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On the seismic design of dissipative bracings

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ABSTRACT: The paper addresses the problem of designing dissipative bracing systems, for seismic protection of structures. Both design of new structures, and retrofitting of existing buildings are in principle considered; but the main focus of the paper is on a strategy for upgrading existing r.c. buildings. After discussing a one d.o.f. model of the building, equipped with dissipative bracings, inelastic response spectra, useful for designing the bracings, are constructed. Problems related to the extension of the design methodology to multi d.o.f. structures are finally considered.

1 INTRODUCTION

A research conducted in the last few years at the University of Rome has led to the characterization of special dissipative bracing systems which may provide seismic protection to framed structures, by means of added energy dissipation capacity and added stiffness; they are based on the yielding of special steel devices inserted in the bracing, for various, concentrical or K-type, configurations. Experimental work has shown that the proposed bracing systems have very good energy dissipation characteristics and high low cycle fatigue life (Ciampi 1990, 1991a).

Many proposals of systems of such kind have been recently reported, and even a few applications, both to design of new buildings and to retrofitting of existing ones, (Pall 1987, Whittaker 1989, Filiatrault 1991). In particular two mechanisms have been successfully used for energy dissipation: yielding of steel devices, in various arrangements, and friction. Irrespective of their relative merits and of the different mechanisms on which they are based, yielding or friction, the different systems give rise to approximately the same global behavior which is characterized by fully dissipative cycles and high low cycle fatigue life. This is very advantageous with respect to traditional bracing systems, which have favourable stiffening capacity, but very poor dissipation characteristics. For design purposes all the systems proposed may be reduced to an equivalent elastoplastic diagonal brace, with two characteristic parameters, stiffness and yield force level.

The potential of dissipative bracings in seismic design can be easily recognized: in fact, by increasing stiffness they improve structural behavior under service conditions; by dissipating energy, in a stable and controlled way, they improve structural response to strong earthquakes. Nonetheless the problem of how to actually design and use them, in the most efficient way, is still relatively open. The most extensive and complete effort towards establishing a design methodology, which is known to the authors, is due to Filiatrault (1990). It explicitly refers to the case of friction braced frames but it can be easily applicable to systems based on yielding devices as well. For this reason in a recent paper, (Ciampi 1991b), concepts taken by that work have been used; it has been found, in particular, that the distribution of stiffness and yield loads in the braces, at different story levels, plays an important role in determining the effectiveness of the seismic protection.

In this paper a different approach is proposed, which is based on inelastic response spectra, computed for a simple one d.o.f. system. Although applicable to the more general design problem, the approach has been here specialized to a particular strategy for retrofitting existing buildings. Compared to Filiatrault's approach, which only gives an optimal choice of the slip load in the bracing, for a chosen stiffness, the present one has the advantage that it allows to quantify the expected level of damage in the structure to be protected, associated with different design choices, and to control, at the same time, the energy dissipation in the bracing.

2 INELASTIC RESPONSE SPECTRA FOR THE DESIGN OF THE BRACINGS

The single degree of freedom system of fig. 1 represents schematically a frame equipped with dissipative bracings. The global structural reaction of this system results from the contributions of the frame, to be considered completely known, since it is an existing structure to be retrofitted, and of the bracing, which has to be designed. Both contributions are assumed, for simplicity, elasto-perfectly-plastic, so that the global structural reaction has the trilateral shape shown in fig. 1.

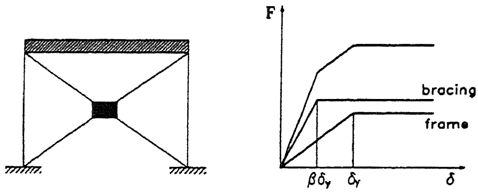

Fig. 1 One d.o.f. model of the braced frame and its structural reaction

Since the very concept of dissipative bracing requires that they assume as much as possible of the dissipation, the plasticization of the bracing is expected to commence before that of the frame; as a consequence, β, ratio of the story displacement which induces yielding in the brace to the one inducing yielding in the frame, shall be always considered less or equal to 1.

Designing the bracing means defining its stiffness K_b, and yield level F_{by}, or two other parameters related to these two. Here the choice has been made to consider, as design parameters for the bracing, the already introduced β and another parameter, α, defined as the ratio of the elastic vibration period, T_b, of the braced frame, (after retrofitting), to the one of the original frame T_f, ($\alpha=T_b/T_f$). Also α is always positive and less than 1; in particular, it is equal to 1 in the case of no bracing.

