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ON THE PASSIVE CONTROL OF THE SEISMIC RESPONSE OF ECCENTRIC STRUCTURES BY USING DISSIPATIVE BRACINGS

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ABSTRACT

The aim of this research is to extend the design considerations, used for dissipative bracings in symmetrical structures, to torsionally coupled spatial frames. The paper deals with a two DOF model of a one-storey space frame subjected to several artificial accelerograms. Two design strategies defined as "local" and "global" respectively, are used. The target is, in both cases, the maximization of an energy index (i.e. the ratio of the energy dissipated by the bracings to the energy input by the earthquake), which is taken representative of the condition of optimal performance for the dissipative bracings.

KEYWORDS

Torsional Coupling, Energy Dissipation, Passive Control, Dissipative Bracings

INTRODUCTION

The possibility of providing seimic protection to structures by using passive control techniques has given rise to great interest. Dissipative bracing systems, in particular, have been the object of numerous researches, some of which have proposed efficient design metodologies in connection with simmetrical systems (Filiatrault, 1990, Ciampi, 1993, 1994, 1995). Very little contribution, instead, has been given to the problem of the seimic protection of spatial structures with torsional coupling. In particular some indications for the design of dissipative bracings have been given in a study of Pekau and Guimond (1991).

The aim of the present study is to formulate a design methodology based on the use of a simplified model that schematizes the floor deck of a one story building as a two degrees of freedom system with resistant elements parallel to the direction of the earthquake (Kan & Chopra, 1977); (Rutemberg & Pekau, 1987); (Goel & Chopra, 1990). The center of mass coincides, in the chosen scheme, with the geometrical center of the deck. The global constitutive law of the resistant elements is of a trilateral form, as a consequence of assuming an elasto-perfectly plastic behaviour for both the frame and the bracing. In designing the dissipative bracings it has been kept under control a particular performance index, indicated as EDI, defined by the ratio of the energy dissipated by the bracings to the energy input by the earthquake. Therefore the condition that would give the best performance of the bracings has been sought, by maximising such function. Subsequently, an a posteriori control has been made to establish the range of the optimal values of the design variables and to check the damage of the frame and of the bracings, in terms of kinematic ductility, the drift of the single frames and the global value of the maximum base shear.

The results obtained confirm the general effectiveness of these control systems even for spatial structures and can be used to give synthetical indications for the design.

PARAMETRIC STUDY OF THE ECCENTRIC STRUCTURES

Structural model

According to Pekau (1991), the structure shown in *Fig.1*, is formed by a stiff horizzontal floor deck of rectangular shape, having dimensions $Dn=3\rho$ and $D=1.73\rho$, where ρ is the mass radius of gyration about the center of mass (CM), which coincides with the centre of the rectangle, and the translational mass m is assumed uniformly distributed. The resisting structural elements are constituted by the frames, oriented along to the Y-direction, emboding dissipative bracings. The single constitutive relations of the frames and of the bracings are of elasto-perfectly plastic type, and since they act in parallel, the global constitutive law referred to each resisting element shows a typical trilateral shape (*Fig.2*). The X axis results to be a simmetry axis for the structure, being the system acted upon by a seismic action orthogonal to such axis; therefore the structure has only two degrees of freedom, that are a translation y, in the Y direction, and a rotation θ about CM.

