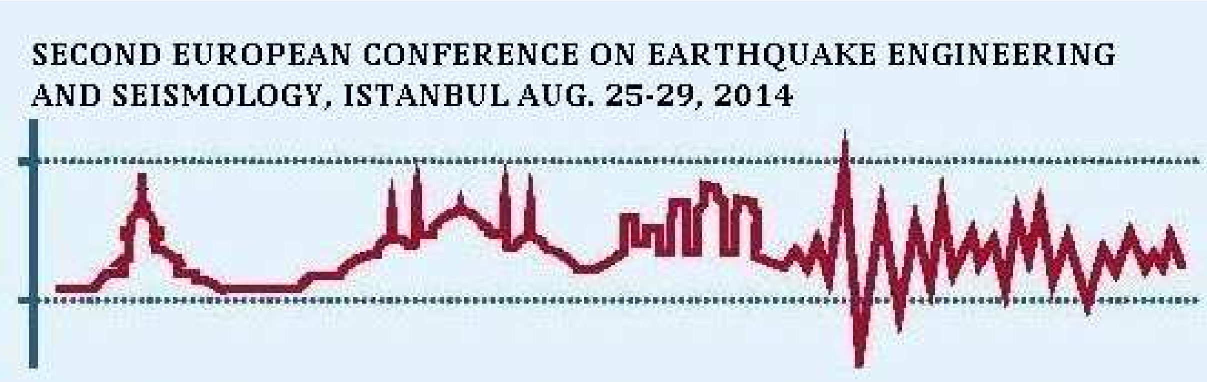

ENERGY-BASED DESIGN OF NON-TRADITIONAL STRUCTURES INCORPORATING HYSTERETIC DAMPERS: EXPERIMENTAL VALIDATION WITH SHAKING TABLE TESTS

Amadeo BENAVENT-CLIMENT[1]

ABSTRACT

Different approaches have been proposed for future seismic design codes following the uniform conceptual framework of the Performance-Based Seismic Design. Current practices using elastic design procedures (force/strength methods), are being complemented in modern codes with new approaches that include displacement-based and energy-based design. The need for an energy-based methodology for earthquake resistant design of structures was recognized as early as the mid-1950s by Housner. One of the advantages is that it can address directly the effects of cumulative damage and low-cycle fatigue associated with long duration earthquakes. The energy-based approach is particularly appropriate in non-traditional structures incorporating passive damping mechanisms. This paper presents a simple energy-based design method to design a particular type of non-traditional structure constituted of frames and hysteretic dampers. In this procedure, the design earthquake is characterized with a bilinear spectrum representing the amount of energy that contributes to damage expressed in terms equivalent velocity, and seismological parameters related to the fault distance etc. The target performance level is characterized in terms of maximum inter-story drift allowed in each story. The procedure provides the lateral strength, lateral stiffness and energy dissipation capacity required to the dampers to be installed in each story. Finally, seismic simulations conducted recently with the shaking table of the University of Granada are presented. The experimental results are compared with the maximum response predicted with the proposed energy-based procedure. It is concluded that the later provides satisfactory results.

INTRODUCTION

Performance-based seismic design (PBSD) concepts provide a suitable framework for future seismic code development. Implementing PBSD concepts require design methodologies able to cope directly and quantitatively with important aspects of the structural response such as the cumulative damage and the low-cycle fatigue effects. Also, these methodologies must be able to deal with the design of non-traditional structures such those that incorporate hysteretic dampers. The energy-based approach provides a general framework to implement the concepts of the PBSD in a practical and simple design formulation, since it gives a full understanding on the behaviour of the structure up to its collapse state, and it allows an explicit and quantitative control of the damage endured by the structure under a given level of seismic hazard. Further, the energy-based approach is particularly appropriate in non-traditional structures incorporating passive damping mechanisms (Soong and Dargush, 1997).

The earthquake resistant design approach is based on the balance of the total energy input exerted by the earthquake and the energy absorbed by the structure. One of the main benefits of this

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approach relies on the fact that the total energy input due to an earthquake is a very stable amount governed by the total mass of the structure and its fundamental period, and it is scarcely influenced by the other parameters such as mass, stiffness or strength distribution. The need for an energy-based methodology for earthquake resistant design of structures was recognized as early as the mid-1950s by Housner, and the fundamental framework has been established by Akiyama (1985, 1999) and other researchers.