In order to construct inelastic response spectra, for the system of fig. 1, useful for designing dissipative bracings, a set of 5 artificial accelerograms, of 20 seconds duration, has been generated; they are compatible with the spectrum defined by the Code Proposal of the Italian Research Council, (known as GNDT Draft Code), for type 2 soil conditions. This Code may be considered an Italian version of the better known European Code.

The strength, or yield level, of the frame, F_{fy}, is normalized with respect to the peak acceleration a_{max} which characterizes the set of accelerograms; the adimensional parameter η_f is therefore defined as the ratio of F_{fy} to the product of the mass of the system, m, times a_{max}, ($\eta_f = F_{fy}/ma_{max}$).

Two commonly used ductility indices have been considered to characterize the response of the frame, in terms of damage, before and after retrofitting: maximum frame displacement ductility, μ_f, and cumulative ductility, μ_{fH}, also known as hysteretic ductility, since it is representative of the energy dissipated through plastic hysteresis.

The following criterion has been chosen as a basis for retrofitting: to accept, in the retrofitted structure, under severe earthquakes, the same level of damage which is normally accepted when designing a new structure according to the Code. With this criterion in mind it is easy to attribute a meaning to different values of the parameter η_f. In fact, according to the equivalent static force format of the GNDT Code, the design strength should be:

$$F_{fy} = R\delta CW$$

where R is a function of the vibration period T, (R=1 for 0<T<0.8 sec, R<1 and decreasing for T>0.8 sec), δ safety factor, W weight of the mass of the system, and C a seismic intensity coefficient, depending on the seismic zonation (3 seismic intensity classes are considered in the Code). It follows: $\eta_f = R\delta C/(a_{max}/g)$, with a_{max} peak ground acceleration and g gravity acceleration.

The same Code gives, as previously reported, a spectrum for the generation of artificial accelerograms, to be used in nonlinear dynamic analysis, and specifies the values of peak accelerations, a_{max}/g, corresponding to the three different seismic intensity classes considered. By comparing the specified peak accelerations with the corresponding seismic coefficient, C, previously introduced, it is possible to note that $C/(a_{max}/g)$ is always approximately equal to 0.3, irrespective of the seismic intensity class which is being considered; as a consequence, since the safety factor, δ, can be assumed equal to 2, at least for the periods for which R=1, η_f is equal to 0.6 . This value of η_f may be then considered as characteristic of structures designed in accordance with the Code prescriptions, and, therefore, according to the general Code philosophy, capable to withstand severe ground motions without collapsing and with an acceptable degree of damage. Following similar lines of reasoning, it may be stated that values of η_f less than 0.6 characterize structures which do not comply with the Code and need retrofitting. Different values of η_f correspond to different levels of structural inadequacy and to different retrofitting situations; as an example, η_f=0.4 may be easily interpreted as characteristic of a structure, designed for a second Italian seismic intensity class, (C=0.07), which has to be upgraded to respond to seismic actions characteristic of the first intensity class (C=0.1); in fact the ratio 0.4/0.6 has about the same value as 0.07/0.1 . Similarly η_f=0.3 may be interpreted as characteristic of a structure which has to be upgraded from the third to the second Italian seismic intensity class, that is from C=0.04 to C=0.07 . In this investigation the following values of η_f have always been considered: 0.6, 0.4, 0.3, 0.2 .

The figures which follow present inelastic response spectra in terms of μ_f and μ_{fH}; they have been obtained by averaging the corresponding response quantities over the defined set of 5 artificial accelerograms, and are presented as functions of the already introduced parameters T_f, η_f, β and α. As an example, figg. 2 and 3 show the variation of μ_f as a function of α for different frame strengths, η_f, at fixed T_f and β; in particular they present the cases corresponding to T_f=0.6 sec and β=0.5 and 1 respectively. Similarly fig.4 presents the response spectrum in terms of μ_{fH} for β=1.

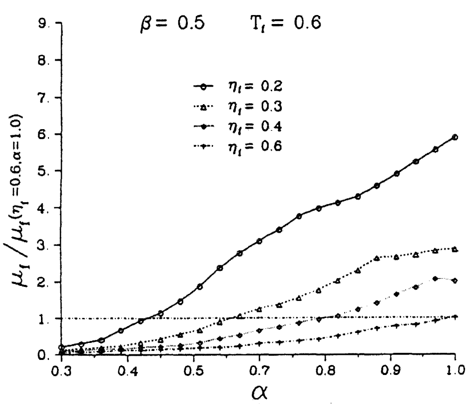

Fig.2 Frame maximum ductility spectrum

In order to make the representation more effective and the three figures more comparable, the ductility values are normalized and expressed as ratios to the values that correspondingly they assume for α=1, (unbraced frame), and η_f=0.6, (well designed structure). With this normalization, values on the curves less than 1 correspond to situations where an acceptable degree of damage in the frame is expected.

