11 WCEE

Paper No. 1933. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

DESIGN CRITERIA FOR EARTHQUAKE RESISTANT BUILDINGS WITH ENERGY DISSIPATORS

LUIS M. BOZZO, DORA FOTI and FRANCISCO LÓPEZ-ALMANSA

Technical University of Catalonia
C. Gran Capitán, s/n (Módulo C1)
08034 Barcelona (Spain)

ABSTRACT

This paper presents a numerical nonlinear parametric study on the seismic response of buildings equipped with energy dissipation devices.The final objective of the assessment is to propose simple preliminary design guidelines. Three reinforced concrete buildings with 3, 7 and 15 stories, representatives of the short, medium and long period ranges, are analyzed. The input consists of ten recorded earthquakes corresponding to medium and stiff local soil conditions. The response is calculated for three types of dissipation connections: *Adding Damping and Stiffness* (ADAS), *Constant Drift Device* (CDD) and *Constant Friction Slip Braces* (CFSB). In the first two cases, a single device is located in each floor and, in the third one, two devices are installed per floor. The threshold (or yielding) forces for each mechanism are selected as the 50%, 75% and the 100% of the equivalent lateral forces recommended by the UBC 1991 for a ductile moment resisting frame. The assessment compares the response of conventional braced buildings and protected ones in terms of reduction of base shear forces and maximum horizontal displacements and interstory drifts. The study includes more than 400 nonlinear analysis. Finally, the paper proposes a simple nonlinear single degree of freedom model useful for preliminary design.

KEYWORDS

Seismic protection, Passive control, Energy dissipators, Buildings, ADAS, CFSB, CDD, Friction, Design criteria, Nonlinear analysis.

INTRODUCTION

Apparently there is the belief that energy dissipators are not as efficient to reduce earthquake induced forces as base isolation systems (Pong *et al.* 1994). This research shows that this is not necessarily the case and that such reduction is a function of the sufficient number and proper design of the devices. Previous research is usually based on assuming constant design parameters along the height of the building. There are studies about how to select an optimal variation for viscous dampers but it is difficult to construct those dampers with different measures and it is not economical to trespass certain dimensions. In the reference by Pong et al. (1994) it is shown that there is a significant reduction in base shear forces for structures with viscous dampers designed to provide a high equivalent damping but it is recognized that it is not economical to construct such devices. By the contrary, energy dissipation mechanisms based on plastification of metals (such as ADAS or TPEA) or friction of plates (CFSB) are easy to construct and their yielding force can be economically modified just varying their thickness and number of plates or changing the normal force, respectively.

Much of the research in energy dissipation systems focuses on experimental testing of a particular device (Aiken and Kelly 1990, Inaudi and Kelly 1992). In a lower extent there are limited numerical analysis of buildings equipped with dissipators (Xia and Hanson 1992). However, there is a lack of a comprehensive study on the seismic efficiency of such systems and, consequently, a general design procedure is not available. This paper presents a numerical nonlinear parametric assessment on the seismic response of buildings equipped with energy dissipation devices. According with the obtained results, simple design rules are proposed.

DESIGN CRITERIA FOR ENERGY DISSIPATION DEVICES

A general expression to model the nonlinear hysteretic response of energy dissipation connections is

$$\begin{aligned} F &= kd && \text{for } d < d_y \\ F &= kd_y \left(d/d_y\right)^n && \text{for } d > d_y \end{aligned} \tag{1}$$

where F is the cyclic axial or shear force in the element; d_y is the yielding displacement; k is the element stiffness $k=F_y/d_y$ (where F_y is the yielding force); and n is a positive exponential to best represent the hysteretic characteristics of the connection (n<1). Consequently, there are three design parameters for each connection: F_y, d_y and n.

The *Adding Damping and Stiffness* (ADAS) device dissipates energy through shear forces (Scholl 1990). The connection is made of X-shaped steel plates installed usually as illustrated in Fig. 1. Within a single connection the steel plates are parallel so they have the same yielding displacement. Based on observations of experimental hysteretic loops and some suggested design guidelines (Hanson *et al.* 1992), the selected design parameters for the ADAS connections are d_y= 0.006 cm and n = 0.25. The remaining design parameter (F_y) is obtained next in this section.

