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ON THE OPTIMAL DESIGN OF ENERGY DISSIPATION DEVICES FOR THE SEISMIC PROTECTION OF BRIDGES

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ABSTRACT

It is presented a methodology for the optimal design of energy dissipation devices for the seismic protection of typical bridge structures. The novelty of the work relies in the adoption of concepts of multiobjective minimization that allow to develop a rational formulation to account simultaneously for both the required maximum performance of the dissipative device and the minimum values of the bridge structural response. Due to the nonlinearity of the problem and the randomness of the seismic action, a numerical approach has been used and the results of the investigation are presented in terms of manageable design graphs which allow for some flexibility in the choice of the design parameters of the dissipative device, while respecting predefined constraints related to serviceability and limit states of the bridge and the device.

KEYWORDS

Multiobjective optimization, seismic protection, design charts, energy dissipation devices, bridge structures

INTRODUCTION

The recent years have seen a continuous growth in the development and application of protective systems for earthquake hazard mitigation (Buckle, 1993) and guidelines or appendix to seismic code have begin to be drafted in those countries where passive control systems have been widely utilized (Dolce, 1995). Moreover, state-of-the-art reports and review papers are now common as well as dedicated session at the conferences (Kelly, 1986, Buckle *et al.*, 1990, Dolce, 1995). Despite the above, in the guidelines a conservative tendency can be noted that is justified not only by the short experience on the actual behaviour of the structural systems, but also by the lack of available aids for a safe design (Buckle 1993). Therefore a general need exists to better (re)consider some problems that have not got to a satisfactory solution yet and a great effort in the research is spent to try to develop simplified design procedures often based on optimality concepts, although the complexity of the problem suggests to resort prefereably to linear systems or to invoke a linearized behaviour (e.g. Chang *et al.*,1993, Dowdell *et al.*, 1995, Inaudi *et al.*,1993, Scholl, 1993).

Since a clearly stated design methodology seems to be still lacking, the present work is aimed at providing a general framework for obtaining design graphs when account is to be made for the actual nonlinear behaviour of the protection devices. The suggested procedure is illustrated according to an oriented application to typical bridge structures endowed with energy dissipation devices and the novelty, with respect to similar attempts, should be preferably sought in the adopted formulation based on a multiobjective optimization. The motivation for the adoption of the said formulation is threefold: it allows to include in the objective function as many criteria as needed and therefore is appropriate when mutually conflicting criteria should be simultaneously satisfied; it further leads to minimal solutions that are curves rather than single points therefore leaving room for some flexibility in the design and, finally, it is sufficiently general to be naturally generalized to different and more complex protection schemes than those herein referred to.

The numerical application develops according to the "constraint method" so that a sequence of scalar minimizations, where the most preferred criterion is constrained to respect predefined restrictions related to the other criteria, have been conducted. Since the dissipative devices are the key elements in the protection strategy, the constitutive parameters of the energy dissipator have been chosen as design variables and a nondimensional index, which expresses the ratio of the energy dissipated by the device to the energy input by the earthquake, has been selected as the primary criterion to be fulfilled; the other "constraint" criteria have been related both to a limit state for the dissipative device and to the required structural response of the bridge. The adopted structural model accounts for the interaction between bridge deck and substructure, and for friction in the bearings. The EC8 indications have been followed to define the seismic input and a number of spectrum compatible accelerograms to get a statistically meaningful response have been generated accordingly and then used throughout the numerical analyses.

THE REFERENCE MODEL

As the primary target of the work is the formulation of a proper design methodology, it is convenient, initially, to keep the reference models for the structure and the seismic input as simple and standard as possible, yet retaining all the the relevant quantities of engineering interest, in order to focuse the attention on the procedure in itself rather than on the generality of the achievable results.

