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COMPARATIVE STUDY OF A MULTISTORY FRAME WITH ENERGY DISSIPATION DEVICES AND WITHOUT THEM

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ABSTRACT

Two multistory reinforced concrete frames are analyzed and compared according to their seismic responses and initial contructions costs; one designed with energy dissipation devices according to a criterion described in this paper, and the other conventionally designed. Both frames have the same dynamic characteristics. The frames are excited with a set of eleven simulated accelerograms. The following responses are compared: maximum overturning moments developed at the foundation, maximum axial force at the same level, maximum floor displacements, displacements ductility demands and plastics hinges in the frames. The initial construction and installation costs of the energy dissipation devices (TADAS) are also compared.

KEYWORDS

Energy dissipation devices, TADAS, construction costs, statistical response, multistorey frames, structural design.

INTRODUCTION

Frequently, the structural system ductility is used as a main resource to absorb and dissipate the extraordinary load demands that earthquakes impose. In the lasts years, studies on external dissipation devices have been intensified. These devices are capable of reacting to the seismic excitations, reducing the damage that those excitations may induce on the main structure.The statistical seismic response of a conventional reinforced concrete frame and of a frame with dissipators are compared in this study. It also includes an evaluation of initial construction costs of both frames.

STRUCTURAL MODELS

Two twenty-story reinforced concrete frames are analyzed in this paper. The first one was conventionally designed, and the second one was provided with energy dissipation devices located along the heigh of the central bay. These frames are shown in figure 1.

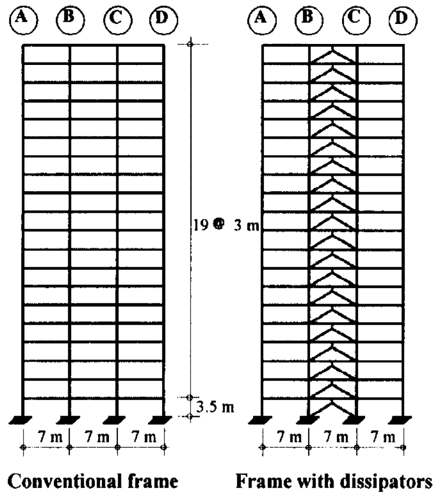


Fig. 1. Frames analyzed in this study

The convetional frame was designed in accordance with Federal District Seismic Regulations (FDSR, 1987). The frame with dissipators was designed following the criteria mentioned below. The dissipation devices work exclusivelly in bending. They are similar to the system named TADAS (Tsai et al, 1993). They are assumed to have bilinear hysteretic behaviour.

The masses lumped at the floor levels of the structural models range between 16.83 t s^2 m^{-1} (11.28 kips-$s^2$$ft^{-1}$) at the first level and 12.55 t s^2 m^{-1} (8.41 kips-$s^2$$ft^{-1}$) at the uppermost level. The static analyses were performed with the **R.C. BUILDINGS** (1994) computer program, and the dynamic analysis with the **DRAIN-2D** (Kannan and Powell, 1973) program. The frames were assumed to be located on the soft soil zone of Mexico City. This zone its highly compressible. Soil-structure interaction was not taken into account.

A set of eleven simulated accelerograms based on the E-W component of the record obtained at the Ministry of Communication and Transportation station in Mexico City on September 19,1985 (SCT-1985) was used in this study.

BASIC DEFINITIONS

Some terms used in this paper are defined, in order to facilitate the understanding of the design procedure.

Conventional frame (CF): Frame designed in a conventional way, according to FDSR 1987. Its lateral story-stiffness and resistance, denoted respectively by $\mathbf{K_T}$ and $\mathbf{R_T}$, are given by columns and girders exclusively.

Conventional frame subsystem (CFS): This is a frame similar to the CF, but with all its member cross sections uniformily reduced. The CFS story-stiffness is defined as $\mathbf{K_{wd}} = \alpha_1\mathbf{K_T}$, and the story resistance as $\mathbf{R_{wd}} = \beta_1\mathbf{R_T}$, where α_1 and β_1 are smaller that unity.

Dissipation system (DS): This DS is added to the CFS. Its lateral stiffness is defined as $\mathbf{K_d} = \alpha_2\mathbf{K_T}$, and its story resistance as $\mathbf{R_d} = \beta_2\mathbf{R_T}$. Therefore, a combined frame-dissipator (FD) system is obtained with lateral stiffness $\mathbf{K_T} = \mathbf{K_{wd}} + \mathbf{K_d}$, and resistence $\mathbf{R_T} = \mathbf{R_{wd}} + \mathbf{R_d}$. Figure 2 shows the concepts defined above.