The *Constant Friction Slip Braces* (CFSB) device dissipates energy through axial forces (Akbay and Aktan 1990). The connections are usually located at the middle of steel braces, as illustrated in Fig. 1. Due to its rigid-plastic hysteretic response, n = 0 and d_y = 0 and therefore the unique design parameter is the sliding threshold force F_y. This parameter is a function of the friction and normal force at the connection. Even though, in general, the friction coefficient varies with velocity and actual reactive force during an earthquake, for the purposes of this study a constant friction coefficient equal to 0.1 is considered. The normal force at the sliding connection can not be achieved by weight and it should be obtained by local prestressing or other mechanisms.

The *Constant Drift Device* (CDD) is located at the intersection of the braces and the stories as illustrated in Fig. 1 similarly as the ADAS connection. This device is a pure friction one which dissipates energy through the sliding at the connection. For low levels of lateral forces the static friction coefficient restraints the horizontal movement between the braces and the stories. Consequently, for low magnitude earthquakes and wind forces the structure behaves as a conventional braced structure. During a severe earthquake the static friction coefficient in the sliding connection is exceeded and there is a relative horizontal movement between the stories and the braces. Due to its rigid-plastic hysteretic response, n = 0 and d_y = 0 and the unique design parameter is the sliding threshold force F_y. Similarly to the CFSB connection, the sliding threshold force is a function of the friction coefficient and the normal force. In general, a building may be decomposed in two substructures: vertical and lateral loads structural resistant systems. The first ones are typical steel or concrete bare frames. The second ones are usually frames with concentric or eccentric braces. In current American design practice it is common to design the braces not to carry gravity loads. Consequently, the normal force at the sliding connection CDD can not be achieved by weight and it should be obtained by local prestressing as in the CFSB device.

Dissipators are located in all the floors. A single ADAS or CDD device is installed at each story and two CFSB connections are placed at each floor. In all the cases the yielding force in the connections varies along the height of the building. For the ADAS and CFSB systems the design criteria to select such variation is to employ the UBC-91 equivalent lateral loads. A preliminary static analysis using those forces provides the axial forces in the diagonal members. For the CFSB system a 50%, 75% and 100% of the aforementioned axial load are directly considered as the design sliding forces. For the ADAS connection similar percentages of the horizontal projection of the braces axial forces at the connection determines the design yielding forces. For the CDD system the threshold yielding force is considered to vary linearly along the height of the building in the same proportion as its weight.

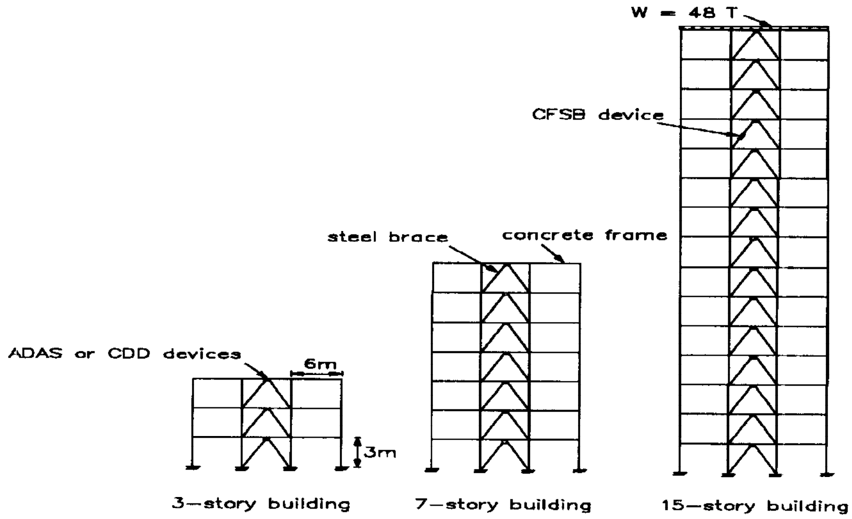

Fig. 1 Three basic structural configurations selected for the parametric study.

A commonly used design criteria for buildings using energy dissipators is that for a severe earthquake the building must remain in the linear elastic range with nonlinearities concentrated at the energy dissipation devices. Taking into account that in high seismicity areas at least an intermediate level of ductility is always mandatory, a recommendable design criteria for an extreme earthquake is to allow the structure to dissipate energy. For base isolated structures and due to the drastic variation between the yielding force in the isolator and the upper structure it is not advisable, in general, to allow the structure to enter clearly in the nonlinear range (Bozzo and Barbat 96). However, this is not the case for structures using energy dissipation devices and the obligatory intermediate level of ductility allows an additional safety factor against extreme earthquakes.