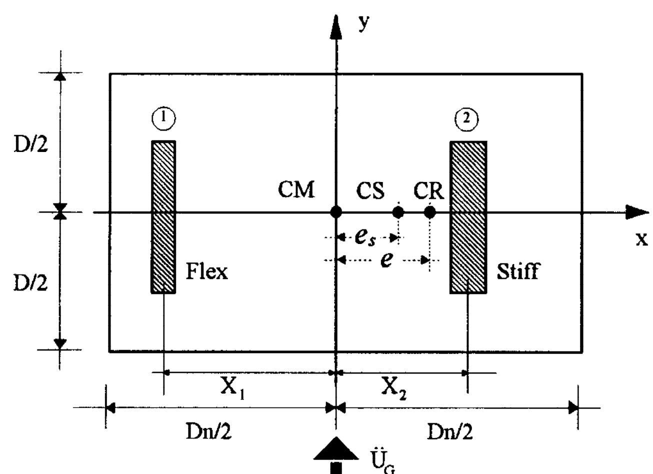

Fig.1

A very important parameter which influences the response of the system, is the eccentricity e, which represents the distance between CM and the stiffness center CR of the system; that is defined as $e = (K_{Y1}x_1 + K_{Y2}\,x_2) / K_Y$,where $K_Y = \Sigma_i K_i$ is the total stiffness in the Y direction and K_{Yi} ed x_i are respectively the stiffnesses of the frames and their distances from CM. The eccentricity e has been normalized with respect to ρ, $e^* = e/\rho$. The parameter e^*, in the elastic range, represents therefore the distance between CM and the point where a static force in the Y direction can be applied without inducing a rotation of the system.The center of strength CS is defined as the point of application of the resultant of the structural strengths F_{yi} and its position with respect to CM is given by the eccentricity $e_S = (F_{Y1}\,x_1 + F_{Y2}\,x_2) / F_y$, where $F_y = \Sigma_i F_{yi}$ is the total strength, and F_{yi} ed x_i are respectively the strengths of the frames and their distances from CM. It follows, from the above definition that, in the case of an elastic structure, the center of stiffness concides with the center of strength. In elastic dynamic analysis the translational and rotational motions are coupled if CM and

CR do not coincide; this aspect is well represented by the parameter Ω_M (Pekau (1987)), defined as the ratio between the rotational and translational frequencies of the uncoupled structure, $\Omega_M = \omega_{M,\vartheta} / \omega_Y$, where the subscript M stands for frequencies computed with respect to CM.

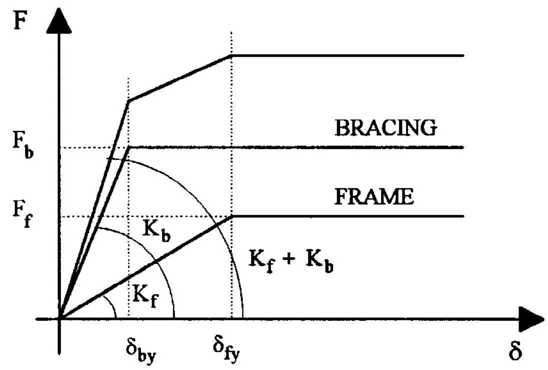

Fig.2

In general it is preferable to use Ω_0 instead of Ω_M, (Pekau, 1987), in fact the first does not depend on e^* since: $\Omega_M^2=\Omega_0^2+e^{*2}$, being $\Omega_0 = \omega_{R,S} / \omega_y$, where the subscript R stands now for frequencies evaluated with respect to CR. On the contrary, in the nonlinear range, rotations and translations may be coupled, even if CR coincides with CM, since it is no longer guaranteed the coincidence CR≡CS. The nondimensional undamped equations of the motion in the case of an indefinitely elastic structure, without considering the contribution given by the bracings, are expressed as follows:

$$\begin{Bmatrix} \ddot{y} \\ \rho\ddot{\vartheta} \end{Bmatrix} + \omega_y^2 \begin{bmatrix} 1 & e^* \\ e^* & \Omega_0^2 + e^{*2} \end{bmatrix} \begin{Bmatrix} y \\ \rho\vartheta \end{Bmatrix} = -\begin{Bmatrix} \ddot{U}_G \\ 0 \end{Bmatrix} \tag{1}$$

Where necessary the viscous damping is accounted for by the matrix C, defined as a function of the mass matrix M and of the initial stiffness K: $C = aM + bK$.