The above introduced graphs are intended to be used for designing the bracing; the two parameters which define such design beeing α and β. Let us assume that β has

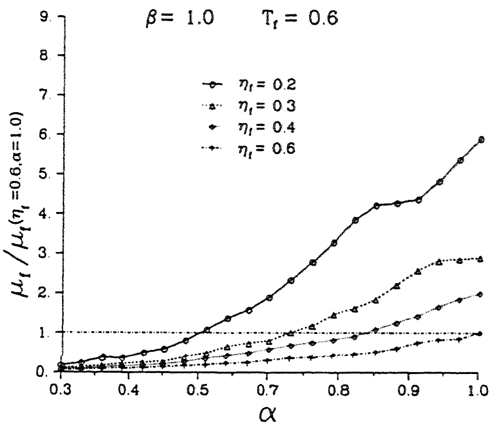

Fig.3 Frame maximum ductility spectrum

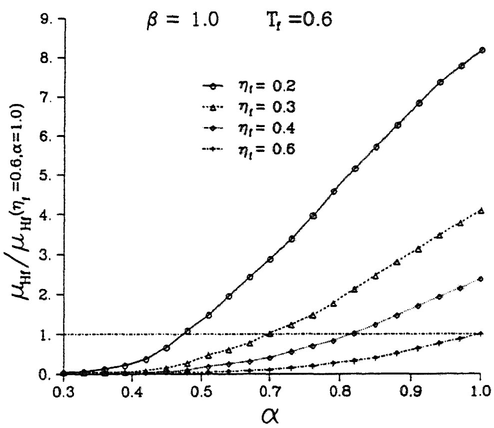

Fig.4 Frame hysteretic ductility spectrum

been decided beforehand; it shall be shown later that the interesting range of β values is between 0.5 and 1 and that a variation in this range does not affect too much the results. As for α, after noting that lower values of α correspond to increasing stiffness of the bracing, relative to the stiffness of the frame, it may be observed that the smaller α the smaller is the damage in the frame; very low values of α might also correspond to zero damage, that is elastic behavior of the frame, even under severe earthquakes.

In this investigation, where the attention is devoted mainly to the problem of retrofitting r.c. frames, which are characterized by a relatively high stiffness, it has seemed not realistic to consider values of α less than 0.3 . In general, for these cases, it is uneconomical to design the bracing so that very low values of α are provided. For this reason it is proposed that α be chosen as the highest value which realizes the previously stated

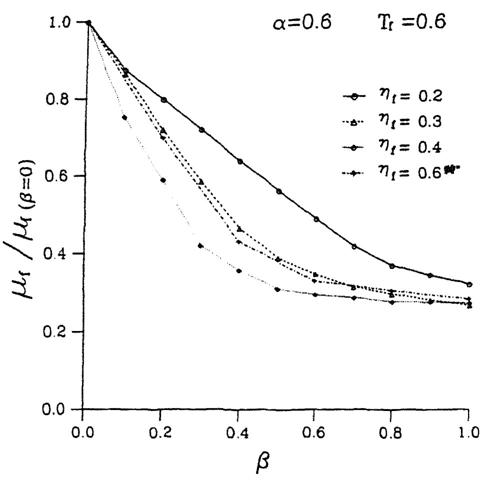

Fig.5 Frame maximum ductility spectrum

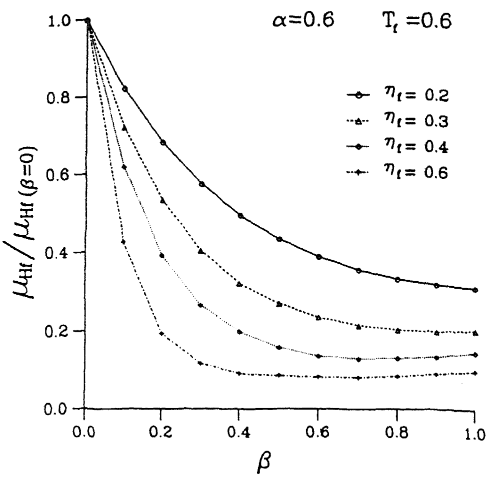

Fig.6 Frame hysteretic ductility spectrum

objective of retrofitting, that is equality of damage with respect to a well designed unbraced frame. According to this methodology, for a given frame to be retrofitted, (assigned T_f and η_f), and after choosing β, the value α is found as the one which corresponds to unitary value of the chosen normalized damage measure.