EARTHQUAKE RESPONSE OF BUILDINGS

This section presents a numerical parametric assessment of the response of buildings incorporating energy dissipation devices designed following the aforementioned criteria. To analyze the response of buildings in the short, medium and long period ranges, the study includes the buildings with three, seven and fifteen stories illustrated in Fig. 1. The buildings are made of reinforced concrete, except for the diagonal braces which are steel elements. The fundamental periods are 0.11, 0.37 and 1.32 s. Variation in these periods were achieved modifying the stiffness and mass. Each corresponding period was increased to 0.17, 0.56 and 1.92 s, and reduced to 0.07, 0.25 and 0.94 s.

The input consists of ten ground motions which represent those expected for medium to stiff local soil conditions. The records are normalized with respect to three Housner intensities instead to their peak accelerations. The first selected level of Housner intensity is 71 cm, which corresponds to the Santa Cruz register during the Loma Prieta 1989 earthquake. The third level of intensity corresponds to 195 cm, which is half the value for the Sylmar register during the Northridge 1994 earthquake. The second level is the average of the previous values.

Table 1 includes the ground motions along with their Housner intensity, initial scale factor, peak acceleration, scaled peak acceleration and duration. It is clear from the table that despite peak accelerations for the Sylmar and Santa Monica registers are very close, their Housner intensity varies by a factor larger than two. This result is not surprising since the damage potential of the Sylmar record is larger than the one of the Santa Monica earthquake.

Table 1. Registers and scale factors for a Housner intensity equal to 71 cm

Register	Housner Intensity (cm)	Max. Accel. (cm s^{-2})	Scale Factor	Normalized Max. Acc. (cm s^{-2})	Duration (s)
Sylmar, Northridge 1994	388.050	826.760	0.1822	150.720	59.98
Santa Monica, Northridge 1994	165.426	865.965	0.4276	370.287	59.98
Gilroy Loma Prieta 1989	113.956	433.616	0.6207	269.160	39.98
Santa Cruz Loma Prieta 1989	70.736	433.117	1	433.117	39.98
Taft S21W, 1952	59.167	174.573	1.1956	208.720	17.59
Parkfield, 1966	38.898	201.010	1.8185	374.631	30.38
Helena N00S, 1935	51.276	143.157	1.3795	197.485	9.99
El Centro, 1940	89.848	349.216	0.7873	274.935	15.98
Lima NS, 1970	28.591	178.951	2.4741	442.725	89.54
Lima EW, 1970	48.892	192.489	1.4468	278.495	89.54

In general, it is well known that for periods larger than about 0.5 s, the maximum displacements for a single degree of freedom system in the linear and nonlinear ranges are similar, regardless of their strength (Newmark and Hall 1982). For short period structures there is an amplification region that depends on the strength.

Figure 2(a) presents the average (on the ten inputs) maximum roof displacements normalized with respect to the lowest Housner intensity (71 cm). The figure presents results for the three structural configurations illustrated in Fig. 1 and for the aforementioned variations in their fundamental periods. The solid thick lines represent the response for the braced frames in the linear elastic range. Dot, dash and solid lines correspond to similar structures using the ADAS and CFSB devices for 50%, 75% and 100% of the UBC-91 equivalent lateral forces, respectively.

The results indicate that, in general, the displacements follow the aforementioned observation by Newmark and Hall only if an average of the response of the ADAS and CFSB is compared to the elastic one. For periods smaller than about 0.3 s the roof displacements are considerably increased using the ADAS and CFSB connectors. As the strength of the dissipators is reduced the displacements increase in the whole range of periods considered. The roof displacements of buildings using the CFSB are smaller than the similar ones for buildings with the ADAS system. The top displacements of buildings using the ADAS are larger than those for the elastic buildings. For example, the average elastic roof displacement for a building with a period T=1.4 s is 10 cm and the corresponding displacements using the ADAS and CFSB devices designed with the 50% of the UBC-91 forces are 13.1 and 6.6 cm, respectively. These values are about 30% larger and 30% smaller than the mentioned elastic displacement.

