preliminary design of such isolation systems is presented, which overcome the limitations of the equivalent linear idealisation and more explicitly acknowledge the nonlinear behaviour of the isolation systems. In particular, a force-based design approach is proposed for the case of existing bridges, while a displacement-based approach is proposed for the case of new bridges.
Comprehensive numerical analyses have been carried out by SAP2000_NonLinear, in order to assess the reliability of the above mentioned design procedure and to evaluate the sensitivity of the structural response to changes in ground motion, bridge and isolation system characteristics. In the numerical analysis, particular attention has been devoted to the variability of the frictional behaviour of the sliding bearings with contact pressure, air temperature and state of lubrication of steel-Teflon interfaces.
The results of the numerical analysis clearly proved the accuracy of the design procedure in predicting the maximum response of the structure. As a matter of fact, the perceptual errors in the attainment of the performance objective of the design resulted of the same order of magnitude as the dispersion in the response due to different input motions. However, the selection of a suitable design temperature is fundamental in order to obtain a structural response which satisfy the performance objective of the design over the whole operational range of temperature. The “optimal” design temperature appears to be strictly related to the inherent behaviour of the auxiliary device.
The structural idealisation assumed in the proposed design procedure has several implications, when thought to be applied to real bridge structures. First of all, highly irregular bridge configurations (e.g. bridge characterised by a continuous deck with piers of very different heights) surely affect the seismic response of the structure, to a greater or lesser degree. The effects of different soil conditions under the piers, as well as the effects of near-fault or non-synchronous ground motions, may also lead to significant modifications in the design force or displacement. Further studies are then needed to extend the proposed design methodology to cover a wider range of ground motion and structural situations.

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