Choice of the parameter values and seismic input

In the parametric study a unitary value has been given to the parameter Ω_0; as known, such a condition corresponds to the maximum coupling of the rotational and translational effects for elastic structures of this type (Pekau, 1987). The total strength of the unbraced frames F_f, in turn, has been assumed equal to the ratio between the maximum base shear (T_{el}), corresponding to an indefinitely elastic behaviour, and the load reduction factor Q ($F_f=T_{el}/Q$). Finally, the stiffnesses and strengths of the bracings (K_{bi}, F_{bi}) with respect to those of the frames (K_{fi}, F_{fi}) have been globally expressed through the nondimensional parameters:

$$\alpha = \frac{K_b}{K_f} = \frac{K_{b1} + K_{b2}}{K_{f1} + K_{f2}} \qquad \tau = \frac{F_b}{F_f} = \frac{F_{b1} + F_{b2}}{F_{f1} + F_{f2}} \tag{2}$$

The following range of variation of the structural parameters, which define the mechanical characteristics of the bracings and the eccentricity e^* have been investigated: Q =[4-8]; α =[0-10]; τ =[0-2.5]; e^*=[0-1.2].

The conventional linear viscous damping has been considered to be 5% of the critical one, whereas for the seismic input, 5 accelerograms, 20 seconds duration each, have been generated; that are compatible with the spectrum defined by the European Code EC8, for type 3 soil conditions. The results are given as averages of the maximum values attained by the interested quantity over the number of the considered accelerograms.

Characteristics response parameters of the model

It is preliminarly observed that: the dissipative bracings are introduced into the structure aiming at the specific task of dissipating energy; therefore, for the construction economy it is convenient that they dissipate as much as possible, in fact, the more energy is dissipated, the more is reduced the response of the structure. These two observations suggest to use, in designing dissipative bracings, an energy functional that accounts for the energy dissipated by the bracings; in particular a nondimensional index, given by the ratio between the total energy dissipated by the bracings, EH_b, and the total energy input by the earthquake,EIR, is introduced:

$$EDI = \int EH_b dt / \int EIRdt \qquad (3)$$

In accordance with results from previous studies (Ciampi, 1995), the choice of the bracing characteristics (stiffness and strength) can be related the maximum values of such index. The remaining important response parameters are then correspondingly checked. Among these latter: maximum kinematic ductility of the frames and bracings and maximum values of displacements and of base shear have been considered.

PARAMETRIC RESULTS AND DISCUSSION

Dimensioning of the dissipative bracings

Two distinct approaches have been followed for dimensioning the dissipative bracings:

- the first one is denominated *Local Method.* In this case the definitions of relative stiffness and strenght given previously in global terms are applied directly to single frames and bracings; eqn. (2) specializes for each single frame as: $K_{bi}=\alpha K_{fi}$, $F_{bi}=\tau F_{fi}$;
- the second is denominated *Global Method.* The parameters of relative stiffness and strength, preserve the "global" meaning of eqn. (2), and, for the choice of the characteristics of the single bracings, it is first imposed that the eccentricity e^* of the system reduces to zero, that is CR≡CM, and then that the position of the center of strenght (CS) be such to minimize and uniform the damage in the frames.

The results of the analyses carried out are presented in terms of maximum values of the response parameters. When appropriate, it is considered for comparison the case of the symmetric unbraced structure. Typical values of the stiffness eccentricity parameters which have been investigated correspond to three very important cases: $e^* = 0.0$; $e^* = 0.5$, moderate eccentricity; $e^* = 1.2$, large eccentricity (Pekau,1991).

Local Method

In the presence of dissipative bracings the variation of the quantities which define the structural model has to be estimated. Among the elastic parameters, the only one that changes is the frequency ω_y; this is modified according to the relation: $\omega_{b,y} = (1+\alpha)^{0.5}\,\omega_y$, where $\omega_{b,y}$ represents the frequency of the structure having zero eccentricity and stiffness equal to the sum of all the stiffnesses, including those relative to the bracings. The equations of motion are similar to those already shown (1), after substituting ω_y with $\omega_{b,y}$. In *Fig.3*, typical curves relating to the EDI index are given for fixed α against τ and for several eccentricity e^*; it is noted that the maximum index values correspond to the case of zero eccentricity. *Fig.4* shows the normalized average of the maximum displacements of the two frames ($Y_{m,n}$): from the figure, it may be observed that the largest reduction of the response occurs in the structure when $e^*=0$, that is in corrispondence with the maximum dissipation of energy (*Fig.3*). The maximum ductilities in the frames μ_f are shown in *Fig.5*. Firstly it can be noted that the ductilities of the braced frames are systematically reduced with respect to the unbraced case ($\tau>0$), for all the different e^*. It is observed also that the frame damage and the plastic demand of the bracings is different in the two frames; particularly it is greater in the flexible frame than in the stiffer frame.