Structural Model

A widely used protection scheme for bridge structures, specially in Italy, is that where the deck is disconnected from the substructure, as in the case of simply supported or continuous span bridges, and the protection devices are located on top of the piers or confined at the abutements. In these cases and when the substructures do not contribute significantly to the overall bridge dynamic behaviour, the following differential equation, governing the motion of a simple oscillator, can be considered satisfatcory to reproduce the longitudinal bridge motion x induced by the seismic ground displacement x_g:

$$m\ddot{x} + c\dot{x} + F(x) = -m\ddot{x}_g \tag{1}$$

In eqn. (1) m is the deck mass, c is the linear damping constant and $F(x)$ is the restoring force which sums the force exerted by the device (F_d) and the friction force (F_f) arising from the conventional bearings necessary to transmit the vertical loads to the substructure. In view of the large number of numerical analyses to be performed, eqn. (1) has been given a nondimensional form obtained by dividing the right and left hand sides by the maximum absolute seismic force $m\ddot{x}_{g,max}$ so that the relevant quantities of the computed structural response can be made independent from the earthquake intensity. This latter is herein referred to as the product Cg where g is the gravity constant and C is the seismic zone index that, according to (EC8, 1989), attains the values $C = 0.15$, 0.25, 0.35 respectively for small, moderate and strong earthquakes. As concern the restoring force $F_d(x)$, that is the behaviour of the energy dissipation devices, a standard kinematic hardening behaviour, characteristic of elastoplastic type devices, which are the most common in Italy, is assumed; hence the behaviour of the entire model can be made dependent upon three parameters: the treshold force F_y, the initial elastic stiffness K_{el} and the secondary or elastoplastic stiffness K_{ep}; which are replaced in turn by the following normalized quantities: $\eta_y = F_y/m\ddot{x}_{g,max}$, $T = 2\pi(m/K_{el})^{1/2}$ and $\kappa = K_{ep}/K_{el}$.

Three sets of analyses have been carried out according to different values of the viscous damping ratio $\nu = c/2(K_{el} \cdot m)^{1/2}$ and the friction coefficient $\beta = F_f/w$, $w=mg$ being the deck weight. Initially, the conventional values ν=5% and β=0, still according to (EC8, 1989), have been considered, then the sensitivity of the results has been checked against a more realistic reduced value for the structural damping ν=2% and β=0, and against simultaneous presence of friction ν=2% and β=2%, where, according to (Mokha *et al.*, 1991), only the average sliding value has been considered disregarding the breakaway, that is the friction peak. The relevant engineering quantities which characterize the response of the model are given accordingly in nondimensional form; among them those needed in the sequel are: the maximum relative displacement $\zeta_{max} = x_{max}/\ddot{x}_{g,max}$ between the deck and the substructure, the kinematic ductility $\mu_c = x_{max}/(F_y/K_{el}) = x_{max}/x_y$ or an equivalent measure of the damage sustained by the dissipative device, the maximum force transmitted to the subsctructure $\eta_{max} = F_{d,max}/m\ddot{x}_{g,max} = \eta_y[1+\kappa(\mu_c-1)]$ and the supplemental hysteretic damping due to the dissipated energy E_h, which for a full cycle reversal is equal to $E_h=4F_y\, x_y(\mu_c-1)$ and do not depend on κ.

Ground Motion

A great attention should normally be devoted to the definition and modelling of the seismic action. In fact, the effectiveness of the design of a protective scheme depends strongly on the knowledge level of the earthquake expected at the site. Further, it is generally recognized that acceleration time histories artificially generated are unduly conservative and whenever possible it is prefereable to use actual seismic records to characterize the seismic action. However, as long as a detailed seismic zoning, capable to provide expected earthquake magnitude, source distance and site conditions, is lacking, it is prefereable to revert to the generation of artificial accelerograms compatible with coded spectra if some generality of the results is sought. Based on the above consideration, two families of pseudo-stationary accelerograms have been generated by means of the THGE program (Preumont, 1984) using the indications given in (EC8, 1989) for the characterization of the spectrum profiles. Two different profiles have been considered: soil type A and C representative respectively of "stiff" and "soft" soil conditions.

PRELIMINARY INVESTIGATIONS

Preliminary investigations have been carried out aiming at selecting, among the possible candidate functions, those characteristic quantities of the structural response more apt to be included in the objective function. The study has also permitted to establish practical engineering bounds for the feasible set of the design variables and the minimum number of accelerograms to employ in the nonlinear analyses in order to achieve a statistically meaningful structural response.