This paper resents an energy-based procedure to design a particular type of non-traditional structure consisting of frames with hysteretic dampers in all stories. For the sake of simplicity, in the present form, the method assumes that the main structure (i.e. the frame) remains elastic. In the design of new structures with hysteretic dampers, this condition can be relaxed and some plastic deformations can be allowed in order to reduce the demands on the dampers. The appropriate energy-based procedure for addressing this case is not covered here due to length limitations of the paper. The energy-based method is validated with the results of seismic simulations conducted recently on a reinforced concrete (RC) frame structure equipped with hysteretic. The paper puts emphasis on the key aspects on which the formulation hinges on.

BACKGROUND

The equation of dynamic equilibrium of an inelastic multi degree-of-freedom system (MDOF) subjected to a unidirectional horizontal ground motion is given by:

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

Here, **M** is the mass matrix, **C** the damping matrix and $\mathbf{Q}(t)$ the restoring force vector; $\ddot{\mathbf{y}}(t)$ and $\dot{\mathbf{y}}(t)$ are the acceleration and velocity vectors relative to the ground; $\ddot{z}_g$ is the ground acceleration, and **r** represents the displacement vector $\mathbf{y}(t)$ resulting from a unit support displacement. Multiplying Eq. (1) by $d\mathbf{y} = \dot{\mathbf{y}}\,dt$ and integrating over the entire duration of the earthquake, i.e. from $t=0$ to $t=t_o$, the energy balance equation becomes:

$$W_k + W_\xi + W_s = E\,. \tag{2}$$

Where $W_k = \int \dot{\mathbf{y}}^{\mathbf{T}}\mathbf{M}\ddot{\mathbf{y}}dt$ is the kinetic energy; $W_\xi = \int \dot{\mathbf{y}}^{\mathbf{T}}\mathbf{C}\dot{\mathbf{y}}dt$ is the damping energy; $W_s = \int \dot{\mathbf{y}}^{\mathbf{T}}\mathbf{Q}dt$ is the absorbed energy, which is composed of the recoverable elastic strain energy, W_{se}, and the irrecoverable plastic energy, W_p, i.e. $W_s=W_{se}+W_p$; and $E = -\int \dot{\mathbf{y}}^{\mathbf{T}}\mathbf{M}\mathbf{r}\,\ddot{z}_g dt$ is, by definition, the input energy which can be expressed in the form of an equivalent velocity V_E as:

$$V_E = \sqrt{\frac{2E}{M}}\,. \tag{3}$$

Where M is the total mass of the structure. Since $W_k + W_{se}$ is the elastic vibrational energy, W_e, the equation (2) can be rewritten as:

$$W_e + W_p = E - W_\xi \tag{4}$$

Further, $W_e + W_p$ can also be expressed in the form of an equivalent velocity V_D so that:

$$W_e + W_p = \frac{MV_D^2}{2}\,. \tag{5}$$

In the energy-based seismic design approach, the V_E–T spectrum characterizes the loading effect of the earthquake for a given level of seismic hazard. Design input energy spectra V_E–T have been proposed in past studies (Zahrah 1984, Akiyama 1985; Benavent-Climent et al, 2002). The term W_p characterizes the cumulative damage (i.e. plastic strain energy) of the structure. W_p+W_e is what

Housner (1956) called the energy that damages a structure subjected to seismic action. For undamped systems V_D=V_E; otherwise (V_E-V_D) is the energy dissipated by the inherent damping of the structure. Several empirical expressions have been proposed that allow us to obtain V_D from V_E (Akiyama 1985; Kuwamura and Galambos 1989; Fajfar and Vidic 1994; Benavent-Climent et al 2002; Benavent-Climent et al 2010). Moreover, attenuation relationships have been established (Chou and Uang, 2000) that directly provide W_s —the absorbed energy— for a given earthquake magnitude, source-to-site distance, site class and ductility factor, in terms of an equivalent velocity V_a defined by

$$V_a = \sqrt{\frac{2W_S}{M}} \ . \qquad (6)$$

STRUCTURAL MODEL AND DESIGN CRITERIA

The main frame is idealized with a lumped-mass shear model. The mass of each story i will be referred to as m_i hereafter. At each story i, the main frame is assumed to remain elastic up to a lateral inter-story drift ${}_f\delta_{yi}$, and the mechanical properties are characterized by the lateral yield strength, ${}_fQ_{yi}$ and the lateral stiffness ${}_fk_i$ (=${}_fQ_{yi}/{}_f\delta_{yi}$). The fundamental period of the main frame (without dampers) is denoted as T_1. The hysteretic dampers installed in a given story i are arranged so as to form a dual system consisting of two inelastic springs connected in parallel. The lateral load-displacement relationship, ${}_fQ_i$-δ_i, of a given i-th story under monotonic loading is represented in Figure 1. The hysteretic characteristics of the dampers are assumed to be elastic-perfectly-plastic, and in each story i they provide a lateral strength ${}_sQ_{yi}$ and a lateral stiffness ${}_sk_i$ as shown in Figure 1.

