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SEISMIC UPGRADING OF GRAVITY-LOAD DESIGNED RC FRAME STRUCTURES BY USING HYSTERETIC DAMPERS

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SUMMARY

Many existing RC frame structures situated in low to moderate seismicity regions are susceptible to undergo severe damage if they are subjected to the maximum credible earthquake expected in these regions. This research investigates an advanced solution for seismic upgrading these type of structures, which consists in installing hysteretic dampers in all stories. This paper proposes the design criteria and a practical methodology for designing the dampers. The methodology is validated through numerical simulations which show that the dampers control the lateral drifs, and hence, the damage on the frame.

INTRODUCTION

A large number of existing reinforced concrete (RC) frame structures located in low to moderate seismicity regions have been designed mainly for gravity loads, and their lateral resistance has been determined without seismic considerations or according to old seismic codes, in which ductile detailing was not explicitly required. It is very unlikely that these buildings, if subjected to a maximum credible seismic event, satisfy basic seismic design criteria such as: (1) to develop a failure mechanism that maintains service load capacity, (2) to prevent the brittle fracture of the structural members, and (3) to withstand large rotations in plastic hinge zones without crushing of concrete in the compressive zone. Kunnath [1] evaluated the seismic capacity of gravity-load-designed (GLD) RC frames located in the eastern and central United States (zones classified of low to moderate seismicity), and concluded that these frames are susceptible to severe damage when subjected to an intense ground shaking at peak ground accelerations (PGA) within the design spectra. Benavent-Climent [2] investigated the seismic behavior GLD RC frames located in the southern part of Spain (also a zone classified of low to moderate seismicity) and concluded that there the ultimate energy dissipation capacity was about one half of the required level. Seismic upgrading of existing buildings raises problems more troublesome than the design of new buildings. On the other hand, given the expected return period and probability of occurrence of an intense ground shaking in these regions, costly upgrading solutions may no be economically viable unless the buildings are critical to maintain essential services to the community following such event. Therefore, finding an effective solution for seismic upgrading this type of frames is a hot issue.

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The different strategies for seismic upgrading existing buildings can be divided into two groups: (a) the ordinary methods and (b) the advanced methods. The basic principle of ordinary methods is to either increase the lateral shear force resistance of the building without impairing the plastic deformation capacity, or to improve the plastic deformation capacity of the frame. In contrast, the strategy of the advanced methods is to change the ordinary structural type into a more preferable one, such as the so-called [3] "flexible-stiff mixed structure".

The "flexible-stiff mixed structure" consists of two parts: (a) the "stiff part", which is equipped with high rigidity and high energy absorption capacity, and (b) the "flexible part" which is equipped with low rigidity and large elastic deformation capacity. The practical applications of the flexible-stiff mixed structures are divided into two types: the energy-concentrating type and the energy-dispersing type. In the energy-concentrating type, the seismic input energy is intentionally concentrated and absorbed in one story by a mechanism specially arranged for that purpose. One example of this type is the base isolated structures in which the laminated rubber bearings and the dampers constitute the "flexible part" and the "stiff part" respectively. In the energy-dispersing type, the seismic input energy is intended to be evenly distributed all over the building, and absorbed by energy dissipating devices installed in every story. One example of this type of structures is the combination of a moment resisting frame and dampers.

This research investigates an advanced energy-dispersing type method for seismic upgrading existing GLD RC moment resisting frames. The method consists in transforming the GLD moment resisting RC bare frame into a "flexible-stiff mixed structure" by installing hysteretic dampers in all stories. The original RC bare frame plays the role of the "flexible part", and the hysteretic dampers constitute the "stiff part" of the mixed structure. This paper proposes the general criteria and a practical methodology for designing the hysteretic dampers. The method is based on the Housner-Akiyama energy theory [3]. The method is validated through dynamic response analyses of several frames subjected to historical accelerograms. The results of the analyses show that the required lateral strength and stiffness of the hysteretic dampers calculated according to the proposed method are adequate for controlling the structural damage and for preventing the collapse of the frame.

THEORETICAL BACKGROUND

One of the most promising approaches for the seismic design of building structures is the Housner-Akiyama theory based on the energy balance. Although the concept of energy based seismic design was proposed by Takahashi [4] and Housner [5] early in 1956, it is not until recently that this approach has gained increased attention since the works and design methods established by Akiyama [3]. The theoretical background of this approach can be summarized as follows. The horizontal oscillations of a discrete multi-degree-of-freedom (MDOF) model subjected to an unidirectional horizontal ground motion are governed by the following matrix differential equation:

$$\mathbf{M}\ddot{\mathbf{y}}(t) + \mathbf{C}\dot{\mathbf{y}}(t) + \mathbf{Q}(t) = -\mathbf{M}\mathbf{r}\,\ddot{z}_g(t) \tag{1}$$

where $\mathbf{M}$ and $\mathbf{C}$ are the mass and damping matrices respectively; $\mathbf{Q}(t)$ is the restoring force vector; $\ddot{\mathbf{y}}(t)$ and $\dot{\mathbf{y}}(t)$ are, respectively, the acceleration and velocity vectors relative to the ground; $\mathbf{r}$ is the influence coefficient vector which represents the displacement vector $\mathbf{y}(t)$ resulting from a unit support displacement; and $\ddot{z}_g(t)$ is the acceleration of the input ground motion. Multiplying Eq.(1) by $\dot{\mathbf{y}}(t)dt$ and integrating from the instant the earthquake starts, t=0, until a given instant t, the equation of the energy balance of the structure at instant t is obtained:

$$W_k(t)+W_\xi(t)+W_s(t)=E(t) \tag{2}$$

where:

$$W_k(t)=\int_0^t \dot{\mathbf{y}}^{\mathbf{T}}\mathbf{M}\ddot{\mathbf{y}}dt \;;\; W_\xi(t)=\int_o^t \dot{\mathbf{y}}^{\mathbf{T}}\mathbf{C}\dot{\mathbf{y}}dt \;;\; W_s(t)=\int_o^t \dot{\mathbf{y}}^{\mathbf{T}}\mathbf{Q}\,dt \;;\; E(t)=-\int_0^t \dot{\mathbf{y}}^{\mathbf{T}}\mathbf{Mr}\ddot{z}_g\,dt$$