As concerns the statistical description of the structural response, two series of 30 accelerograms have been generated, one each for the two spectrum considered. Each series has then been ordered according to three different sequences and analyses have been carried out for the following groups 5-10-15-20-25-30 of each ordered sequence. The average values of the structural response, here undestood as the average of the peak absolute values, the standard deviation and their ratio have been computed and plots have been constructed to display the trend of these statistics against the increasing number of the accelerograms used. The results, given in (Ciampi *et al.*, 1995), show that 15 realizations are necessary to have a completely stable response, however 10 accelerograms can be considered sufficient for pratical purposes in view also of the savings in the computational effort.

As concerns the feasible set of the design variables $\mathbf{v}_p = [T, \kappa, \eta_y]^T$, that is the range outside which the characteristic quantities of the structural response would attain values beyond practical feasible ranges, it has been found to coincide with the interval: $\Omega \subset \Re^3 = [0.5 \leq T \leq 3.0$ sec; $0.03 \leq \eta_y \leq 0.45$; $0 \leq \kappa \leq 0.1]$. The motivations for the adopted bounds are the following (Ciampi *et al.*, 1995): the bounds on T avoid excess of ductility or displacement demand; lower values of η_y are limited by the braking force, whereas higher values result in an anti-economical use of the dissipative device that would not yield significantly; finally values greater than 0.1 for κ would lead to unreasonable large values of the force transmitted to the substructre in contrast with the protection philosophy.

As concerns the candidate quantities to be included in the objective function, the characteristic quantities of the structural response have been grouped into four homogeneous groups: static, kinematic, damage descriptors and energy quantities. The first two groups are of interest for the serviceability and limit state of the bridge, the third group characterize the low cycle fatigue of the dissipative device, whereas the fourth group is representative of the overall structural behaviour. The spectra of all the above quantities have been constructed and hierarchically ordered depending on their sensitivity against the variation of the design variables $\mathbf{v}_p$ and on the smoothness of their own variation. The quantities showing the uppemost/lowermost sensitiveness and more regular trends resulted in: η_{max}, ζ_{max} and ζ_{res} (residual displacement), μ_c and μ_h (hysteretic ductility), E_h and E_{ir} (relative input energy). Among them the energy quantities showed the best behaved thus being also the primary quantities to consider for insertion in the objective function.

OPTIMAL DESIGN

The approach followed attempts to solve a conflicting problem, searching the conditions for the maximum exploitation of the dissipative device, the key element in the protection strategy, associated with the minimum values of the bridge structural response and with the acceptable damage for the devices. To this end multiobjective optimization, that allows for a vector-valued objective function, is used since it offers the possibility to deal effectively with all the different, mutually conflicting, requirements inherent in the faced design problem. The motivations for using the above approach in the optimal design are multifold: the possibility to explore a broader range of alternatives than with conventional scalar minimization; a basis for explicit trade-off between conflicting objectives; the possibility to add in a natural way further criteria in the optimal design and more important the possibility to obtain not just a single solution point that does not give design flexibility, but rather to get a solution in the form of a so called minimal curve that offers actrative flexibility for design purposes. In fact, usually, there exist no unique point which would give an optimum for all the criteria simultaneously, thus a new optimality concept, than that of scalar optimization, should be introduced: a vector $\mathbf{v}_p^*$ is called a minimal solution if there exist no feasible vector $\mathbf{v}_p$ which would decrease some criterion without causing a simultaneous increase in at least one criterion.

Design Methodology

Several methods for solving nonlinear vector optimization have been presented in the literature (Duckstein, 1984). Usually the original problem is turned into a sequence of scalar optimization problems, which can be solved numerically by applying numerical techniques of nonlinear programming. Among the available methods, the "constraint method", capable to generate solutions even in non-convex cases, is here preferred, because fits naturally the empirical procedure one would follow in the absence of appropriate numerical tools, as will be clear in the next paragraph. In the "constraint method" the original vector optimization is replaced by:

$$\min_{\mathbf{v}_p \in \Omega \cap \Omega_k(\varepsilon)} f_k(\mathbf{v}_p) \tag{2}$$

where:

$$\Omega_k(\varepsilon) = \{ \mathbf{v}_p : \mathbf{v}_p \in \Re^3,\ f_i(\mathbf{v}_p) \leq \varepsilon_i,\ i \neq k \} \tag{3}$$