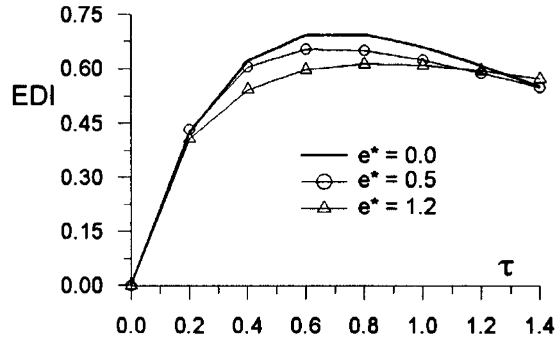

Fig.3 - α = 3

The "local" design method for bracings leads to the following conclusions:

- the dissipative bracings always reduce the structural response;
- the EDI index attains the maximum value when the eccentricity $e^*= 0$; this corresponds also to the maximun reduction of the structural response;
- the damage in the two frames is not uniform when $e^* \neq 0$.

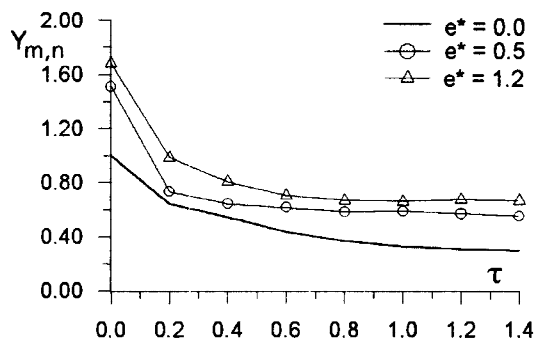

Fig.4 - α = 3

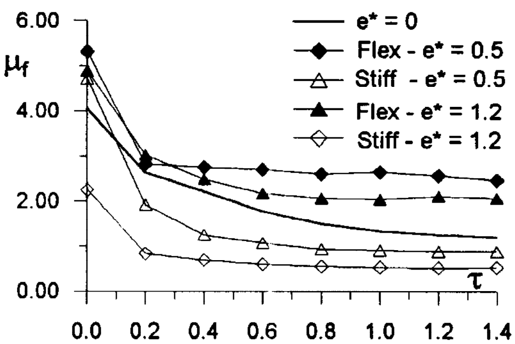

Fig.5 - α = 3

Global Method

The *Local Method* allows to reduce the response with respect to the unbraced case, but it does not allow to eliminate the typical drawback of the torsionally coupled structure, that is the nonuniform response of the frames. To reduce also this negative aspect it is possible to use an alternative method called here *Global Method* which makes the structural response prevalently translational. It allows to take advantage of the characteristics of the dissipative bracings in a such way to modify the original value the eccentricity e^*; it is in fact possible to use the stiffnesses and strenghts of the bracings to calibrate the values of e^* and e_S.

Since the stiffnesses and strenghts of the two bracings are unknown it is necessary to introduce four relations to define their values. For the determination of the stiffnesses it is imposed both the respect of the global measure of the parameter α and the reduction to zero of e^*, being the latter a condition which maximizes the EDI index. For the strengths, it is imposed the respect of a global value of τ, with a proper condition on the eccentricity e_S:

$$\text{a)}\ \begin{cases} e^* = \dfrac{(K_{f1}+K_{b1})x_1+(K_{f2}+K_{b2})x_2}{K_{f1}+K_{f2}+K_{b1}+K_{b2}} = pe^{*\prime} \\ \alpha = \dfrac{K_{b1}+K_{b2}}{K_{f1}+K_{f2}} \end{cases} \qquad \text{b)}\ \begin{cases} e_s = \dfrac{(F_{yf1}+F_{yb1})x_1+(F_{yf2}+F_{yb2})x_2}{F_{yf1}+F_{yf2}+F_{yb1}+F_{yb2}} = e_s{}' \\ \tau = \dfrac{F_{yb1}+F_{yb2}}{F_{yf1}+F_{yf2}} \end{cases} \qquad (4)$$