It is clear that in practice a building would rarely be designed to stand a severe earthquake in the linear elastic range and consequently the aforementioned comparisons are rather conservative. Consequently, it is possible to conclude that the maximum displacements are not increased significantly using any of the energy dissipation systems compared to a traditional design, at least for the medium and long period ranges. Furthermore, the use of the CFSB dissipators reduces the maximum displacements compared to an elastic braced frame for periods larger than 0.3 s.

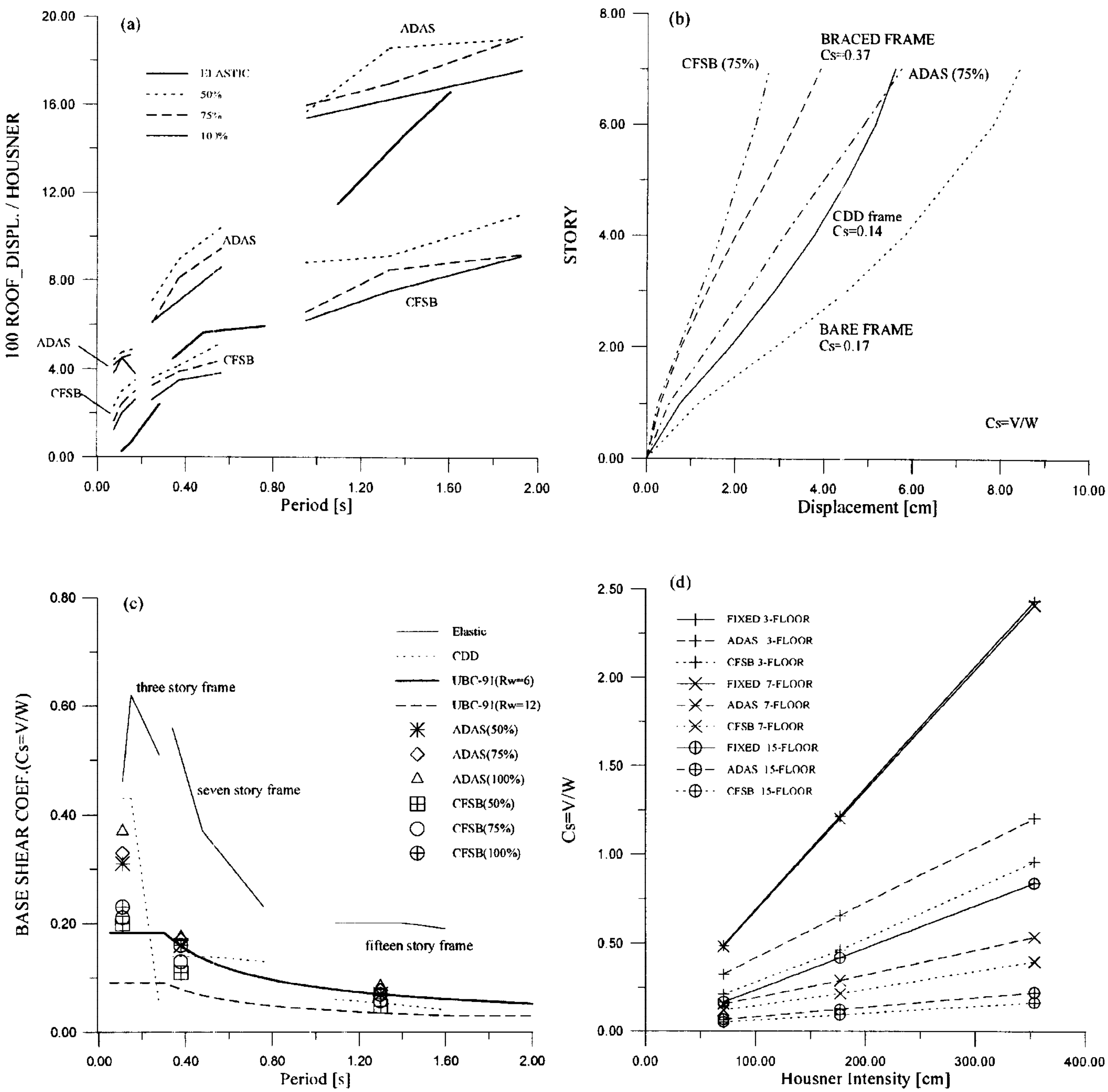

Fig. 2 (a) Average normalized maximum roof displacements. (b) Average maximum displacement. (c) Maximum base shear coefficient. (d) Base shear coefficient vs. Housner intensity.