$W_k(t)$ is the kinetic energy at the instant t; $W_\xi(t)$ and $W_s(t)$ are the energy absorbed by damping and the energy dissipated by the spring system respectively up to the instant t; in turn, $W_s(t)$ is the sum of the plastic strain energy cumulated by the whole structure up to the instant t, $W_p(t)$, and the elastic strain energy stored by the structure at the instant t, $W_{se}(t)$, i.e. $W_s(t)= W_p(t)+W_{se}(t)$; $E(t)$ is the total amount of energy input by the earthquake up to the instant t. The sum of $W_k(t)$ and $W_{se}(t)$ constitutes the elastic vibrational energy of the system, $W_e(t)$, that is $W_e(t)=W_k(t)+ W_{se}(t)$. By using the new notation Eq.(2) can be rewritten as follows:

$$W_e(t)+W_\xi(t)+W_p(t)=E(t)\,. \tag{3}$$

The left hand side of Eq.(3) is the structural energy absorption/dissipation capacity and it can be interpreted as the seismic resistance of the structure. The right hand side represents the earthquake loading effect in terms of input energy. Akiyama [3] showed that the total input energy E imparted to the structure by a given earthquake up to the instant t_o that the quake fades away, i.e. $E(t_o)$, is a very stable amount that depends mainly on the total mass, M, and the fundamental natural period of the structure, T. This fact constitutes the basis of the so-called *energy based seismic design* (EBSD) approach. The term $W_p(t)$ in Eq.(3) is a quantitative description of the *damage caused by the earthquake to the structure.* The difference between $E(t)$ and $W_\xi(t)$, is the so-called *input energy attributable to damage,* $E_D(t)$:

$$E_D(t)=E(t)-W_\xi(t) \tag{4}$$

By substituting Eq.(4) in Eq.(3), the energy balance of the structure can be rewritten as follows:

$$W_e(t)+W_p(t)=E_D(t) \tag{5}$$

$E(t)$ and $E_D(t)$ can be expressed in terms of equivalent velocities, $V_E(t)$ and $V_D(t)$, by:

$$V_E(t)=\sqrt{2E(t)/M_T} \qquad ; \qquad V_D(t)=\sqrt{2E_D(t)/M_T}\;. \tag{6}$$

Above equations can be applied for the prediction of the response of flexible-stiff mixed structures [6], [7]. Lets denote by t_m the instant the structure reaches it maximum lateral displacement. On one hand, at $t=t_m$ Eq.(5) can be rewritten as follows:

$$W_e(t_m)+W_p(t_m)=E_D(t_m) \tag{7}$$

It has been shown [6] that when the inelastic strain energy W_p consumes an important part of the total energy input attributable to damage E_D, such is the case of the structures subjected to earthquakes, $E_D(t_m)\leq E_m(t_o)$. For design purposes we can make the safe-side assumption that $E_D(t_m)=E_m(t_o)$. Accordingly, Eq.(7) yields:

$$W_e(t_m)+W_p(t_m)=E_D(t_o) \tag{8}$$

On the other hand, at the instant t_o that the seismic motion fades away, the elastic vibrational energy, W_e, is almost 0, therefore Eq.(5) can be approximated by:

$$W_p(t_o)=E_D(t_o) \tag{9}$$

The seismic response of a "flexible-stiff mixed structure can be predicted on the basis of Eqs. (8) and (9).

DESIGN CRITERIA FOR THE HYSTERETIC DAMPERS

In the "flexible-stiff structure" here we deal with, the GLD RC frame constitutes the "flexible part", and the collection of hysteretic dampers installed in each story form the "stiff part" of the mixed structure. The hysteretic dampers are designed on the basis of the following two criteria.

Maximum lateral displacement criterion.

The hysteretic dampers must guaranty that the maximum interstory drift in each *i*-story, $\delta_{max,i}$ is smaller than a maximum allowed value, $\delta_{a,i}$, in order to prevent severe damage in the RC frame, that is, $\delta_{max,i} \le \delta_{a,i}$. The maximum interstory drift $\delta_{max,i}$ can be predicted [8] by the following expression, based on Eq.(8):

$$\delta_{\max,i} = \frac{(\sum_{j=i}^{N} m_j g)\bar{\alpha}_i \alpha_e}{{}_f k_i} \, {}_s r_{m1} \left[\sqrt{\left(2Cc_e\right)^2 + \left(\frac{4Cc_e}{K}\right) + \frac{1}{{}_s r_{m1}^2}} - 2Cc_e \right] \tag{10}$$

where m_i is the mass of the *i*-th story; *N* is the total number of stories; *g* is the gravity acceleration; ${}_f k_i$ is the lateral stiffness of the *i*-th story considering only the bare frame; $K={}_s k_i / {}_f k_i$ where ${}_s k_i$ is the lateral stiffness provided by the hysteretic dampers installed at the *i*-th story (*K* is taken equal for all stories); and c_e is an empirical coefficient that relates the cumulative and the maximum inelastic deformation in each story (c_e is taken equal for all stories). The rest of coefficients in Eq.(10) are defined as follows:

$$C = \frac{\gamma_1}{\chi_1} \quad ; \quad \chi_1 = \frac{{}_f k_1 T_f^2}{4\pi^2 M} \quad ; \quad \gamma_1 = \sum_{i=1}^{N} s_i \quad ; \quad s_i = \left(\sum_{j=i}^{N} \frac{m_j}{M}\right)^2 \frac{{}_f k_1}{{}_f k_i} \bar{\alpha}_i^2 \tag{11a}$$

$${}_s r_{m1} = \frac{{}_s \alpha_1}{\alpha_e} \quad ; \quad \alpha_e = \frac{2\pi \, V_D(t_o)}{g \, T_f} \qquad {}_s \alpha_i = \frac{{}_s Q_{y,i}}{\sum_{j=i}^{N} m_j g} \tag{11b}$$

here, T_f is the fundamental period of the structure considering only the bare frame; *M* is the total mass of the building; ${}_s Q_{y,i}$ is the yield shear force of the hysteretic dampers installed at the *i*-th story; and $\bar{\alpha}_i = \alpha_{opt,i} / \alpha_1$ is the optimum distribution of the yield shear force coefficient proposed by Akiyama [3], which is given by the following expression, where $x' = (i-1/)N$:

$x' \le 0.2$: $$\bar{\alpha}_i = 1 + 0.5x' \tag{12a}$$

$0.2 < x' \le 1.0$: $$\bar{\alpha}_i = 1 + 1.5927x' - 11.852x'^2 + 42.583x'^3 - 59.48x'^4 + 30.16x'^5 \tag{12b}$$

In deriving Eq.(10) it was assumed that the distribution of the yield shear force coefficient ${}_s\alpha_i / {}_s\alpha_1$ in the hysteretic dampers was the optimum distribution given by Eq.(12), that is, ${}_s\alpha_i = {}_s\alpha_1 \bar{\alpha}_i$.

Ultimate energy absorption capacity criterion.

The ultimate energy absorption capacity of the hysteretic dampers must be large enough to dissipate the total seismic input energy attributable to damage when the quake fades away, $E_D(t_o)$. The ultimate energy absorption capacity of the hysteretic damper can be quantified by two empirical coefficients *u* and *b* that depend on the mechanical properties of the steel and the geometry of the energy absorbing device [8]. In a "flexible-stiff mixed structure", the ${}_s r_1$ required for using up the energy absorption capacity of the hysteretic damper, ${}_s r_{u1}$, can be related to the coefficients u and b by the following expression proposed by Benavent-Climent [8], which is based in Eqs.(8) and (9):

$$ {}_s r_{u1} = \left[\frac{2uC}{K} \left\{ \frac{b}{u} + 4c_e^2 CK + 2c_e + 4c_e^2 uK + \sqrt{16c_e^4 K^2 C^2 (u+1)^2 + 16c_e^3 KC(1+u+0.5c_e^{-1} b)} \right\} \right]^{-0.5} \quad (13) $$

METHODOLOGY

The methodology proposed in this paper can be summarized as follows. First, the mechanical properties of the bare RC frame ${}_f k_i$, ${}_f\delta_{y,i}$, T_f, χ_1, γ_1, M are obtained. Here, ${}_f\delta_{y,i}$ is the yield interstory drift of the bare frame and the rest of quantities were already defined in previous paragraphs. Second, the seismic input energy demand expressed in terms of the equivalent velocity V_D is determined according to the site and soil conditions. Design values of V_D have been proposed by Akiyama [3] for high seismicity regions such as Japan, and by Benavent-Climent [9] for moderate seismicity regions such as the Mediterranean area. Last, the required lateral stiffness ${}_s k_i$ and lateral strength ${}_s Q_{y,i}$ that must be provided by the dampers installed in each story is determined as follows. ${}_s k_i$ is taken proportional to ${}_f k_i$ in all stories. The ratio $K={}_s k_i/{}_f k_i$ must satisfy the following conditions [8]:

$$ K \geq \frac{\chi_1}{c_e \gamma_1} \left(\frac{1-\beta_i^2}{\beta_i^2} \right) \quad \text{and} \quad K \geq 10 \quad (14) $$

where

$$ \beta_i = \frac{\delta_{\max,i}\ {}_f k_i}{\left(\sum_{j=i}^{N} m_j g \right) \bar{\alpha}_i \alpha_e} \quad (15) $$

So as to control the damage on the frame, the maximum response interstory drift must be smaller than the maximum allowed value $\delta_{a,i}$. For design purposes, in Eq.(15) we propose to take $\delta_{max,i}=\delta_a={}_f\delta_{y,i}$.

As already assumed in deriving Eq.(10), we adopt the optimum distribution of the yield shear force coefficient on the hysteretic dampers, that is ${}_s\alpha_i = {}_s\alpha_1 \bar{\alpha}_i$, defined by Eq.(12). The required strength ${}_s Q_{y,i}$ in the hysteretic dampers is obtained as follows:

$$ {}_s Q_{y,i} = \left(\sum_{j=i}^{N} m_j g \right) \bar{\alpha}_i\ {}_s r_1\ \alpha_e \quad (16) $$

In Eq.(16), ${}_s r_1$ the maximum value between ${}_s r_{m1}$ and ${}_s r_{u1}$, that is, ${}_s r_1 = max\{ {}_s r_{m1}, {}_s r_{u1} \}$. ${}_s r_{m1}$ and ${}_s r_{u1}$ are determined by the two design criteria proposed in previous paragraph. Regarding the first design criterion, the required ${}_s r_{m1}$ for limiting the maximum interstory drift $\delta_{max,i}$ in a given story i can be calculated by solving ${}_s r_{m1}$ in Eq.(10). The ${}_s r_{m1}$ corresponding to the $\delta_{max,i}$ of each story must be calculated, and the maximum value among them is adopted for the overall structure, that is:

$$ {}_s r_{m1} = \max \left\{ 0.5\beta_i K \left[1 - \sqrt{1 + \frac{(\beta_i^2 - 1)}{KCc_e \beta_i^2}} \right] \right\} \qquad \textit{for } i=1, N \quad (17) $$