where $p = e^{*\prime}/e^*$. The analytical relations, a) and b) express the above said conditions, concerning the stiffnesses (a) and the strengths (b). Of the four parameters only e^* is chosen before hand (p=0) while the other three are tied to the maximization of the index EDI. The optimal value of $e_S^*=e_S/e_{SO}$, where e_{SO} is eccentricity of the unbraced structure, results to be dependent only on the load reduction factor Q, used in the design of the frames without bracings. This dependence is shown by the *Figg.6-7*, where the contour lines of EDI are plotted in the e_S^*-τ plane, for the two cases $e_S^*=0.5$ for Q=4, and $e_S^*=0.25$ for Q=8.

By the *Figg.6-7* it is possible to observe that as Q increases, $e_{S,opt}^*$, which gives the max for EDI, decreases. Extensive numerical analyses have permitted to reach the conclusion that a convenient expression of the relation between Q and the parameter e_S^* is given be $e_{S,opt}^* = -0.0625\ Q + 0.75$. This last relation allows to evaluate the optimal value for e_S^*, by using only Q.

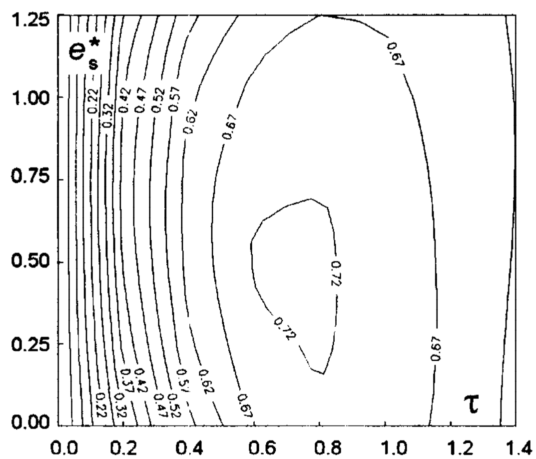

Fig.6 - EDI, e^* = 0.5, Q=4

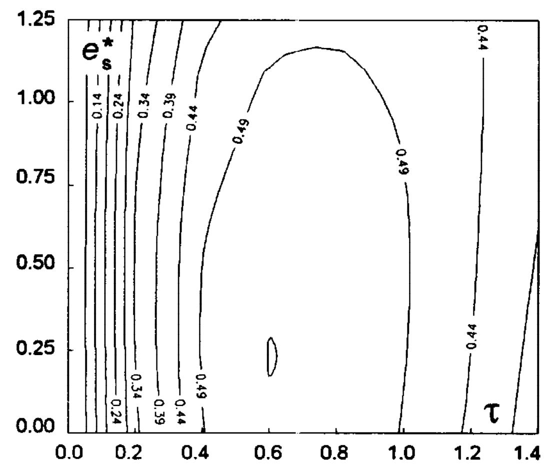

Fig.7 - EDI, e^* = 0.5, Q=8

To complete the design of the dissipative bracings the parameters α and τ have to be determined. Again the concept of maximizing EDI will be used, after some observations on the choice of α.