DIMENSIONS OF THE FRAMES

One objetive of the study is to compare the statistical response of the CF with the FD systems. For this reason, it is convenient to start from the CF design when trying to obtain the FD design. Two conditions are imposed to make this comparative analysis: (1) To have the same vibration period, and (2) The values of α_1 and α_2 remain constant during the entire process. It is stressed that in a practical design situation these conditions are not necessary; however, in this study it was considered adequate, in order to compare the structural responses.

Convetional frame (CF)

A plane-frame model was used in this study to obtain the preliminary cross sections of the CF. Several iterations with different cross sections were made in order to obtain a CF frame with a fundamental period of vibration equal to 2 s, which complies with FDSR specifications. The dominant period of vibration of the SCT-1985 record is 2 s. The cross section dimensions obtained for the columns of the first four levels are equal to 1.20 × 1.20 m; those of the last four levels are 0.80 × 0.80 m, and those of the girders are equal to $b \times h = 0.45 \times 0.95$ m. The concrete strength is assumed equal to 2000 t m^{-2}. The lateral interstory-stiffness $\mathbf{K_T}$ ranges from 26281 t m^{-1} at the first level to 3269 t m^{-1} at the top level.

Conventional frame subsystem (CFS)

From the convetional frame (CF) a new structure (CFS) with uniformily reduced element cross sections is obtained. The dimensions ($b' \times h'$) of the CFS elements are related to those of the CF elements ($b \times h$) as follows (Silva, 1993): $b' = \sqrt[4]{\alpha_I}\, b$ *and* $h' = \sqrt[4]{\alpha_I}\, h$. These expressions imply that the frame stiffness depends only on the moments of inertia of the cross sections elements. Several studies (Silva et al, 1994) were performed on structural frames, taking into account the α_1 and α_2 values. Those studies have demostrated that an adequate structural response is obtained for values of α_1=0.25 and α_2=0.75. These values are used in this stage.

The cross section dimensions of the columns for the first four levels are equal to 0.85 × 0.85 m, those at the top levels are equal to 0.57 × 0.57 m, and those of the girders are equal to $(b' \times h') = 0.32 \times 0.67$ m. The CFS lateral stiffnes $\mathbf{K_{wd}}$ is assumed approximately equal to **0.25** $\mathbf{K_T}$. The CFS period of vibration with uniformily reduced element cross sections is 3.8 s. Steel braces and dissipation devices (TADAS) were added to the CFS in order to increase the CFS lateral stiffness and to obtain a period of vibration similar to that of CF.

RESISTANCE OF THE FRAMES

Convetional frame (CF)

The CF was designed for a seismic coeffcient equal to 0.2g (g : gravity) in accordance with the FDSR (using a seismic behaviour factor Q = 2). The design is governed by the worst of the following load effect combinations: **1)** $1.4\,(D + L_{max})$ and **2)** $1.1\,(D + L_{inst} + E)$, where 1.4 and 1.1 are load factors; D, L_{max}, L_{inst} and E represent dead, live and earthquake load effects, respectively. The longitudinal steel in the columns ranges from 1.03 to 3.30 percent of the total cross section area. In the girders it varies from 0.5 to 1.3 percent.

Combined frame-dissipator system (FD)

The criterion of anlysis for the FD assumes that the CFS mainly resists vertical load and that the dissipation system takes only horizontal effects. The following two loading conditions need to be analyzed:

I) The conventional frame subsystem (CFS) under the exclusive action of vertical loads, with the two following load combinations: **I.1)** $1.4\ (D + L_{max})$ and **I.2)** $1.1\ (D + L_{inst})$.

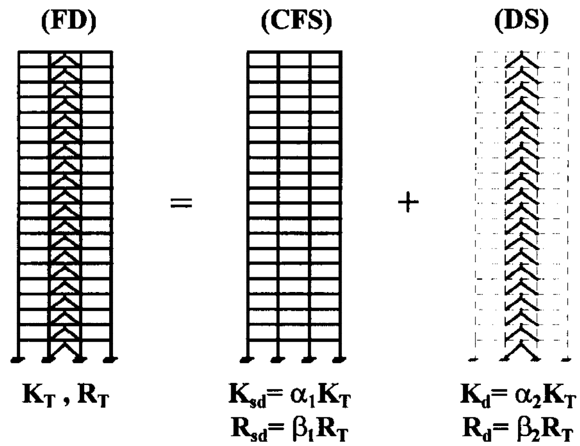


Fig. 2 Stiffnes and strenght of the combined frame dissipator system.