and $\varepsilon = [\varepsilon_1, \varepsilon_2 \ \ldots\ \varepsilon_{k-1}, \varepsilon_{k+1}, \ \ldots\ \varepsilon_m]^T$ is a vector of real numbers such that $\Omega_k(\varepsilon) \neq \varnothing$. The set of design variables collects the constitutive parameters of the dissipative device, as already said, whereas the vector objective function f: $\Re^3 \rightarrow \Re^4$ is chosen so as to collect one representative quantity for each of the four groups into which the characteristic quantities of the structural response have been divided:

$$\mathbf{v}_p = [T,\ \eta_y,\ \kappa]^T \in \Omega = [T_{min} \leq T \leq T_{max}\ ;\ \eta_{y,\,min} \leq \eta_y \leq \eta_{y,\,max}\ ;\ \kappa_{min} \leq \kappa \leq \kappa_{max}] \tag{4}$$

$$f = [f_k,\ \{f_i\}]^T = [-EDI,\ \{\eta_{max}, \zeta_{max}, \mu_c\}]^T \tag{5}$$

In eqn. (5) the nondimensional index $EDI = E_h/E_{ir}$ has been introduced to get a relative measure of the dissipated energy with respect to the energy input by the earthquake. Since the proposed target is to maximize the performance of the dissipative device, note the minus sign in eqn. (5), and since the results of the previous paragraph indicate the energy quantities as the most suited for the purpose, the *EDI* index is also selected as the preferred criterion for the optimization $f_k = -EDI$, whereas the other criteria $\{f_i\}$ play restrictions to the set of feasible solutions by keeping the structural response and the device damageability at minimum values.

Optimal Design Curves - Discussion of the Results

The following interpretation can be given to the above formulation: *EDI* is taken as scalar objective function while $\{\eta_{max}, \zeta_{max}, \mu_c\}$ are constrained by suitably chosen constants ε_i; by sistematic variation of these constants the entire minimal solution curve, that is the optimal design curve, can be obtained. This kind of approach is advantageous if a continuous monitoring of the solution is required. In fact, it is possible to start the procedure with a standard scalar minimization if large values for ε_i are selected, so that the criteria $\{\eta_{max}, \zeta_{max}, \mu_c\}$ do not initially affect the solution. Easy interpretable 2D plots of the solution can then be constructed by letting one of the $\{T, \eta_y, \kappa\}$ to act in turn as a parameter. It is found that these geometrical representations does not vary significantly with κ therefore, this latter design variable can be disregarded as long as only *EDI* is concerned and the *EDI* surface representation can be limited to the T-η_y subspace, fig. 1a.