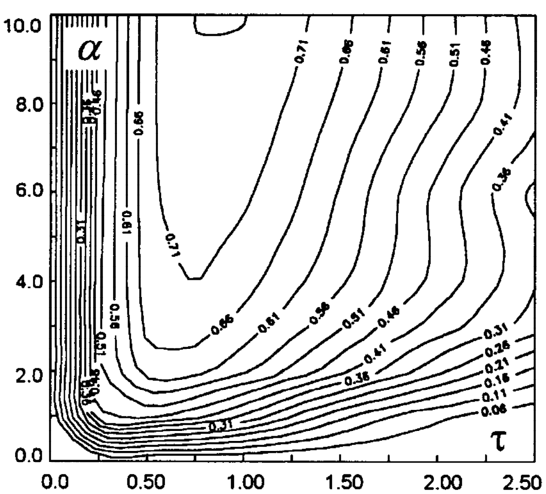

Fig.8 - e^* = 0.5, e_S^*= 0.5, Q = 4

According, for example, to *Fig.8* and to the EDI maximization criterion, one would select the following values α=10, τ=0.75. However, for practical engineering purposes it is desiderable to adopt the lowermost possible values for α. To this end, it is help considering the EDI section at τ = const in correspondence with the maximum EDI value (*Fig.8*). From the *Fig.9* it is seen that EDI reaches values next to maximum also for relatively low α values. In fact already at α=4 the index reaches a value which is nearly 95% of the maximum. Furthermore the ductility in the frames (*Fig.10*), at α=4 has already been largely reduced with respect to the case of unbraced frames (τ = 0), and larger α would not cause substantial reductions to μ_f. Therefore in the choice of α, it is convenient to seek for the lowest α values which already give sufficiently high EDI.

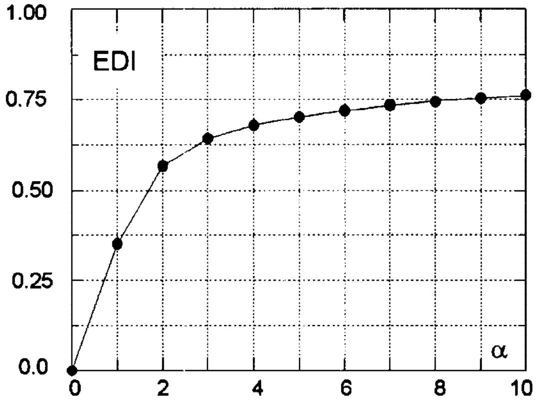

Fig.9 - e^{*} = 0.5, τ=0.75, Q=4

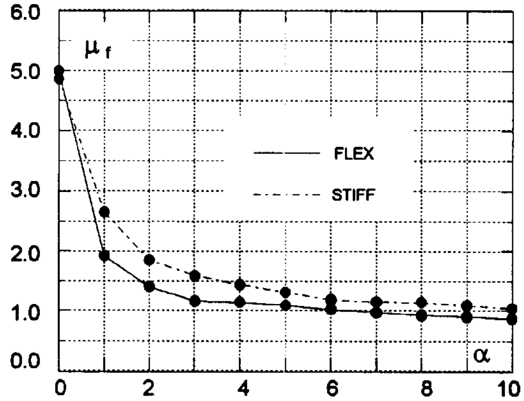

Fig.10 - e^{*} = 0.5, τ=0.75, Q=4

At this point, it is worth recalling that the assumption e^{*}=0 (p=0) considered up to now, could not result the optimal choice at the local level, as shown in the example of *Tab.1*, where it is possible to note that the α values of the single bracings (α_{b1}, α_{b2}) became excessive. In such cases it is no longer convenient to keep p=0, but it is preferable to accept small values for e^{*} that lead to small values also for α_{b1}, α_{b2}.

Tab.1 - e^{*} = 0.5, Q = 4, p = 0

α	1	2	3	4	5	6	7	8	9	10
α_{b1}	3	5	7	9	11	13	15	17	19	21
α_{b2}	0.5	1.25	2	2.75	3.5	4.25	5	5.75	6.5	7.25