Figure 2(b) presents the average maximum floor displacement for the seven-story frame with a fundamental period equal to 0.48 s. The lateral displacements using the CDD connection are about 40% larger than the displacements for a similar braced frame responding in the linear elastic range. The figure also illustrates the maximum horizontal displacements for a similar structure without the braces. The variation of the horizontal displacements along the height for the bare frame and for the CDD frame is very similar, at least for the relative stiffness selected in this study between the braced and the bare frame. The important difference between them is that the lateral displacements for the bare frame are about 50% larger than those for the CDD frame and more than twice the displacements for the braced frame. Not only the average maximum roof displacements follow the aforementioned trends, but also the maximum ones.

Figure 2(c) presents the average base shear coefficient corresponding to the lowest Housner intensity. The solid thin line represents the response for a similar conventional braced structure responding in the linear elastic range. The thick lines represent the design lateral forces using the UBC-91 code for a normal occupancy building (I=1.0) located in zone 4 (Z=0.4) with stiff soil conditions (S=1.0). The study includes two global ductility reduction factors: R_w=12 corresponds to a ductile moment resisting frame and, R_w=6 corresponds to a frame with an intermediate level of ductility. In general, the code does not allow to design a building in zone 4 with an intermediate level of ductility. However, this case is considered to define a practical design bound for the lateral forces. The dot line represents the response for a structure with the CDD dissipators. Symbol points represent the response for similar structures using the ADAS and CFSB devices for different percentages of the yielding and sliding threshold forces, respectively.

The results illustrated in the figure clearly indicate the reduction in forces that can be achieved using the dissipators, specially for the medium and long period ranges. In general, the base shear coefficient is smaller for frames using CFSB devices compared to similar frames using CDD and ADAS connections and much smaller than a traditional braced frame. In general, the base shear coefficient for a frame incorporating the CDD connections is an average between the corresponding values for frames incorporating the ADAS and the CFSB devices. For example, a seven-story structure with a fundamental period equal to 0.3 s has an average maximum base shear coefficient equal to 0.56 if it responds in the linear elastic range. The same structure with the CDD dissipators has a base shear coefficient equal to 0.14 which is similar to the design base shear for a moment resisting frame with R_w=6. The base shear for this frame using CFSB dissipators designed with a threshold sliding force equal to 50% of the axial force obtained using the UBC-91 lateral loads is 0.11. The base shear for a frame using ADAS dissipators and designed with a threshold yielding force equal to 50% of the axial force obtained using the UBC-91 lateral loads is 0.16. A structure designed according to the UBC is likely to have an overstrength factor of at least 50%, in particular taking into account the difference between working and ultimate strengths. Consequently, increasing the UBC-91 design base shear coefficient for the ductile moment resisting frame (R_w=12) by 50% gives C_s=0.13 which is very close to the design value using the CDD dissipators and even larger than the base shear for the CFSB dissipator. Clearly, the important difference is that the structure using the CDD, ADAS or CFSB dissipators responds in the linear elastic range while the design according to the UBC-91 requires a high level of global ductility.

For structures with a fundamental period larger than 0.3 the base shear is also reduced. For example the base shear for the fifteen-story frame structure with a period 1.4 s and using the CDD, ADAS and CFSB connections is 0.05, 0.07 and 0.05, respectively. Considering again the overstrength factor of 50%, these values are very close to the design base shear coefficient for a ductile moment resisting frame using the UBC-91.

In general it is clear that the base shear coefficient increases as the design threshold sliding or yielding force increases. However the increment is non proportional. For example, if the sliding design force for the seven story frame using CFSB devices increases from 50% to 100% of the UBC-91 equivalent lateral forces the base shear coefficient increase only from 0.11 to 0.16.

Figure 2(d) presents the base shear coefficient for different levels of Housner intensity. The solid, dash, and dot lines represent the response for the three, seven and fifteen story frames, respectively. The reduction of forces is not significant for the very stiff structure, although it is more relevant for the medium and long period structures. The reduction is similar for increasing values of Housner intensity. For example, a structure with a fundamental period of 0.34 s has a reduction of forces equal to 4 (0.56/0.14) corresponding to a Housner intensity of 71 cm. This structure subjected to the same earthquakes scaled to a Housner intensity of 195 cm has a reduction of forces of 3.5 (1.55/0.44).