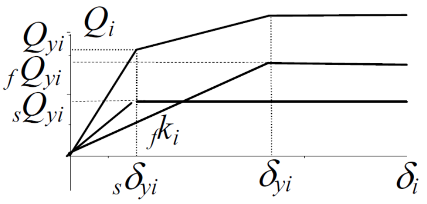

Fig. 1: Idealized inter-story drift-shear force curve of each story i

The goals of the energy-based procedure are: (i) to determine the ${}_sQ_{yi}$ and ${}_sk_i$ of the dampers needed in each story to achieve the required building performance levels, expressed in terms of maximum allowed displacement $\delta_{\max i}$, for a given earthquake hazard; and (ii) to evaluate the energy dissipation demand on the hysteretic dampers. To keep the main frame within the elastic range, it is imposed that:

$$\delta_{maxi} \leq {}_f\delta_{yi} \ . \qquad (7)$$

Accordingly, the lateral yield strength of the entire frame-device structure at the i-th story, Q_{yi}, is:

$$Q_{yi} = {}_sQ_{yi} + {}_fQ_{\max,i} = {}_sQ_{yi} + {}_s\delta_{yi}\,{}_fk_i \qquad (8)$$

where ${}_s\delta_{yi}$ (=${}_sQ_{yi}/{}_sk_i$) is the yield deformation of the dampers and ${}_fQ_{max,i}$=$\delta_{maxi}\cdot{}_fk_i$ is the maximum lateral force sustained by the frame, both at the i-th story. For the building-device structure surviving the earthquake, the plastic strain energy accumulated in the i-th story, W_{pi}, must not exceed the ultimate energy dissipation capacity of the dampers installed in that story, W_{ui}. In turn, W_{pi} and W_{ui} can be expressed in the form of two non-dimensional coefficients ,η_i and η_{ui}, defined by:

$$\eta_i = \frac{W_{pi}}{{}_sQ_{yi}\,{}_s\delta_{yi}} \quad ; \quad \eta_{ui} = \frac{W_{ui}}{{}_sQ_{yi}\,{}_s\delta_{yi}} \qquad (9)$$

thus, the above condition can be written as:

$$\eta_i \leq \eta_{ui} \ . \qquad (10)$$

FORMULATION OF THE METHOD

For the sake of convenience, δ_{maxi}, Q_{yi}, ${}_fQ_{maxi}$, ${}_sQ_{yi}$ and ${}_fk_1$ will be expressed herein by the plastic deformation ratio μ_i, the shear-force coefficients α_i, ${}_f\alpha_{maxi}$, ${}_s\alpha_i$, and the stiffness ratio χ_1,defined by:

$$\mu_i = \frac{(\delta_{maxi} - {}_s\delta_{yi})}{{}_s\delta_{yi}} \quad ; \quad \alpha_i = \frac{Q_{yi}}{\sum_{k=i}^{N} m_k g} \quad ; \quad {}_f\alpha_{maxi} = \frac{{}_fQ_{maxi}}{\sum_{k=i}^{N} m_k g} \quad ; \quad {}_s\alpha_i = \frac{{}_sQ_{yi}}{\sum_{k=i}^{N} m_k g} \quad ; \quad \chi_1 = \frac{{}_fk_1}{k_{eq}} \tag{11}$$

Here, N is the total number of stories, g is the acceleration of the gravity, $k_{\text{eq}}=4\pi^2 M/T_1{}^2$ and the base story is taken as i=1.

<u>*Stiffness distribution of the dampers among the stories*</u>

The ratio between the lateral stiffness of dampers and main frame in each story is referred to as:

$$K_i = \frac{{}_sk_i}{{}_fk_i} \tag{12}$$

In choosing the values K_i the following considerations must be taken into account. Oviedo et al (2010) defined a strength ratio $\beta_{min,1}$ for the base story as:

$$\beta_{min,1}={}_sQ_{y1}/({}_sQ_{y1}+{}_fQ_{y1}) \tag{13}$$

and recommended $0.2\leq\beta_{min,1}\leq0.5$ because in this range the protection to the main frame due to the dampers is maximized. The proposed method enforces that the yield story-drift ratio:

$$\nu_i={}_s\delta_{yi}/{}_f\delta_{yi} \tag{14}$$

must be less than 1 to guarantee minimum protection to the main frame. Oviedo et al (2010) recommended using low values of ν_i (less than about 0.4) because: (i) it makes more effective the protection to the main frame; and (ii) tends to widen the "uniform" range of the $\beta_{min,1}$ at which the protection of the main frame with the dampers is maximized and kept almost invariant irrespective of $\beta_{min,1}$. As pointed out by Oviedo et al (2010), widening this range has a relevant impact on engineering practice because the structural performance would be less affected in the case of modifications of $\beta_{min,1}$ due to uncertainties such as construction and on-site installation practices and/or material strength reliability.