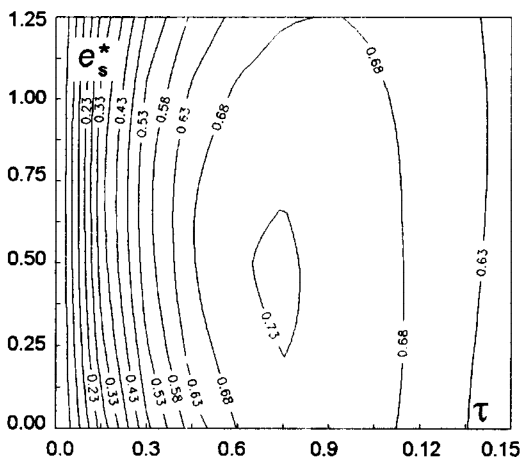

Fig.11 - EDI, e^{*} = 0.5, Q=4, p =0.25
α_{flex} =9.5, α_{stiff} =3.87

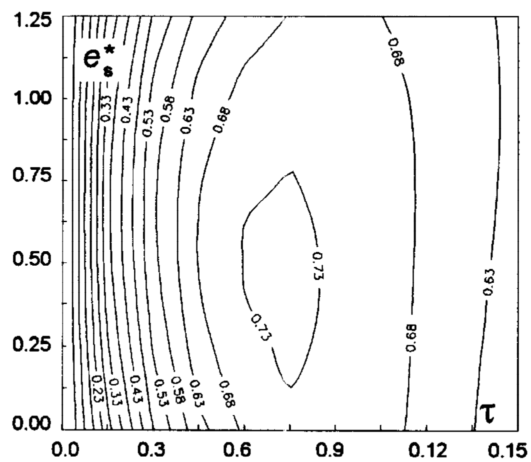

Fig.12 - EDI, e^{*} = 0.5, Q=4, p=0.5
α_{flex} =8, α_{stiff} =4.25

The results given in *Figg.11-12*, as compared to that of *Fig.6*, in fact, show that for the relative eccentricity values p of the order 0.25 and 0.5, the value of e_S^{*}, remain close to 0.5, and also that the ductility of the frame and the bracing (*Figg.13-14*), are not far off the values attained for the case e^{*}=0, even for the partial re-centering ($p \neq 0$).

CONCLUSIONS

It has been studied the optimal performance of a two degrees of freedom model endowed with dissipatve bracings. In spite of its simplicity the model has allowed to reach some important conclusions related to the design of dissipative bracing when torsional coupling is present.

The principal results obtained are:

- the dissipative bracings are always effective in reducing the structural response, more o less satisfactorily, depending on the different approaches which can be followed in the design process.
- the Global Method appears more effective: using the additional stiffness given by the bracings to recenter partially or totally the stiffness center CR, and the additional strengths, to move the strength center CS it permits to obtain a greater response reduction and a sufficiently uniform damage in the frames and bracings;
- the optimal value of e_S^* seems to depend only on the plastic demand of the frames: the results of the analyses indicate that a simple relation exists between optimal e_S^* and the structural factor Q, indipendently on all the other structural parameters;
- even though it appears desirable to reach the total re-centering of the elastic stiffness, sometimes this can imply unrealistic bracings, so that also a partial stiffness re-centering can prove satisfactory, if also the responses of the single frame are to be kept sufficiently close to each other.

Future developments of this study will consider more realistic models of the structure, obtained by the insertion of resisting elements also in the direction ortogonal to the earthquake motion, as to include the full torsional coupling in the two plane dimensions.

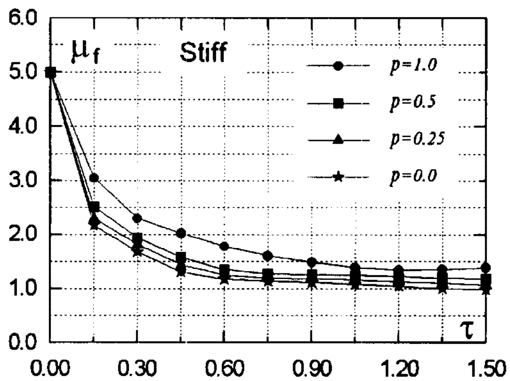

Fig.13 - $e^* = 0.5$, $es^*=0.5$, $\alpha=4$

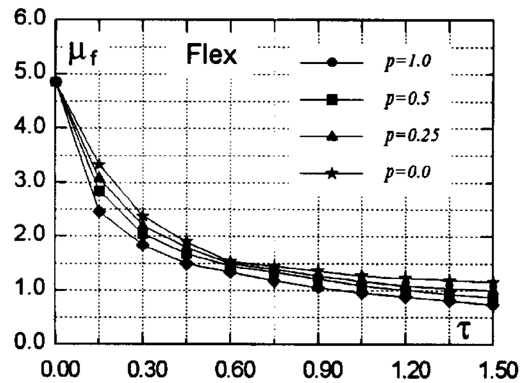

Fig.14 - $e^* = 0.5$, $es^*=0.5$, $\alpha=4$

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