EQUIVALENT NONLINEAR SINGLE DEGREE OF FREEDOM SYSTEM

The nonlinear single degree of freedom model proposed in this section has been derived from the multi-degree of freedom systems considered in the previous one by assuming that all the devices yield simultaneously. This assumption is consistent with the global design criteria since the yielding forces along the height were obtained using the equivalent lateral loads of the UBC-91 and, therefore, all the devices yield at the same time for the code static loads. In the present study it has been considered that, at least for the purposes of preliminary design, the devices plasticize simultaneously also under dynamic loads. The equivalent SDOF model has a bilinear constitutive relationship. The initial stiffness is obtained from the one of the conventional braced frame and, since all the devices are assumed to have a rigid-plastic behavior (as the CDD and CFSB), the reduced stiffness is equal to the one of the bare frame.

In Figs. 3(a) and 3(b) the maximum displacements versus the natural periods are plotted for the cases of the yielding forces in the devices equal to 50% and 100% of the forces obtained through the UBC-91 lateral loads, respectively. In both cases, the MDOF and the SDOF elastic systems have similar maximum displacements. The aforementioned Newmark and Hall statement is verified precisely for the two levels of lateral loads. For periods lower than about 0.5 s the maximum horizontal displacements are amplified depending on the strength of the single degree of freedom system. The maximum displacements for the nonlinear SDOF systems are in between those of the MDOF systems with ADAS and CFSB. Therefore, for MDOF systems, the Newmark and Hall observation is less approximate.

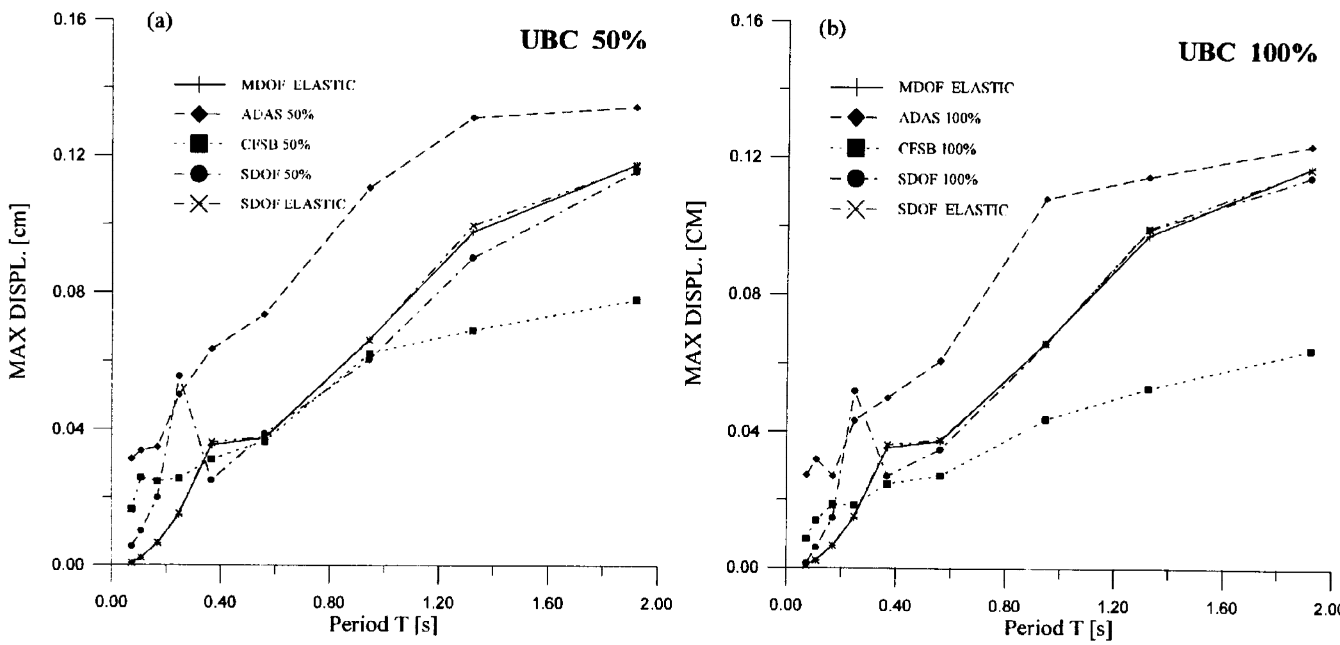

Fig. 3 Average maximum displacements for single and multidegree of freedom models. Yielding force equal to 50% (a) and 100% (b) of the force obtained with the UBC-91 lateral loads.