Regarding the second design criterion, the required ${}_s r_{u1}$ can be readily calculated with Eq.(13). A key parameter in applying Eqs. (10), (13) or (17) is the value adopted for the empirical coefficient c_e, which is strongly influenced by the type of restoring characteristics of the structural elements and by the relative strength between the "flexible part" and the "stiff part" of the mixed structure. For systems which restoring force characteristics exhibit stiffness degradation, such as RC frames, Akiyama [10] proposed to take $c_e=2.5$ for design purposes. The smaller c_e is the more safe-side is the seismic design of the dampers. In this research we adopted $c_e=2.5$. It is important to note however, that $c_e=2.5$ is not the lower bound of c_e, and it was not specifically derived for the GLD type of frames dealt with here.

NUMERICAL VALIDATION

Description of the RC bare frames

In order to validate the proposed method through numerical simulations, a series of dynamic response analyses were carried out. In total, six models were investigated. These models represent typical RC moment resisting bare frames located in the southern part of Spain. The models differ in the number of stories and the type of beams. Models with 3, 6 and 9 stories were considered. As for the type of beams "non-flat beams" and "flat beams" were investigated. Non-flat beams are those whose depth-to-width ratio is larger than one, and its width is smaller than or equal to that of the column. The "flat beams" are those whose depth-to-width ratio is less that one, being its depth limited to the thickness of the slab and its width larger than that of the column. The "flat beams" are commonly used in the Mediterranean area and Eastern Europe mainly for architectural and economic reasons, although they have serious structural disadvantages such as the big deformability and poor ductility. All frames have three equal spans of 6 m. All floor levels have equal height (2.75 m), except the lowest one (3.75 m). The bare frames are designed for sustaining a gravitational uniformly distributed live load of 3 KN/m^2, a wind pressure of 0.9 KN/m^2 and the lateral seismic forces prescribed by the old Spanish seismic code PDS-74 [11]. The mass of each story was $m_i=56.7\ KNs^2/m$. Compressive strength of concrete of 17.5 N/mm^2, and yield strength of reinforcement of 410 N/mm^2 are used. Since gravity forces governed over those due to wind and to seismic loads, the reinforcing details of beams were identical regardless of the story level. These frames were studied in past research [2] and it was shown that their ultimate energy absorption capacity was about one half of the required level. A detailed description of the models can be found in Reference [2].

Seismic input energy demand

It is assumed that the frames are located in the moderate-seismicity southern part of Spain and lay on soft soil. According to recent research [9], the estimated seismic input energy demand expressed in terms of the equivalent velocity is $V_E(t_o)=104\ cm/s$. Assuming that the fraction of critical damping of the structure is $\xi=0.05$ and by applying Akiyama´s equation [3], the input energy attributable to damage expressed by means of equivalent velocity terms $V_D(t_o)= V_E(t_o)/(1+3\xi+1.2\xi^{0.5})=73.3\ cm/s$.

Design of the hysteretic dampers

The energy absorbing device used in this numerical simulation is the brace-type hysteretic Damper Using steel Plates (DUP damper herein) developed by Benavent-Climent [8]. The DUP damper is an assemblage of steel plates with slits (energy absorbing device) and two longitudinal and link elements (auxiliary elements). The steel plates with slits are arranged in such a way that they are subjected primarily to shearing deformations when the longitudinal elements are axially loaded. The DUP damper is installed in a frame as a conventional brace element. For steel plates made with SM490 type mild steel (yield stress:349 N/mm^2; maximum stress:508 N/mm^2, rupture strain:24%), the value of the non-dimensional empirical parameters u and b that define the ultimate energy absorption capacity of the damper are [8]: $u=4.02$ and $b=1325$. The lateral strength and stiffness of the hysteretic dampers was determined by applying the method proposed in this paper. The process can be summarized as follows. The properties of each story of the bare frame ${}_fk_i$, ${}_f\delta_{y,i}$, T_f, χ_1, γ_1 were calculated and are shown in columns 3 to 7 of Table 1. The total mass M was 170, 340 and 510 KNs^2/m for the frames with 3, 6 and 9 stories respectively. The maximum allowed interstory drift, $\delta_{a,,i}$, was $\delta_{a,,i}=\delta_{max,i}={}_f\delta_{y,i}$. As for the stiffness ratio $K={}_sk_i/{}_fk_i$ we adopted $K=10$. The resulting lateral stiffness ${}_sk_i$ of the dampers is indicated in column 10 of Table 1. As for the non-dimensional coefficient ${}_sr_1$, the maximum value between ${}_sr_{u1}$ and ${}_sr_{m1}$ was adopted. ${}_sr_{u1}$ was calculated with Eq.(13). Regarding ${}_sr_{m1}$, the maximum value among the ${}_sr_{mi}$ obtained with Eq.(17) was taken. ${}_sr_{ui}$ and ${}_sr_{m1}$ are indicated in columns 8 and 9 of Table 1. Finally, the required lateral strength of the hysteretic dampers, ${}_sQ_{y,i}$, was determined with Eq.(16) and is also indicated in the last column of Table 1.