As a comment to the choice of two different measures of damage, μ_f and μ_{fH}, it has to be noted first that the corresponding curves, in the adimensionalized representation, have very similar shape; then it may be considered that, if maximum ductility has a closer relation to the usual Code philosophy, hysteretic ductility seems to give a more satisfactory representation of damage, since it takes into account the entire history rather than only a maximum value; such possible greater meaningfulness finds its correspondence also in a more regular behavior of the curves themselves (compare figg 3 and 4).

Figg.5 to 7 serve to demonstrate the effects of varying β, when α and T_f are fixed. The ordinates show again the two considered damage measures, μ_f and μ_{fH}, normalized, this time, to the values that they attain for the unbraced frame case, (β=0), so that all the curves start with value 1 for β=0; these curves, which are decreasing for increasing β, may be easily interpreted as damage reduction in the frame, caused by the use of the dissipative bracings. As an example the figures present the case α=0.6 and α=0.8, which are two values of α not too small, but already quite effective, and T_f=0.6 sec, as before. The rate of decrease of the curves is high only in the range 0<β<0.5; for β>0.5 the curves vary much less; in this latter range they may reach a minimum for a β value less than 1, or continue decreasing up to β=1, but

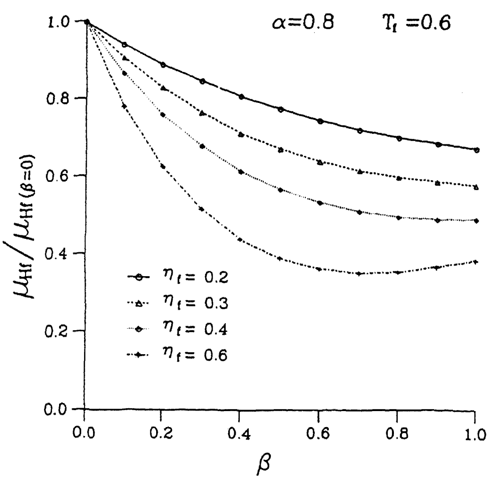

Fig.7 Frame hysteretic ductility spectrum

always with a very modest variation.
These considerations justify what has been anticipated about the choice of β in the range 0.5-1 . In general β<1 and close to 0.5 should be preferred, based on the different quality and dependability of the dissipation which may be offered by the bracing, compared to the one which may be offered by the frame. This aspect cannot be included, of course, in a simple way in the model under consideration, (fig.1). The decrease of μ_f with β is less than the corresponding decrease of μ_{fH}, and, as already observed, the curves which refer to this latter measure have a more regular behavior, (compare figg.5 and 6). Characteristic of such regularity is, as an example, the fact that, for increasing η_f, there is a stronger reduction of the

relative damage, as a function of β, and, at the same time, minimum values of such damage tend to appear for $\beta<1$. Incidentally, with reference to fig.7 it may be observed that it is possible to increase very much the level of seismic protection of a structure, which is already well designed ($\eta_f=0.6$), even for modest stiffness of the brace, ($\alpha=0.8$); in fact, corresponding to a minimum value, which is reached for β close to 0.7, there is a reduction of the hysteretic ductility which is more than 60% , (fig.7).

3 EXTENSION TO MULTI D.O.F. SYSTEMS

The considerations developped for a single d.o.f. system may be extended to multi d.o.f. systems; and the extension is much more direct, as much more uniform is the expected response, in terms of damage distribution within the structure. The important issue is to find the proper stiffness and yield force distribution in the braces, at different story levels, which facilitates such uniform response along the structure; this implies avoiding damage concentrations, in specific locations of the frame, and engaging, as much as possible uniformly, all the braces in the energy dissipation. The problem, already approached in (Ciampi 1991b), is under investigation at present. Here only preliminary results are presented.

They refer to a very simple scheme where the structure to be retrofitted is modelled as a shear-type building, having regular variation of stiffness and strength parameters at different story levels; the distribution of stiffnesses and yield levels in the braces, to be designed, is also assumed to be systematically similar to the corresponding distribution in the frame.

This simplification makes the extension very direct, because, beeing fixed the shape of their variation, the relevant parameters are easily identified with the corresponding ones of a single d.o.f. system.

Scope of this first part of the investigation is only to verify that designing the bracing on the basis of the methodology illustrated for a single d.o.f system produces, even in multi d.o.f. systems, damage levels which are comparable with those which would take place in the same frame designed accordingly to the Code.