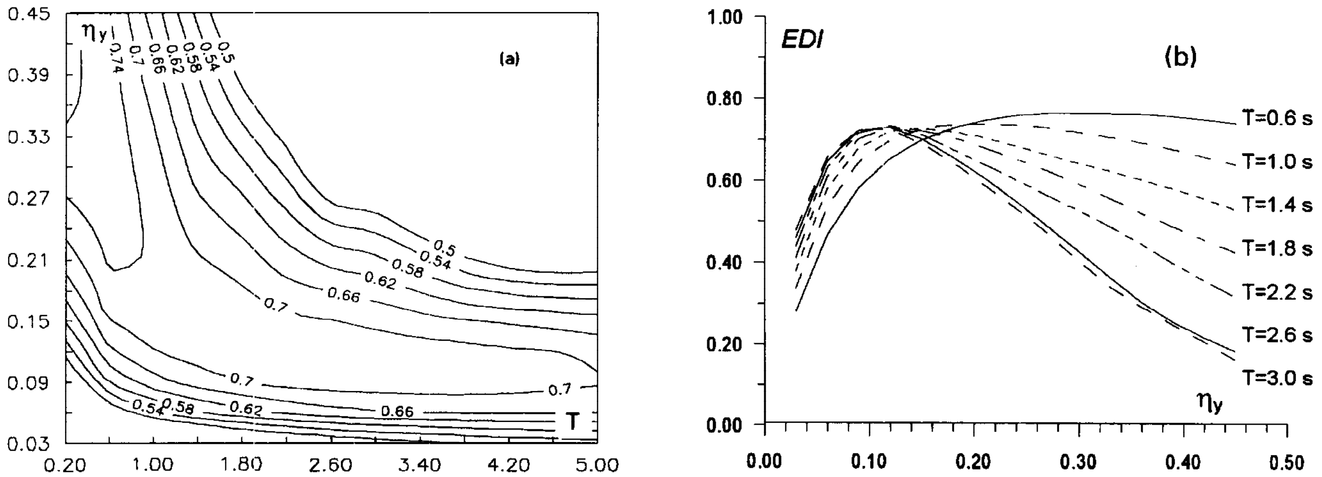

Fig. 1. *EDI* surface (spectrum profile C, ν=5%) - (a) contour line plot; (b) cross sections at *T*=ε.

This approach leads to a single minimum solution point $\mathbf{vp}^*\equiv(T=0.2,\ \eta_y=0.43,\ \kappa=\forall)$ with no freedom for the designer, fig. 1a; moreover, $\mathbf{vp}^*$ does not always provide for a satisfactory engineering solution even if it falls inside Ω. In order to increase the designer choices an optimal solution curve can be constructed proceeding exactly in the same way as for the "constraint method", but placing the constraints on the design variables. Usually it is preferred to start fixing K_{el}, or equivalently T, to account for the serviceability state of the bridge and then find η_y according to eqn. (2). This means that one is looking at the stationary points $\mathbf{vp}^*$ of the curves $EDI[\eta_y,\ T=\varepsilon,\ \kappa=\forall]$. These curves are plotted in fig. 1b for a discrete set of ε values. Now if the $\mathbf{vp}^*$ points are traced in the subspace $T\text{-}\eta_y \subset \Omega$ and the entire procedure is repeated for the two spectra (soil profile A and C) and the three different sets of damping and friction values considered, the optimal design graphs of fig. 2 are obtained. It is interesting to observe that analogous graphs can be constructed if the role of T and η_y is reverted and that these latter graphs are identical to those of fig. 2 only if the objective function is strictly convex, as in the present case where they both correspond to the ridge of the EDI surface of fig. 1a. In the opposite case one should give preference to a period shift or force limited based design as the results are no longer equivalent. However, if a single optimal point should be still selected one should preferably look at the point $\mathbf{vp}^*\equiv(T=1.25,\ \eta_y=0.17,\ \kappa=\forall)$, fig. 2b, as compared to that of fig. 1a, which corresponds to the point where the derivative of the optimal design curve strongly decrease approaching zero, thus indicating that further gains in η_y as T increase are modest. The $\mathbf{vp}^*$ points of the cross sections of the EDI surface at $T=\varepsilon$ are generally well defined, but some flatness of the curves can be observed specially for the lower periods, fig. 1b. This means that points in the neighbour of $\mathbf{vp}^*$ can have almost the same chances to be selected as optimal design solutions, that is, the locus of feasible design range can be extended from a curve to an area if points within a predefined tolerance with respect to $\mathbf{vp}^*$ are equally well accepted as design points.