Inoue and Kuwahara (1998) defined a similar strength ratio:

$$\beta_i={}_sQ_{yi}/({}_sQ_{yi}+{}_fQ_{max,i}) \tag{15}$$

and proposed the following optimum value:

$$\beta_{opt,i}=1-(K_i+1)^{-0.5} \tag{16}$$

It is worth noting that if the base story of main frame is on the brim of yielding, i.e. ${}_fQ_{max1}={}_fQ_{y1}$, then

$$\beta_{min,1}=\beta_i. \tag{17}$$

Akiyama (1999) characterized the strength ratio between the dampers and the main frame by:

$$r_{q,i}={}_sQ_{yi}/{}_fQ_{max,i} \tag{18}$$

that is obviously related to β_i by:

$$\beta_i=(1+r_{q,i})^{-1} \tag{19}$$

The optimum value $r_{q,opt,i}$ is obtained making $\beta_i=\beta_{opt,i}$ in Eq. (18), using Eq.(14) and solving for $r_{q,i}$:

$$r_{q,opt,i} = \frac{\sqrt{K_i+1}}{\sqrt{K_i+1}-1} - 1 \tag{20}$$

If the maximum inter-story drift allowed by Eq. (7) is adopted, i.e. $\delta_{max,i}={}_f\delta_{y,i}$, K_i can be expressed in terms of ν_i and $r_{q,i}$ as follows:

$$K_i = \frac{1}{r_{q,i}\,\nu_i} \tag{21}$$

Making $r_{q,i}=r_{q,opt,i}$ in Eq.(20), using Eq. (19) and solving for K_i, the following relation is obtained between K_i and ν_i when the optimum strength ratio $r_{q,opt,i}$ is used and the frame is allowed to displace laterally up to the onset of yielding $\delta_{max,i}={}_f\delta_{y,i}$:

$$K_i = \frac{1-2\nu_i}{\nu_i^2} \quad (22)$$

Eqs. (19) and (21) are plotted in Figure 2, and it can be seen that for a reasonable range of $0.15\leq\nu_i\leq0.4$ the stiffness ratio varies from K_i=1.25 to K_i=15.

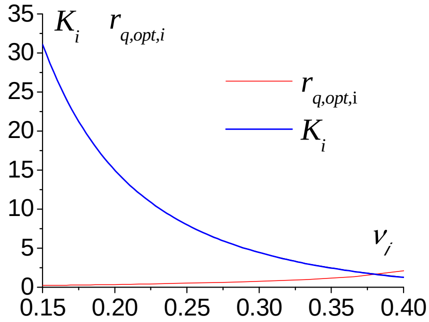

Fig. 2: Relationship between ν_i , K_i and $r_{q,opt,i}$ for $\delta_{max,i}={}_f\delta_{y,i}$

There is no need to make K_i equal in all stories, although this criterion has been often used in the past (Inoue and Kuwahara, 1998).

Strength distribution of the dampers among the stories

The lateral strength distribution of the entire building-device structure, Q_{yi}/Q_{y1}, can be expressed in terms of shear-force coefficients by $\bar{\alpha}_i=\alpha_i/\alpha_1$. The criterion adopted in the proposed method to determine the $\bar{\alpha}_i$ distribution is to attain an approximately even distribution of damage among the hysteretic dampers. The damage in the dampers installed in a given story *i* can be characterized by the non-dimensional parameter η_i defined by Eq.(9). Past studies (Akiyama 1985) showed that the strength distribution $\bar{\alpha}_i$ that makes η_i approximately equal in all stories ($\eta_i=\eta$) in a low- to- medium rise multi-story building subjected to seismic loads coincides with the maximum shear-force distribution in an equivalent elastic undamped shear strut with similar stiffness distribution, and can be approximated by (Benavent-Climent, 2011):

$$\bar{\alpha}_i = \frac{\alpha_i}{\alpha_1} = \exp\left[\left(1-0.02\frac{{}_fk_1}{{}_fk_N}-0.16\frac{T_1}{T_G}\right)\bar{x}-\left(0.5-0.05\frac{{}_fk_1}{{}_fk_N}-0.3\frac{T_1}{T_G}\right)\bar{x}^2\right] \quad (23)$$