TABLE 1: PROPERTIES OF THE FRAMES AND DAMPERS

MODEL	Story	${}_fk_i$ (KN/m)	${}_f\delta_{y,I}$ (cm)	T_f (s)	γ_1	χ_1	${}_sr_{mi}$	${}_sr_{u1}$	${}_sk_i$ (KN/m)	${}_sQ_{yi}$ (KN)
3 STORY NON-FLAT BEAMS	3	10592	1.50	1.25	1.51	1.23	0.142	0.017	105920	134.25
	2	8861	2.03				0.204		88610	192.86
	1	5299	2.79				0.392		52990	245.58
3 STORY FLAT BEAMS	3	9729	1.57	1.35	1.49	1.23	0.136	0.017	97290	123.07
	2	7698	2.22				0.201		76980	176.80
	1	4554	3.07				0.386		45540	225.12
6 STORY NON-FLAT BEAMS	6	7878	1.89	2.14	2.59	2.19	0.104	0.018	78780	92.91
	5	7604	1.95				0.209		76040	141.28
	4	7993	1.82				0.302		79930	178.46
	3	10956	2.05				0.194		109560	203.31
	2	10606	2.00				0.256		106060	232.60
	1	6424	2.96				0.352		64240	258.45
6 STORY FLAT BEAMS	6	8259	1.71	2.29	2.53	2.19	0.107	0.018	82590	88.60
	5	6975	2.00				0.219		69750	134.73
	4	7434	1.87				0.310		74340	170.18
	3	9597	2.24				0.198		95970	193.88
	2	9196	2.21				0.262		91960	221.81
	1	5609	3.29				0.352		56090	246.46
9 STORY NON-FLAT BEAMS	9	8487	1.68	3.08	3.46	3.10	0.080	0.018	84870	73.36
	8	7325	1.95				0.174		73250	110.10
	7	7429	1.86				0.263		74290	140.33
	6	10296	1.91				0.198		102960	165.55
	5	10339	1.87				0.242		103390	185.54
	4	10528	1.79				0.283		105280	201.60
	3	11541	2.27				0.199		115410	220.82
	2	10726	2.25				0.255		107260	240.73
	1	6608	3.24				0.336		66080	256.71
9 STORY FLAT BEAMS	9	8176	1.72	3.28	3.40	3.09	0.072	0.019	81760	67.24
	8	6890	2.01				0.168		68900	100.91
	7	7090	1.88				0.256		70900	128.62
	6	9266	2.05				0.194		92660	151.73
	5	9259	2.01				0.237		92590	170.06
	4	9364	1.95				0.275		93640	184.78
	3	10118	2.51				0.193		101180	202.40
	2	9362	2.46				0.254		93620	220.64
	1	5777	3.58				0.327		57770	235.28

Numerical modeling of the structural behavior

The beams and columns that constitute the bare frame were discretized as linear members with two plastic hinges at their ends. The non-linear hysteretic behavior of the member-end hinges was described with the model proposed by Park [12]. This model uses a nonsymmetric trilinear curve that describes the momentrotation M-θ relationship under monotonic loading, in conjunction with three parameters α, β and γ that establish the rules under which the inelastic loading reversals take place. A detailed description of the monotonic curve and the values of α, β and γ adopted can be found in Reference [2]. The hysteretic characteristics of the dampers under cyclic loading were described with the non-lineal model proposed by Benavent-Climent [8] for the DUP damper, which is a realistic model based on experimental results.

Selection of ground motions and evaluation methodology.

Three acceleration records were used in this numerical verification: El Centro (1940), Taft (1952) and Kobe (1995). They cover a broad range of fundamental periods within the amplified region of the response spectra. They represent also different types of earthquakes from the standpoint of the rate of the energy input to the structure. El Centro and Kobe earthquakes input the energy in a very short time, while in the Taft record the energy is input gradually. The three records were scaled so as the seismic input energy expressed by the equivalent velocity V_E coincided with the value used for designing the dampers, that is V_E=*104 cm/s*.

Numerical results

Figures 1, 2 and 3 show the distribution of the total plastic strain energy, W_p, among the stories, and within each story between the frame and the hysteretic dampers. More precisely, $W_{p,hinges,i}$ indicate the plastic strain energy dissipated by the hinges located at beam and column ends of the *i*-th story. $W_{p,brace,i}$ is the total plastic strain energy dissipated by the brace-type hysteretic dampers installed in the *i*-th story. It is clear from the Figures than most of the total plastic strain energy dissipated by the structure is absorbed by the hysteretic dampers. However, some damage was observed in a few number of hinges.