Two structures have been considered, respectively 3 and 6 story high, with same interstory height along the height and same masses at each level, characterized by the two elastic fundamental vibration period 0.3 and 0.6 seconds respectively.

Different distributions of stiffness and strength have been considered. The stiffnesses have been calibrated so that the vibration periods of the unbraced frame have the above referred values 0.3 and 0.6 sec. The strengths have been properly scaled to correspond to different normalized strengths η_f, so that they offer resisting shears which always touch and envelope the corresponding required seismic shears; these latter beeing evaluated on the basis of a static conventional analysis, with triangular distribution of forces along the height, as for the GNDT Code specifications.

The nonlinear dynamic analyses have been conducted by using the same 5 artificial accelerograms, referred before; the results are given as averages over the set of accelerograms.

The investigation has shown that distributions of strengths which give rise to uniform resistance conditions, that is which are similar to the required seismic shear distribution, always produce strong concentration of damage, at specific stories, whatever is the associated distribution of stiffnesses, both in the unbraced and in the braced frames; the same occurs for constant strength distributions. Good results have been obtained using linear distribution of strengths along the height. This is also in accordance with some results reported by Rega (1984).

Here are presented, as an example, only the results which refer to the case of a 6 story building, with constant stiffness distribution and one of the considered linear strength distributions. In fig.8 are shown, for comparison, the distributions, along the height of the frame, of the two damage measures, μ_f, (fig.8a), and μ_{fH}, (fig.8b); the 4 curves refer to the unbraced frame to be retrofitted, ($\alpha=1, \eta_f=0.4$), the same frame with increased strength so to correspond to Code design, ($\alpha=1, \eta_f=0.6$), the frame retrofitted using two choices of braces ($\beta=0.5$, $\beta=1$) and the corresponding values of the α parameter, which follow from the use of the spectra and from the application of the equal damage criterion.

The figure shows that, for the considered conditions, the results anticipated on single d.o.f. systems do apply to multi d.o.f. systems. In fact the average ductilities, in the retrofitted frame, are reduced to values very close to those which occur in the corresponding 'well designed' frame and are also similarly distributed along the height. This observation applies to both damage definitions (fig.8a and 8b).

As also expected, there is very little difference in behavior, consequent to choosing $\beta=0.5$ or $\beta=1$.

4 CONCLUSIONS

A methodology, based on inelastic response spectra for single d.o.f. systems, has been introduced, which allows to design

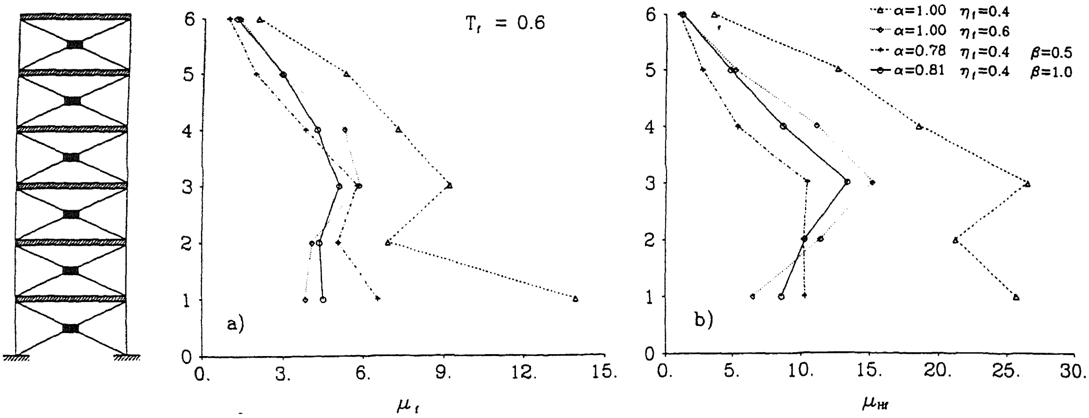

Fig. 8 a) Maximum story ductility, b) hysteretic story ductility

dissipative bracing systems for retrofitting buildings, which are not seismically safe, according to the usual Code based standards. The same response spectra could also be used for the seismic design of new buildings, for which dissipative bracings might providehigher degrees of seismic protection; but this aspect is not particularly stressed in the paper, which is mainly devoted to present a retrofitting strategy.

The application of the simple methodology to real multi d.o.f. systems requires the definition of proper distributions of stiffnesses and yield forces at different story levels, so to favour uniform engagement of the bracings in the energy dissipation process and to avoid concentration of damage at specific locations of the frame.

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