Here $\bar{x}=(i-1)/N$, ${}_fk_N$ is the lateral stiffness of the uppermost N-th story of the main structure, and T_G (predominant period of the ground motion) defines the change of slope of the V_D-T bilinear spectra. From the definition of ${}_s\alpha_i$ and K_i given by Eqs.(11) and (12), the following relation must be satisfied:

$${}_s\alpha_i = \bar{\alpha}_i \, {}_s\alpha_1 \frac{K_i(K_1+1)}{K_1(K_i+1)} \quad (24)$$

Lateral strength to be provided by the EDDs of the first story

Once the K_i's are fixed and assuming the lateral force distribution given by Eq. (23), the lateral shear-force coefficient to be provided by the dampers of the first story, ${}_s\alpha_1$, must be calculated in order to obtain the required lateral shear force coefficient of the dampers in the other stories ${}_s\alpha_i$ with Eq.(24). The equation that governs the ${}_s\alpha_1$ required for a given seismic hazard and building performance level are derived next by establishing the energy balance of the structure.

Neglecting the elastic strain energy stored by the dampers, the elastic vibrational energy of the whole building, W_e, can be approximated from the maximum shear force sustained by the main structure on the first story as follows (Akiyama, 1985):

$$W_e = \frac{Mg^2T_1^2}{4\pi^2}\frac{{}_f\alpha_{max1}^2}{2}\ . \tag{25}$$

From Eq.(9) and taking into account the coefficients defined in Eq.(11), the plastic strain energy accumulated in the *i*-th story W_{pi} can be expressed as follows:

$$W_{pi} = \eta_i\, {}_sQ_{yi}\, {}_s\delta_{yi} = \eta_i \frac{{}_sQ_{yi}^2}{{}_sk_i} = \eta_i\, {}_s\alpha_i^2 \Big(\sum_{k=i}^{N} m_k g\Big)^2 \frac{1}{{}_sk_i}\ . \tag{26}$$

Provided that the strength distribution given by Eq.(23) is adopted, η_i can be assumed equal in all stories, i.e. η_i=η. Thus, taking into account Eq.(12) and using the non-dimensional parameters ${}_s\alpha_i$ and $\bar{\alpha}_i$ defined above, the total plastic strain energy dissipated by the dampers of the whole structure, W_p, can be expressed in terms of the plastic strain energy dissipated by the dampers of the first story, W_{p1}, by introducing a new ratio γ_1=W_p/W_{p1}, which is obtained as follows:

$$\gamma_1 = \frac{W_p}{W_{p1}} = \frac{\sum_{i=1}^{N}\left[\eta_i\, {}_s\alpha_i^2\Big(\sum_{k=i}^{N} m_k g\Big)^2 / {}_sk_i\right]}{\eta_1\, {}_s\alpha_1^2 M^2 g^2 / {}_sk_1} = \sum_{i=1}^{N}\left\{\left[\bar{\alpha}_i\left(\sum \frac{m_j}{M}\right)\frac{(K_1+1)}{(K_i+1)}\right]^2 \frac{{}_fk_1K_i}{{}_fk_iK_1}\right\} \tag{27}$$

thus

$$W_p=\gamma_1 W_{p1}=\gamma_1\, {}_sQ_{y1}\, {}_s\delta_{y1}\,\eta= \frac{\gamma_1\, {}_sQ_{y1}^2\eta}{{}_sk_1} = \frac{\gamma_1\, {}_s\alpha_1^2 M^2 g^2 \eta}{K_1\, {}_fk_1} = \frac{\gamma_1\, {}_s\alpha_1^2 M^2 g^2 \eta}{K_1\chi_1 k_{eq}} = \frac{\gamma_1\, {}_s\alpha_1^2 M g^2 \eta T_1^2}{4\pi^2 K_1\chi_1} \tag{28}$$

Substituting Eqs.(25) and (28) in Eq.(5) gives:

$$\frac{Mg^2T_1^2}{4\pi^2}\left[\frac{{}_f\alpha_{\max1}^2}{2} + \frac{\gamma_1}{K_1\chi_1}\eta\, {}_s\alpha_1^2\right] = \frac{MV_D^2}{2}\ . \tag{29}$$

A new parameter α_e is now introduced that represents the base shear-force that the main structure should have in order to absorb by itself —i.e. without dampers— the amount of input energy $MV_D^2/2$ supplied by the earthquake.

$$\alpha_e = \frac{2\pi V_D}{gT_1}\ . \tag{30}$$

Using Eq.(30), Eq.(29) can be rewritten as follows:

$$\frac{{}_f\alpha_{\max1}^2}{2} + \frac{\gamma_1}{K_1\chi_1}\eta\, {}_s\alpha_1^2 = \frac{\alpha_e^2}{2}\ . \tag{31}$$