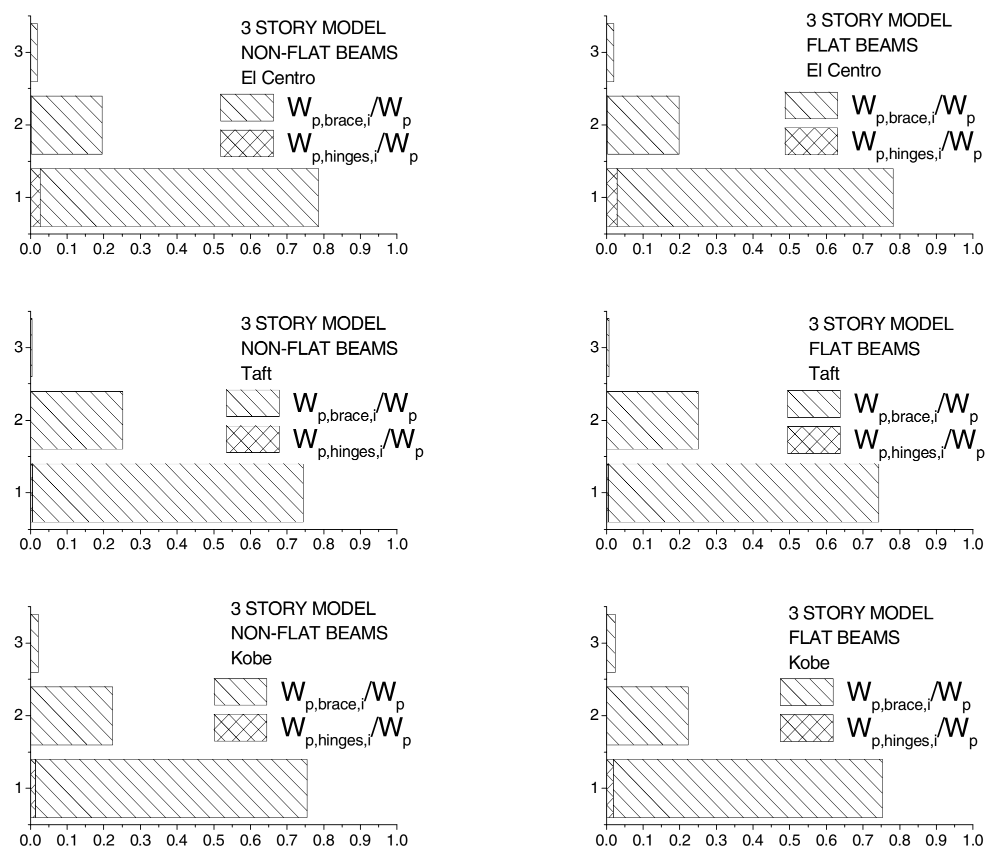

Fig.1: Distribution of the plastic strain energy in the 3-story models

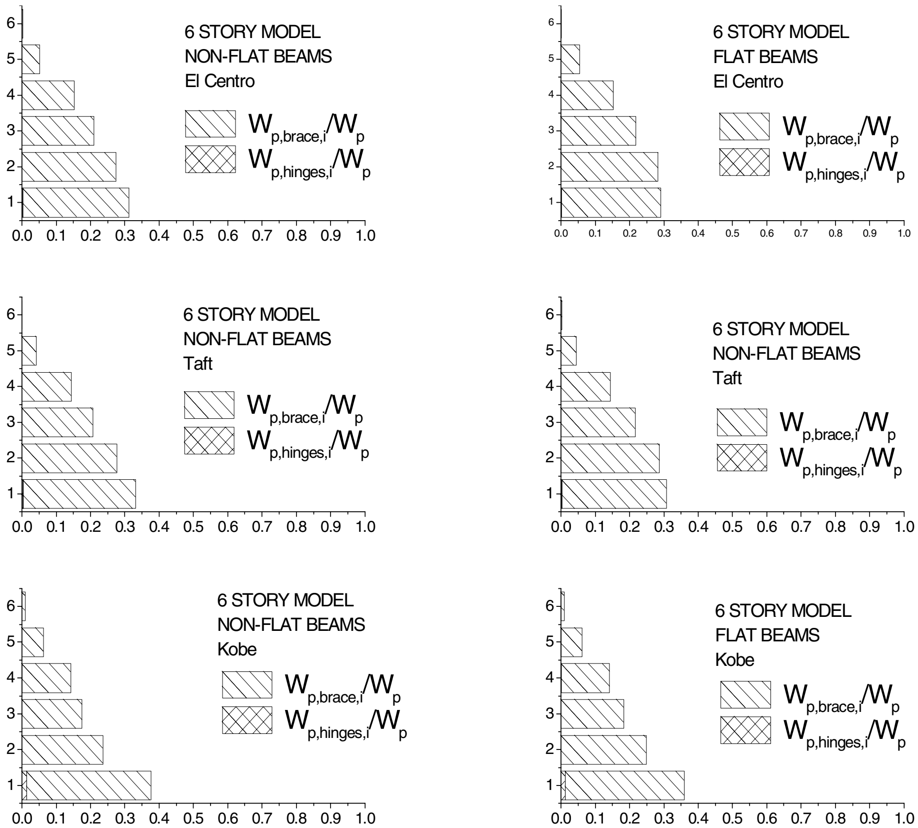

Fig. 2: Distribution of the plastic strain energy in the 6-story models

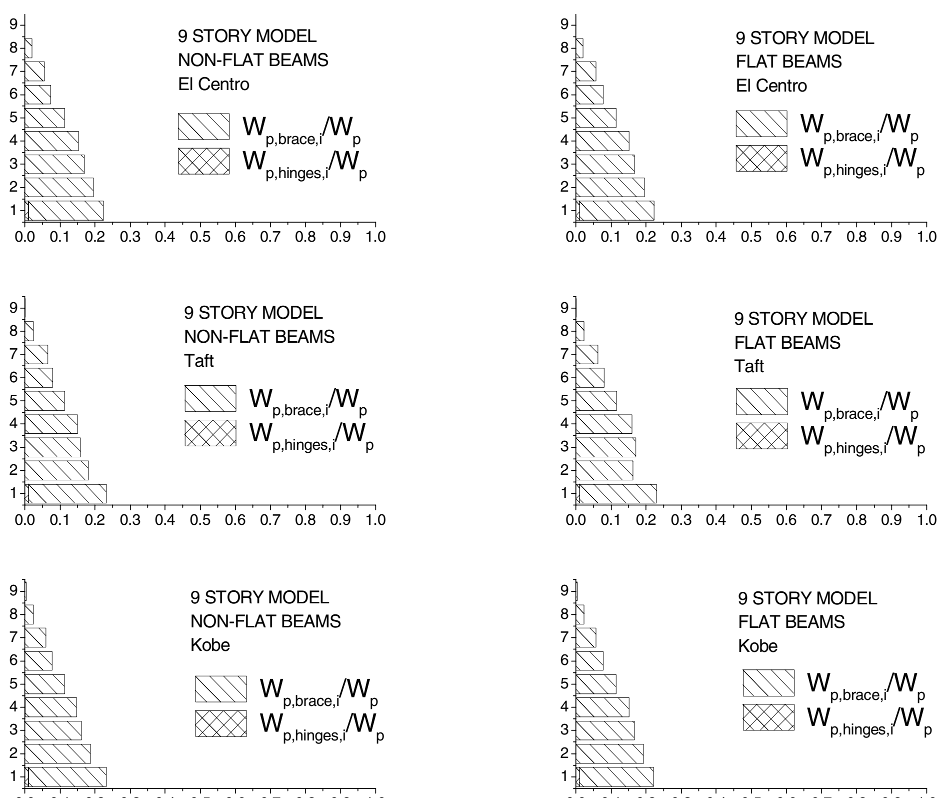

Fig. 3: Distribution of the plastic strain energy in the 9-story models

The damage suffered by these hinges was quantified by using the index proposed by Park and Ang [13]. Fig.4, 5 and 6 show the cases that presented the maximum values of this index. It was observed that the damage generally occurred in the plastic hinges located at the column ends of the first story and the beam ends of the uppermost story. The damage index of Park and Ang in these hinges was below 0.5, which corresponds to minor or moderate damage. It must be noted, however, that the damage in these hinges can be reduced or even cancelled, if a smaller value for the maximum allowed drift $\delta_{a,i}$ is adopted.

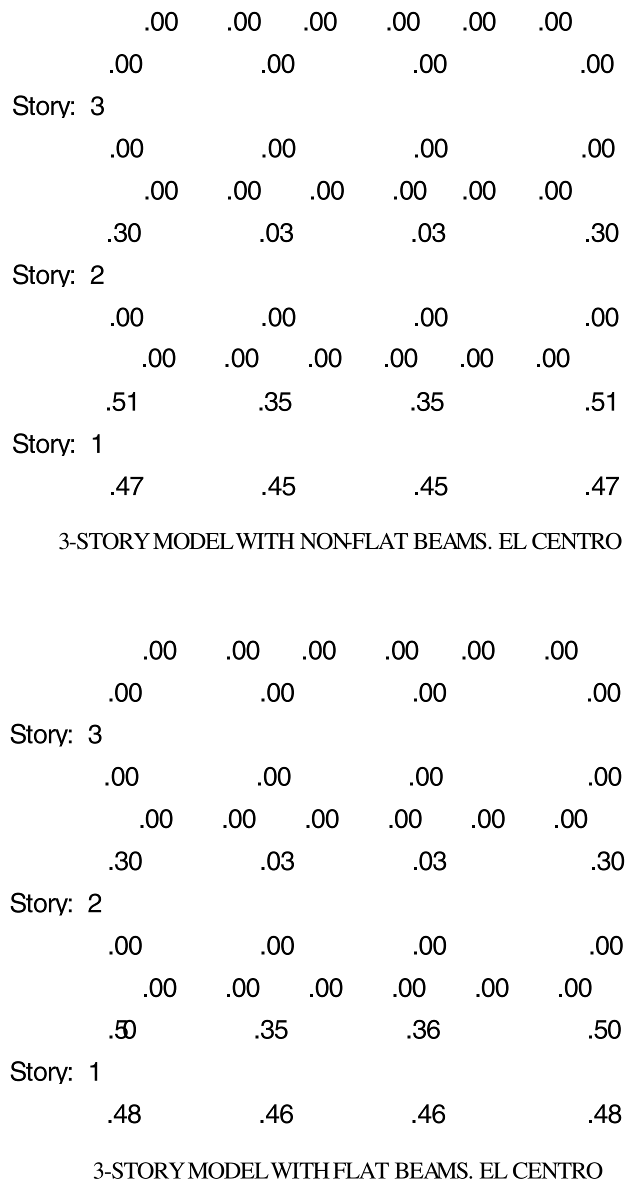

Fig. 4: Index of damage of Park and Ang in the 3-story models under El Centro earthquake

.00 .39 .35 .39 .35 .00
.00 .00 .00 .00
Story: 6
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 5
.00 .00 .00 .00
.0 0 .00 .00 .00 .00 .00
.00 .23 .23 .00
Story: 4
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 3
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 2
.00 .00 . 00 .00
.00 .00 .00 .00 .00 .00
.37 .18 .18 .37
Story: 1
.39 .40 .40 .39

STORY MODEL WITH NON-FLAT BEAMS

.00 .48 .44 .48 .44 .00
.00 .00 .00 .00
Story: 6
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 5
.00 .00 .00 . 00
.00 .42 .00 .42 .00 .00
.38 .24 .24 .38
Story: 4
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 3
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 2
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 1
.39 .41 .41 .39

6-STORY MODEL WITH FLAT BEAMS

Fig. 5: Index of damage of Park and Ang in the 6-story models under Kobe earthquake

.00 .35 .36 .35 .36 .00
.00 .00 .00 .00
Story: 9
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 8
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 7
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 6
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 5
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .23 .23 .00
Story: 4
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 3
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 2
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 1
.40 .49 .49 .40

9-STORY FRAMES WITH NON FLAT BEAMS

.00 .44 .44 .44 .44 .00
.00 .00 .00 .00
Story: 9
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 8
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 7
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 6
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 5
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .22 .22 .00
Story: 4
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 3
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 2
.00 .00 .00 .00
.00 .00 .00 .00 .00 .00
.00 .00 .00 .00
Story: 1
.39 .48 .48 .39