II) The combined frame dissipator system (FD) under the exclusive action of horizontal loads. In this case the design load is *1.1 E*. The load combination effects **I.2)** and **II)** are added up to obtain the total design load. The frame is designed for the worst load combination.

A modal spectral analysis (using a seismic behaviour factor Q = 1), according to Mexico City soft-soil design spectrum was carried out.

The next step in the design consist in applying an adequate reduction factor ϕ_D to the internal forces of the dissipation system. In order to select the ϕ_D values, several dynamic step-by-step analyses were carried out. The frame was excited with SCT-1985 accelerogram. The ϕ_D values were adjusted by trial and error, until achieving an acceptable behaviour of the dissipation devices and tolerating little damage in the concrete frame.

SEISMIC STATISTICAL RESPONSE OF THE FRAMES

The CF and FD response under a set of eleven simulated accelerograms is evaluated in this stage. The variables selected to describe the response are mean values and standard deviations of maximum interstorey displacement, maximum axial forces, maximum overturning moments at the foundations and displacement ductility demands at dissipators, as well as plastics hinges at the frames. The results are described in the following.

Maximium horizontal displacements

The mean values and standard deviations (σ) of the maximium floor displacements relative to the base are shown in Table 1. Notice that in the top levels the FD displacements are larger than those of CF; however, those in the lower levels are similar.

Maximum interstorey drifts

The mean values of maximum interstorey drifts of the FD range between 4.04 cm (level 5) and 1.70 cm (top level). They are also shown in Table 1. Standard deviations range between 0.61 cm and 0.10 cm. Both frames present very similar mean interstorey drifts. This means that their damage levels must be similar. This permits to make some comparisons in these structures.

Table 1. Statistics of maximum floor displacements (left) and maximum interstorey drifts (right, in cm)

Floor	C F		F D	
	Mean	σ	Mean	σ
20	53.42	4.20	62.50	4.79
19	52.31	4.21	60.79	4.72
18	50.87	4.37	58.69	4.66
17	49.09	4.42	56.21	4.54
16	47.47	4.46	53.60	4.39
15	45.72	4.49	50.84	4.29
14	43.81	4.51	47.95	4.19
13	41.85	4.40	44.90	4.09
12	39.73	4.32	41.77	4.02
11	37.39	4.24	38.58	3.94
10	34.89	4.13	35.33	3.85
9	31.77	3.79	32.01	3.75
8	28.11	3.52	28.59	3.59
7	24.21	3.05	24.96	3.23
6	20.27	2.55	21.15	2.77
5	16.26	2.00	17.11	2.25
4	12.35	1.48	13.08	1.79
3	8.60	1.02	9.13	1.22
2	5.14	0.68	5.37	0.74
1	2.16	0.42	2.10	0.32

Storey	C F		F D	
	Mean	σ	Mean	σ
20	1.10	0.15	1.70	0.10
19	1.44	0.22	2.10	0.17
18	1.78	0.20	2.48	0.19
17	1.63	0.20	2.61	0.23
16	1.75	0.18	2.76	0.19
15	1.91	0.15	2.89	0.20
14	1.96	0.24	3.05	0.20
13	2.12	0.19	3.14	0.24
12	2.34	0.26	3.19	0.20
11	2.49	0.19	3.25	0.20
10	3.13	0.46	3.32	0.26
9	3.65	0.31	3.42	0.32
8	3.89	0.52	3.62	0.48
7	3.94	0.54	3.82	0.56
6	4.01	0.58	4.03	0.61
5	3.91	0.56	4.04	0.54
4	3.74	0.50	3.95	0.57
3	3.47	0.40	3.75	0.48
2	2.98	0.30	3.27	0.42
1	2.16	0.42	2.10	0.30

Ductility demands of dissipation devices

The mean values and standard deviations of the ductility demands of the dissipation devices ($\delta_{max}/\delta_{yield}$) are shown in Table 2. Notice that the largest ductility demand at the dissipation devices occurred at the lowest seven stories. The maximum mean value is 2.87. That is, ductility demands at dissipators are small in this case. At the top level (20th) a ductility demand smaller than unity was obtained, which means that the dissipation device had an elastic response.

Maximum axial forces on the foundation

The mean values and standard deviations of the vertical forces on the foundation, resulting from the superposition of gravity and seismic actions, are shown in Table 3. These forces were obtained from the combination of gravitational and seismic actions. They take into account the load contributions transmitted by the columns and braces (FD case). Table 3 shows that the largest mean values of vertical loads acting on the CF foundation correspond to the extreme columns A and D (see figure 1); however, for the frame with dissipators these values correspond to the inner columns B and C. This force increment is due to the contribution of the diagonals located on the central bay (see figure 1).