The relation between η_i=η and μ_i is a key parameter in the energy-based seismic design and it has been addressed in different ways in the past (Akiyama 1985; Akiyama, 1999; Uang and Bertero, 1990; Cosenza and Manfredi 1997; Manfredi 2001; Manfredi et al, 2003). Based on the results of regression analyses performed with 128 near-fault and 122 far-field earthquake records, Manfredi et al. (2003) proposed the following formulae for estimating the equivalent number of plastic yield excursions n_{eq} at the maximum deformation that a single-of-freedom (SDOF) system of mass *m,* elastic period *T* and yielding force F_y must develop in order to dissipate the total amount of hysteretic energy input by the earthquake:

$$n_{eq} = 1 + c_1 I_d \sqrt{\frac{T_{NH}}{T}} (R-1)^{c_2} \ . \tag{32}$$

Here T_{NH} is the initial period of medium period region in the Newmark and Hall (1982) spectral representation. R is the reduction factor defined as $R=mS_a/F_y$ where S_a is the elastic spectral acceleration. I_d is a seismological parameter (Cosenza and Manfredi, 1997) defined by

$$I_d = \frac{\int_0^{t_o} \ddot{z}_g^2 dt}{PGA \cdot PGV} \tag{33}$$

where PGA and PGV are the peak ground acceleration and velocity, respectively. In Eq.(22), Manfredi et al. (2003) proposed to take c_1=0.23, c_2=0.4 for near-fault earthquakes; and c_1=0.18, c_2=0.6 for far-field earthquakes. For the dampers with elastic-perfectly-plastic characteristics dealt with in this study, n_{eq} is by definition (Manfredi et al, 2003): $n_{eq}=W_{pi}/[{}_sQ_{yi}(\delta_{maxi}-{}_s\delta_{yi})]$, which coincides with η_i/μ_i for a given story i. Further, to apply equation (32) to the proposed method, the multi-story structure is assimilated to an equivalent SDOF system with elastic period T_1, mass M and $F_y={}_sQ_{y1}+{}_fk_1\cdot{}_s\delta_{y1}$. Taking into account that S_a is approximately equal to $(2\pi/T)S_v$, and that the elastic spectral velocity S_v coincides approximately with V_D (Housner, 1956; Akiyama 1985), Eq.(32) can be rewritten as:

$$\frac{\eta_i}{\mu_i} = n_{eq} = 1 + c_1 I_d \sqrt{\frac{T_{NH}}{T_1}} \left(\frac{K_1 \alpha_e}{(K_1+1)_s \alpha_1} - 1 \right)^{c_2} \tag{34}$$

Akiyama (1999) proposed simpler design expressions for η_i/μ_i that depend on the strength ratio $r_{q,i}$ and the hysteretic rule. For elastic-perfectly plastic systems:

for $r_{q,i} \leq 1.0$: $\eta_i / \mu_i = n_{eq} = 4 + 4r_{qi}$ (35)

for $r_{q,i} > 1.0$: $\eta_i / \mu_i = n_{eq} = 8$ (36)

For systems which displacement-restoring force curve exhibit stiffness degradation (Clough model):

for $r_{q,i} \leq 1.0$: $\eta_i / \mu_i = n_{eq} = 3.75 + 1.25r_{qi}$ (37)

for $r_{q,i} > 1.0$: $\eta_i / \mu_i = n_{eq} = 5$ (38)

In the proposed method, the same $n_{eq}=\eta_i/\mu_i=\eta/\mu$ is adopted for all stories. Since η_i was also assumed as constant, i.e. $\eta_i=\eta$ —because the optimum distribution $\bar{\alpha}_i$ was adopted—, the maximum plastic deformation ratio μ_i has the same value $\mu_i=\mu$ $(=\eta/n_{eq})$ in all stories. On the other hand, taking into account Eq.(7), the maximum base shear-force coefficient of the main structure ${}_f\alpha_{max1}$ is:

$${}_f\alpha_{\max 1} = \frac{\delta_{\max 1}\ {}_fk_1}{Mg} \ . \tag{39}$$

From the definition of μ_i $(=\mu)$ —Eq.(11)— particularized for the first story, it is obtained that $\delta_{max1}={}_s\delta_{y1}(\mu+1)$, and substituting in Eq.(39) gives:

$${}_f\alpha_{\max 1} = \frac{{}_s\delta_{y1}\ {}_fk_1(\mu+1)}{Mg} = \frac{{}_s\delta_{y1}\ {}_sk_1(\mu+1)}{K_1 Mg} = \frac{{}_sQ_{y1}(\mu+1)}{K_1 Mg} = \frac{{}_s\alpha_1(\mu+1)}{K_1} \ . \tag{40}$$