9-STORY FRAMES WITH FLAT BEAMS

Fig. 6: Index of damage of Park and Ang in the 9-story models under Kobe earthquake

Fig.7 shows with lines and symbols the response maximum interstory drifts obtained from the dynamic analyses. The bold lines indicate the maximum allowed interstory drifts $\delta_{a,i}={}_f\delta_{y,i}$. The dash lines show the maximum interstory drift predicted with Eq.(10) on the basis of the ${}_sr_1$ used for designing the dampers. It is observed that in general the response maximum interstory drift is smaller than the maximum allowed value $\delta_{a,i}$. The difference between $\delta_{a,i}$ and the response drift obtained in the dynamic analyses tends to increase in the upper stories. This is due to the fact that, for the investigated frames, the strength required to the hysteretic dampers was governed by the maximum displacement allowed in the first story, ${}_f\delta_{y,1}$.

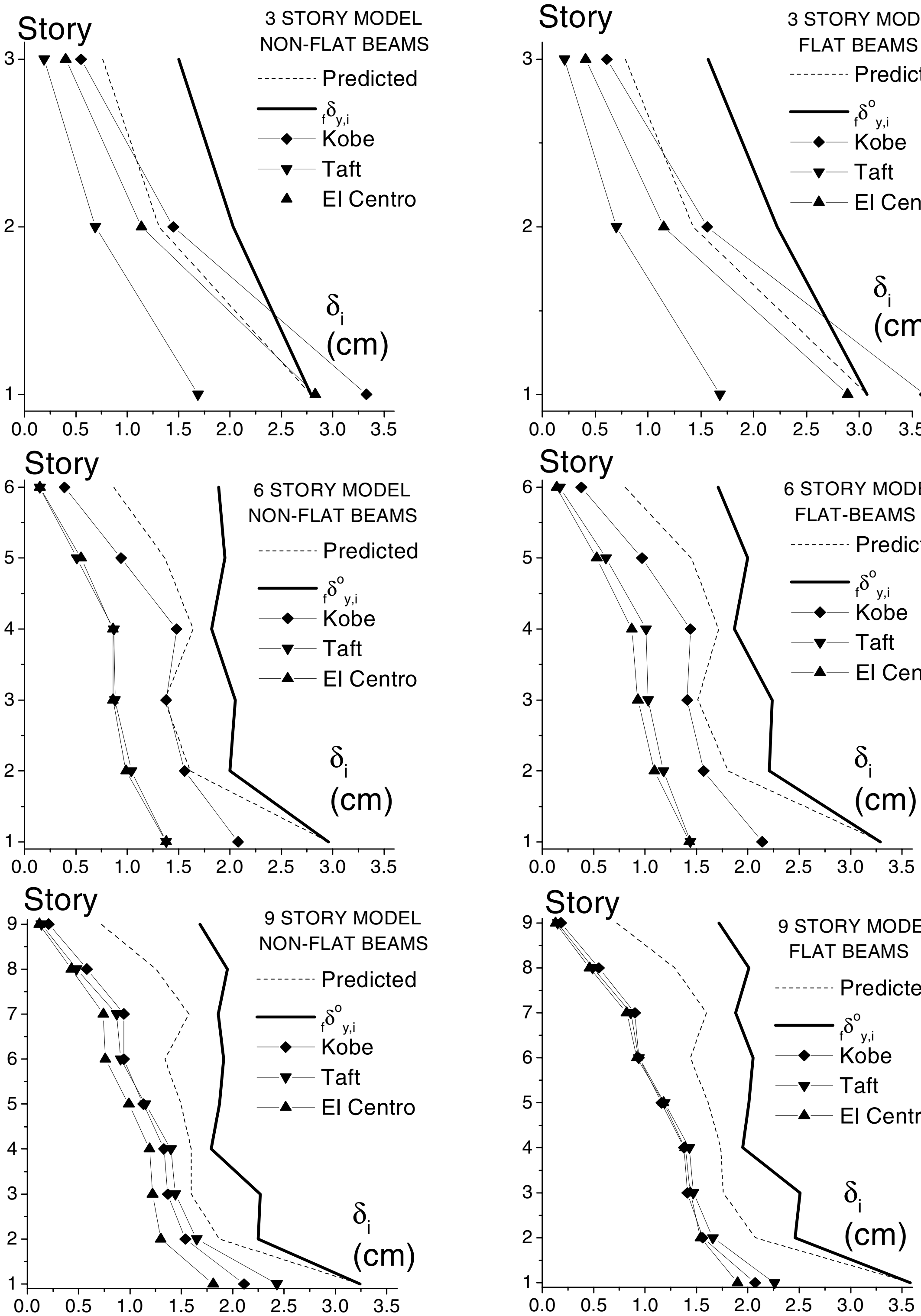

Fig. 7:Maximum interstory drifts

CONCLUSIONS

In this research, an advanced solution for seismic upgrading gravity load designed RC frames is investigated. The solution consists in installing hysteretic damper in all stories. This paper proposes the general criteria and a practical methodology for designing the dampers. The design criteria are: (a) the hysteretic dampers must guaranty that the maximum lateral displacement is smaller than the yield interstory drift of the bare frame in order to prevent severe damage on it; and (b) the ultimate energy absorption capacity of the dampers must be large enough so as to absorb the total energy attributable to damage inputted by the earthquake. For validating the proposed method a series of dynamic response analyses were carried out with six RC frame models. The models were subjected to three historical earthquakes: El Centro (1940), Taft (1952) and Kobe (1995). The results of the analyses show that the hysteretic dampers reduce drastically the damage on the beams and columns of the frame, and control effectively the maximum interstory drifs.

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