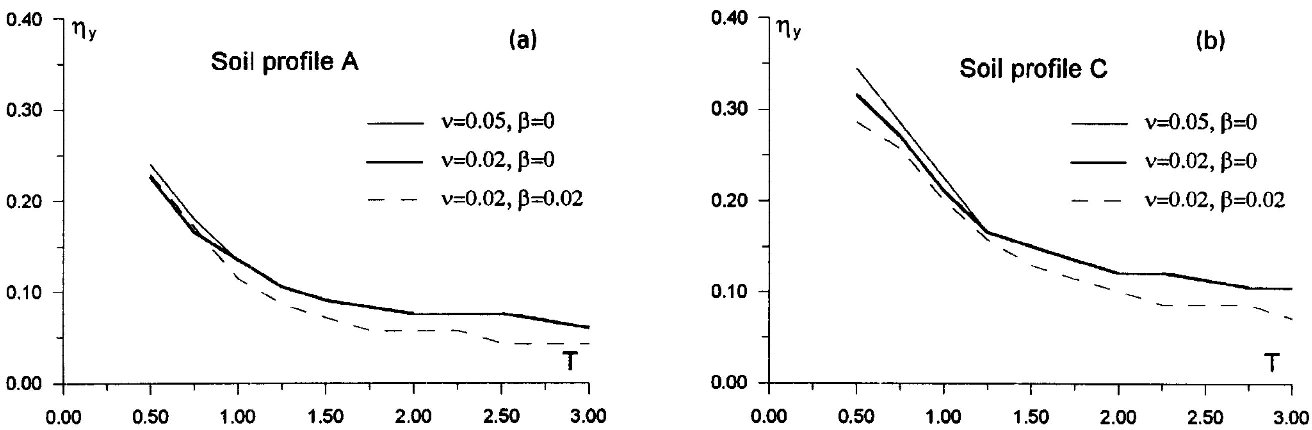

Fig. 2. Optimal design curves for different damping and friction values.

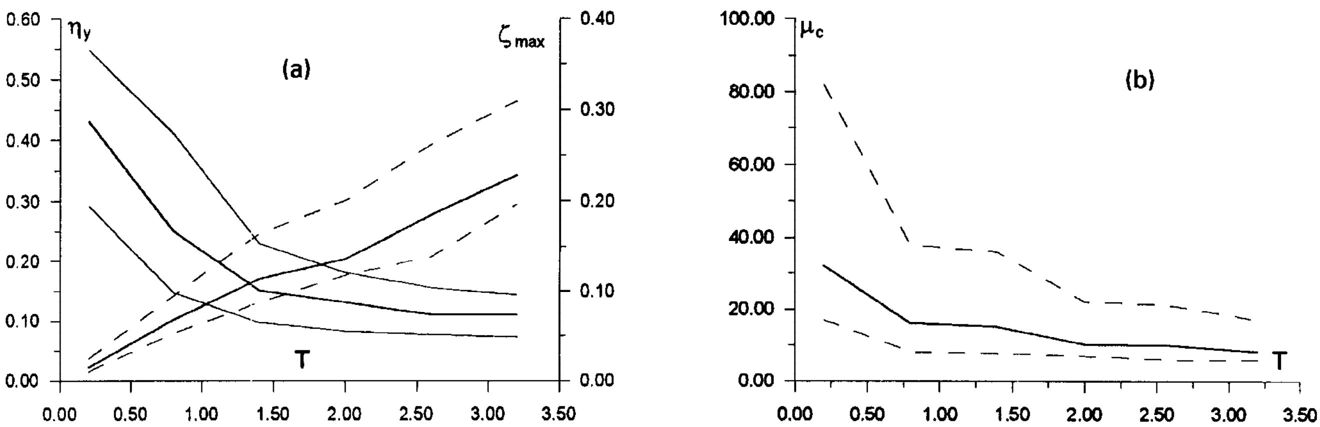

Fig. 3. Optimal design locus (spectrum profile C, ν=5%) and "constraint" loci due to (a) ζ_{max} and (b) μ_c.

The extended design locus, corresponding to a 5% tolerance is shown in fig. 3a. Finally, the last step to be accomplished, for a full vector minimization, is to let decrease the contants ε_i so as to span all the practical values attainable by the other criteria $\{\eta_{max}, \zeta_{max}, \mu_c\}$and to control consequently the restriction posed on the optimal curves. Instead of presenting modified design curves, it is preferred to maintain the same representation of fig. 3a and superimpose to the optimal design locus the corresponding ζ_{max} and μ_c loci, fig. 3a-b, so that the designer can perform a direct check of the alteration of the response quantities against a modification of the design variables. The effects of the hardening parameter κ, no longer neglectable for the other criteria, are shown separately in fig. 4 for the η_{max} and ζ_{max} curves.