Substituting Eq. (40) in Eq.(31), recalling that $\mu=\mu_i=\eta/n_{eq}$ and solving for μ gives:

$$\mu = K_1 \left\{ \sqrt{\left(\frac{n_{eq}\gamma_1}{\chi_1}\right)^2 + \frac{2n_{eq}\gamma_1}{K_1\chi_1} + \frac{\alpha_e^2}{{}_s\alpha_1^2}} - \frac{n_{eq}\gamma_1}{\chi_1} \right\} - 1 \,. \tag{41}$$

For the other stories $\mu=(\delta_{maxi}-{}_s\delta_{yi})/{}_s\delta_{yi}$, then, using Eqs.(24) and (41), and solving for δ_{maxi} gives the equation that predicts the maximum displacement of a given story i:

$$\delta_{\max i}=\frac{\bar{\alpha}_i\, {}_s\alpha_1 (K_1+1)(\sum_{j=i}^{N} m_j\, g)}{{}_f k_i\,(K_i+1)}\left\{\sqrt{\left(\frac{n_{eq}\gamma_1}{\chi_1}\right)^2+\frac{2n_{eq}\gamma_1}{K_1\chi_1}+\frac{\alpha_e^2}{{}_s\alpha_1^2}}-\frac{n_{eq}\gamma_1}{\chi_1}\right\} \tag{42}$$

PROCEDURE

First, a preliminary design of the main frame (without dampers) is made, and the basic properties m_i, ${}_fk_i$, ${}_f\delta_{yi}$ and T_1 are determined by using approximate formulae or by creating a finite element based model and performing a pushover analysis. The main frame must be designed to remain elastic under the action of the gravity loads and the imposed lateral displacements relative to the ground $d_i=\sum_{s=1}^{i}\delta_{allow,s}$ applied at each floor i. Here $\delta_{allow,s}$ is the maximum inter-story drift at story s determined by the designer according to the predetermined seismic performance level sought for a given earthquake hazard. The goal of the proposed method is to determine the lateral stiffness ${}_sk_i$, the lateral strength ${}_sQ_{yi}$, and the normalized energy dissipation demand η of the dampers to be installed in each story i, so that $\delta_{max,i}\leq\delta_{allow,i}$ for a given earthquake hazard. The basic steps involved in the procedure are summarized as follows.

Step 1: Characterize the earthquake hazard level. If n_{eq} is being calculated with Eq.(34) the earthquake hazard must be characterized in terms of V_D, T_G, T_{NH}, I_d and the proximity to the source. If Eqs.(35)-(38) are being used for estimating n_{eq}, only V_D and T_G are required.

Step 2: Prescribe the maximum inter-story drift allowed in each story i, $\delta_{allow,i}$, in accordance with the acceptance criteria for building components at the target performance level. Adopt a limiting value for ν_i. As explained in previous section, ν_i must be smaller than 1 and adopting smaller values for ν_i improves the efficiency of the system.

Step 3: Calculate $\bar{\alpha}_i$ for each story i with Eq.(23), α_e with Eq.(30) and χ_1 with Eq.(11).

Step 4: Choose a set of values for K_i, and compute γ_1 with Eq.(27). From i=1 to i=N proceed for each story as follows. Starting with ${}_s\alpha_1$=0, iterate in Eq.(42) —with n_{eq} given by Eq.(34) or by Eqs.(35) to (38) — increasing the values of ${}_s\alpha_1$ until the predicted δ_{maxi} gets close to δ_{allowi} within an acceptable tolerance. In these iterations, ${}_s\alpha_1$ shall not be larger than the value given by the following expression, so that ${}_s\delta_{yi}\leq(\nu_i\, {}_f\delta_{yi})$:

$${}_s\alpha_1\leq\frac{\nu_i\, {}_f\delta_{yi}\ {}_fK_i\, K_1 (K_i+1)}{\bar{\alpha}_i (K_1+1)\sum_{k=1}^{N}(m_k\, g)} \tag{43}$$

Above expression is obtained using Eqs.(11), (24) and making ${}_s\delta_{yi}\leq(\nu_i\, {}_f\delta_{yi})$. If in a given story i it is not possible to find a ${}_s\alpha_1$ that makes δ_{maxi} close enough to δ_{allowi}, restart step 4 with different values for K_i. If a satisfactory solution is not found with reasonable values of K_i, the preliminary design of the main frame should be modified, the new values of ${}_fk_i$, ${}_f\delta_{yi}$ and T_1 should be calculated and the procedure should be restarted in Step 3. Once the appropriate ${}_s\alpha_1$ is obtained, keep this value as ${}_s\alpha_{1i}$=${}_s\alpha_1$ and proceed with the next story. The parameter ${}_s\alpha_{1i}$ represents the shear-force coefficient required for the dampers of the first story so that the maximum inter-story drift at the i-th story does not exceed δ_{allowi}.