Maximum overturning moments at the foundation

Seismic overturning moments are obtained adding the products of the shear forces induced by the earthquake on the columns by the corresponding interstorey heights. The histories of the overturning moment for CF and FD subjected to the SCT-1985 excitation are shown in figure 3. Table 4 shows the mean values and standard deviations of the maximum overturning moments. This table shows that the moment for FD is 29 % smaller than for the conventional frame. This clearly indicates that FD presents an advantage regarding the initial costs of the foundation.

Table 2. Displacement ductility demands

Storey	Mean	σ
20	0.38	0.02
19	1.19	0.10
18	2.12	0.22
17	2.16	0.23
16	2.23	0.22
15	2.14	0.20
14	1.98	0.18
13	2.15	0.18
12	2.33	0.20
11	2.13	0.18
10	1.96	0.18
9	1.96	0.29
8	2.14	0.34
7	2.33	0.27
6	2.55	0.17
5	2.71	0.20
4	2.87	0.19
3	2.85	0.23
2	2.69	0.26
1	2.53	0.16

Table 3. Statistics of the maximum vertical load acting on the foundation (ton)

Frame		Column A		Column B		Column C		Column D	
		C	T	C	T	C	T	C	T
C F	Mean	1604.99	958.25	647.33	—	645.77	—	1606.52	957.22
	σ	20.95	21.28	2.40	—	2.18	—	18.95	19.52
F D	Mean	756.20	413.56	1358.92	749.37	1356.61	750.23	755.84	414.05
	σ	15.38	22.45	38.40	39.67	35.82	37.37	17.23	20.41

C = compression force , T = tension force

Table 4. Statistics of maximum overturning moments (ton.m)

Frame	Maximum (+)		Maximum (−)	
	Mean	σ	Mean	σ
C F	30223.08	399.61	30285.17	500.98
F D	21530.41	655.94	21594.76	643.89

Hysteretic loops at the dissipators

The bilinear hysteretic behaviour of the dissipation devices corresponding to storeys 3, 9, 13 and 19 subjected to SCT-1985 accelerogram are presented in figure 4. The flexural behaviour of the dissipating devices is studied by plotting bending moments at the plastic hinges vs. the corresponding plastic rotations. Notice that the dissipation devices absorb more energy at the lower levers than at the upper ones. A possible alternative structural solution could be to place dissipation devices only on the lower levels of the buildings (Ruiz, E., 1995).