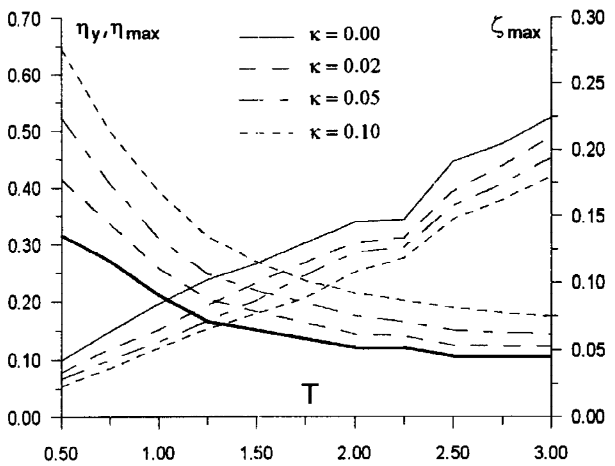

Fig. 4. Optimal design curve (spectrum C, ν=5%) and "constraint" curves due to η_{max}, ζ_{max}.

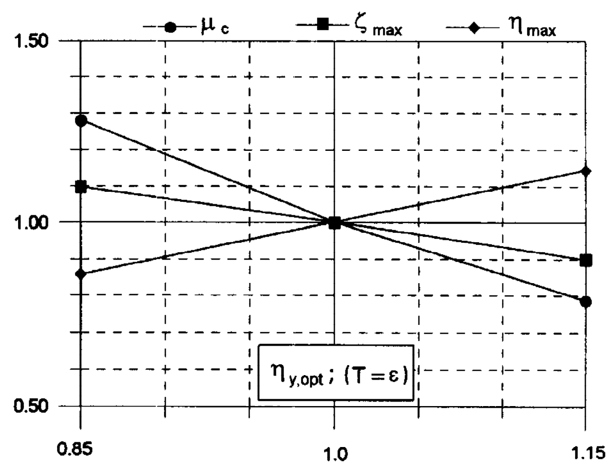

Fig. 5. Local behaviour of the "constraint" criteria at the optimal solution.

In conclusion, it is interesting to analyse the local behaviour of the solution in the neighbour of one point v_P* of the optimal curve. In fact, the emphasys given to *EDI* in the minimization procedure would raise the question wether this single index is actually capable to represent the whole bridge behaviour. A sketch is given in fig. 5 where all the concerned quantities are plotted normalized at their respective values attained at the optimal solution $v_P^* \equiv (\eta_y=\eta_{opt}, T=\varepsilon)$. The local variation of the "constraint" criteria $\{\eta_{max}, \zeta_{max}, \mu_c\}$ is given in the interval $[\eta_{opt}\text{-}15\% \leq \eta_y \leq \eta_{opt}+15\%, T=const]$ of the stationary point *EDI*[v_P*]. Note that, the local behaviour of $\{\eta_{max}, \zeta_{max}, \mu_c\}$ can also be inferred by the inspection of the curves in figs. 3a-b and 4 where it is apparent that the directional derivatives of ζ_{max} and μ_c and of η_{max}, with respect to η_y, are opposite in sign. Thereofore, it can be concluded that it is not possible a gain in η_{max} without a simultaneous loss in ζ_{max} and μ_c and viceversa, consequently the optimal solution for *EDI* represents also the best compromise for the combination of values of the other structural quantities. In other words, it appears actually possible to achieve a multiobjective optimization through a scalar minimization based on a single energy criterion synthetized by the index *EDI*.

CONCLUSIONS

It is has been presented a methodology for the optimal design of energy dissipation devices used for the seismic protection of typical bridge structures. The adopted formulation attempts to solve the problem of searching the conditions of maximum exploitation of the dissipative device, the key element in the protection scheme, yet complying with practical constraints related to the bridge and device behaviour, and develops according to a sequence of scalar minimization aiming at reconstructing a full multiobjective optimization that proves very effective in dealing with simultaneous mutually conflicting criteria such as those above concerned. The adopted model accounts for the nonlinear interaction between bridge deck and substructure and for friction in the bearings and the numerical analyses have been carried out using 10 spectrum compatible artificial accelerograms defined according to the EC8 indications. The results of the investigations indicate that it is possible to construct optimal design graphs which allow for a simple selection of the design parameters of the dissipative device, while respecting predefined constraints required by the serviceability and limit states of both the bridge and the dissipative device. Finally it is worth noting that although the methodolgy is illustrated with reference to a particular, yet typical, case, it has the potential to be easily generalized to more complex protection schemes and to incorporate different, problem oriented, optimality criteria than those here concerned.

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