Figures 3(a) and 3(b) point out that the nonlinear SDOF model is conservative for the MDOF system with friction devices. For periods larger than 0.4 s the difference between them increases. The displacements for buildings with the ADAS devices are larger than those for the equivalent system. In general, the response of the ADAS connection is a parallel line above the one for the equivalent system.

CONCLUSIONS

The ADAS, CFSB and CDD energy dissipation systems efficiently reduce the earthquake induced forces, in particular for structures with a fundamental period larger than 0.3 s. In general, the CFSB devices designed with an axial threshold force equal to 50% of the design brace axial force according to the UBC-91 lateral forces produce the lowest seismic base shear coefficient. The CDD devices produce an intermediate level of seismic coefficient and the ADAS connection produce the highest one. Nevertheless, the difference is not very significant, specially for periods larger than about 0.3 s. In all the cases the base shear coefficient increases with the yielding or sliding threshold force. However, the variation of the seismic coefficient is not very significant with respect to the aforementioned design force.

The base shear coefficient for a structure with the ADAS, CFSB or CDD is usually reduced between one half and one fourth of the same coefficient for a similar braced structure without the devices. For structural periods larger than about 1 s such coefficient for a frame using the dissipators is similar to the design UBC-91 base shear for a ductile moment resisting frame. However, it is clear that the structure is much more protected using the ADAS, CFSB or CDD dissipators. For structural periods between 0.3 and 0.8 s the base shear coefficient for a frame using the dissipators is similar to the design UBC-91 base shear for an intermediate ductile moment resisting frame.

The lateral displacements for a frame using the CFSB device are smaller than those for a frame using the CDD or ADAS devices. For the intermediate and long period range the lateral displacements for a frame using the CFSB device are even smaller than the linear elastic displacements for the traditional braced frame.

In the period range between 0.3 and 0.8 s, the lateral displacements for a frame using the ADAS or CDD dissipators are larger than the lateral displacements for a similar traditional braced frame, although the displacements are smaller than the displacements for a similar traditional bare frame. For periods larger than 1 s the lateral displacements for a frame using the CDD dissipators are smaller than the lateral displacement for a similar braced frame. For the same period range the displacements for a frame using the ADAS dissipators are larger than the lateral displacements for a similar braced frame.

REFERENCES

Aiken, I.D. and Kelly, J.M. (1990). *Earthquake Simulator Testing and Analytical Studies of Two Energy-Absorbing Systems for Multistory Structures* UCB/EERC report 90/03. Earthquake Engineering Research Center, University of California at Berkeley.

Akbay, Z. and Aktan, H. M. (1990). Intelligent Energy Dissipation Devices. *Proceedings of the Fourth U.S. National Conference on Earthquake Engineering,* 427-435, Palm Springs.

Bozzo, L.M. and Barbat, A. (1996). Nonlinear Response of Structures with Sliding Base Isolation. *Journal of Control* (to be published).

Inaudi, J.A. and Kelly, J.M. (1992). *A Friction Mass Damper for Vibration Control* UCB/EERC report 92/15. Earthquake Engineering Research Center, University of California at Berkeley.

Hanson, R.D., Xia, C. and Su, Y.F. (1992). Design of Supplemental Steel Damping Devices for Buildings. *Proceedings of the Tenth World Conference on Earthquake Engineering* 4139-4142 Madrid, Spain.

Newmark, N., and Hall, W.(1982). *Earthquake Spectra and Design*, Engineering Monographs on Earthquake Criteria, Structural Design and Strong Motion Records, EERI-v3.

Pong, W.S., Tsai, C.S., and Lee, G.C. (1994). *Seismic Study of Buildings Frames with Added Energy-Absorbing Devices* Technical Report NCEER-94-0016, State University of New York at Buffalo.

Scholl, R. (1990). Improve the Earthquake Performance of Structures with Added Damping and Stiffness Elements. *Proceedings of the Fourth U.S. National Conference on Earthquake Engineering,* 489-498, Palm Springs.

Xia, C. and Hanson, R.D. (1992). Influence of ADAS Element Parameters on Building Seismic Response. *Journal of Structural Engineering, ASCE, 118*, 1903-1918.