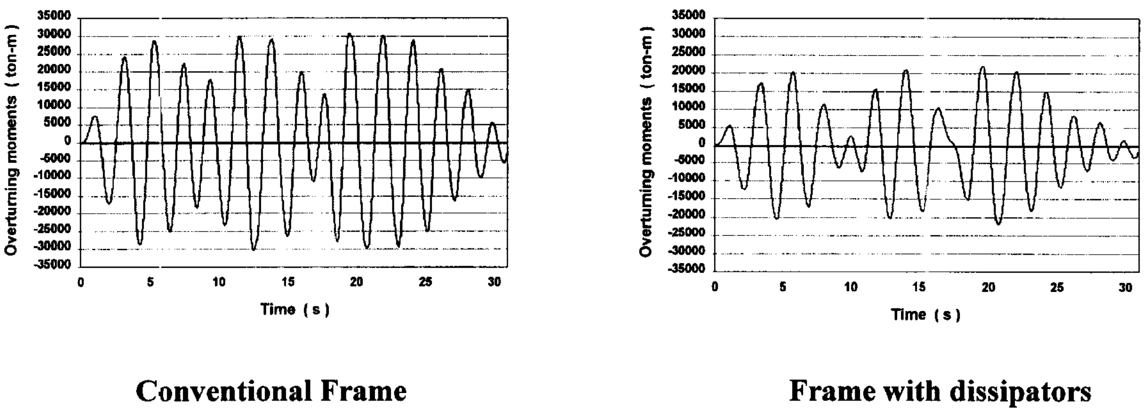


Fig. 3. Overturning moments in the frames subjected to SCT-1985 accelerogram

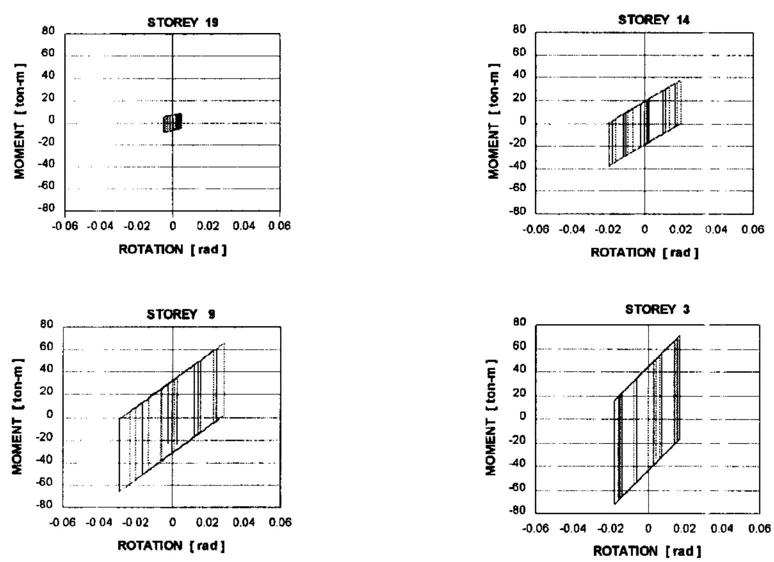


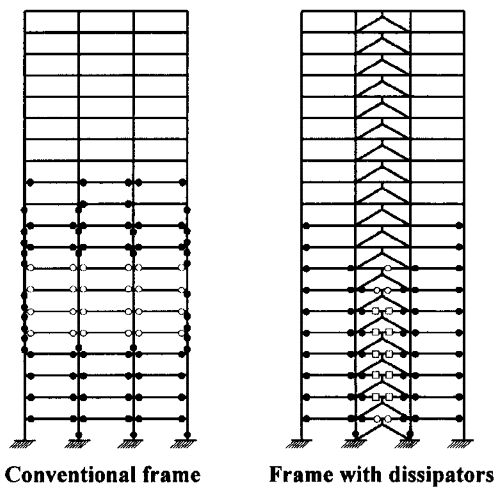
Fig. 4 Bilinear behaviour of 4 dissipators. Excitation: SCT-1985 accelerogram.

Plastics hinges at the frames

The locations of the plastic hinges at both frames are shown in figure 5, which also shows the ranges of the maximum plastic rotations at those locations. It can be seen that the damage level in FD is acceptable, because it is governed by the plastic hinges at the girders, and at the base of the middle storey columns (see figure 5). The ocurrence of plastic hinges in these storeys propitiated the occurrence of larger interstorey drifts also in this zone, (see Table 1).

COMPARISON OF INITIAL COSTS

An analysis of initial costs of both frames is performed in the following. The cost include those associated to the installation of the dissipation system (Mejía, 1995). The cost of FD is higher (45.46 percent) than that of the CF. However, this difference is smaller if the comparison include the total initial cost of both buildings, including nonstructural elements, finishings, etc. In the latter case the cost increase of the building with energy dissipation devices is only 9.40 percent (assuming that the building is formed by four frames in two orthogonal directions). The foundation costs not are taken into account at this stage. The economic comparison is referred to direct costs. The net profit and the indirect costs are specific to each firm.



● $0.000 < \theta_{max} \leq 0.013$ ○ $0.013 < \theta_{max} \leq 0.026$ □ $0.026 < \theta_{max} \leq 0.039$

Fig. 5 Plastics hinges in the frames; rotations in radians

CONCLUSIONS

This study has provided some useful concepts to improve our understanding of the behaviour of buildings with energy dissipation devices, as well as about their design philosophy. The following conclusions were obtained.

1. If buildings with dissipations devices are properly designed, earthquake damage in them can be signicantly reduced.
2. For the case analyzed maximum mean values of the displacement ductility demands at the dissipation devices resulted smaller than their ductile capacity. Those demands ranged between 2 and 3. This was a consequence of establishing the condition that the concrete frame suffered a damage comparable to that of the conventional frame.
3. The largest amount of energy dissipation occurred at the devices of the lowest storeys (see figure 4).
4. The maximum overturning mean values in FD are 29 percent smaller than those of the conventional frame. This indicates that the FD presents some advantage regarding the initial costs of the foundation.
5. The maximum mean values of vertical forces acting at the foundation are bigger (112 %) at the extreme columns of the CF than those at the FD; however, the forces in the inner columns (B and C axies) of FD are bigger (110 %) than those of CF.
6. The inelastic structural response of the frame with dissipators was better than that of CF (see figure 5).

7. The cost of the dissipation system represents 23.2 % of the total initial construction cost of the building with dissipators.
8. The total initial construction cost of the frame with dissipator is 9.4 percent higher than that of the conventional frame.
9. The differences in construction costs of buildings studied here are larger than those obtained for a conventional building and for a 10 storey concrete frame with dissipators analyzed previously (Ruiz et al, 1995).

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