Step 5: Select the maximum of the ${}_s\alpha_{1i}$, i.e. ${}_s\alpha_{1max}$=max{${}_s\alpha_{1i}$}, which gives the required lateral strength for the dampers of the first story. Obtain the lateral strength required in the other stories, ${}_s\alpha_i$, by making ${}_s\alpha_1$=${}_s\alpha_{1max}$ in Eq.(24). Calculate the lateral stiffness ${}_sk_i$ and the lateral strength ${}_sQ_{yi}$ required for the dampers of each story taking into account Eqs. (11) and (12).

Step 6: Once the lateral stiffness ${}_sk_i$ and the lateral strength ${}_sQ_{yi}$ of the EDDs to be installed in each story are determined, an appropriate type of hysteretic damper must be chosen. To this end, it is necessary to check that the normalized ultimate energy dissipation capacity of the damper η_{ui} is larger than the demand η_i (=η) as indicated by Eq.(10). η is simply calculated by making ${}_s\alpha_1$=${}_s\alpha_{1max}$ in

Eq.(34) or using Eqs. (35)-(38) to obtain n_{eq}, substituting this n_{eq} and ${}_s\alpha_1={}_s\alpha_{1max}$ in Eq.(41) to calculate μ (=μ_i), and recalling that η=$n_{eq}\mu$. The estimation of η_{ui} for a given type of hysteretic damper is beyond the scope of this paper; yet a procedure is proposed by Benavent-Climent (2007).

EXPERIMENTAL VALIDATION

To validate experimentally the proposed method, dynamic tests were conducted on a 2/5 scale model with the 3×3m^2 shaking table of the University of Granada. Figure 3 gives an overall view of the tests. The test structure consisted of a reinforced concrete frame with one-and-half stories and one-and-half spans. Two brace-type hysteretic dampers were installed in each story as shown in Fig. 3. The structure was subjected to a sequence of seismic simulations in which a ground motion record recorded at Calitri (Italy) during the 1980 Campano-Lucano earthquake was scaled in time by (2/5)$^{0.5}$ and in acceleration to levels of increasing intensity. One of the seismic simulations, referred to as C200 hereafter, represented the design earthquake prescribed by the Spanish seismic code for Granada (Spain). The seismic simulation C200 was carried out scaling the earthquake record to a peak acceleration of 0.31g. During this simulation, the main structure remained elastic and all plastic strain energy consumed by the system was dissipated by the dampers. The maximum lateral inter-story drift measured in the first (ground) story during this simulation was 0.89 cm, which represents 0.64% of the first story height. A detailled description of the test can be found elsewhere (Benavent-Climent et al, 2014). Hereafter, the maximum inter-story drift meassured in the first story during seismic simulation C200 (0.89 cm) is compared with the prediction provided by the procedure explained in is paper.

Fig. 3: Overall view of the shaking table test

To predict the maximum displacement with the proposed procedure, first the parameters V_D, T_G, T_{NH}, I_D that characterize the seismic shaking applied to the table during simulation C200 were determined from the measurements provided by the instrumentation during this test (i.e. the actual acceleration measured in the shake table), giving V_D=61cm/s; T_G=0.75s, T_{NH}=0.9s and I_D=23.5. The test model was idealized with a two-mass lumped model. The mass lumped at the first and second floor levels were, respectively, m_1=6480 kg and m_2=5970 kg. The lateral stiffness ${}_fk_i$ and strength ${}_fQ_{yi}$ of the frame (without dampers) estimated with a numerical model gave ${}_fk_1$=2 kN/mm, ${}_fQ_{y1}$=17.6 kN, and ${}_fk_1$=1.2 kN/mm, ${}_fQ_{y2}$=15.3 kN, for the first and second stories. The fundamental period of the frame (without dampers) was T_1=0.564s. The stiffness ratio of the first story was K_1=10 and the base shear force coefficient provided by the dampers ${}_s\alpha_1$=0.45. Using Eq.(34) for estimating n_{eq} with c_1=0.18 and c_2=0.6 (far-field earthquake), the proposed formulation predicts a maximum lateral displacement of $\delta_{max,1}$=0.83 cm, which is very close to the experimental result (0.89 cm).

CONCLUSIONS

An energy-based design procedure is presented to design multi-story frames with hysteretic dampers. The procedure provides the lateral strength, the lateral stiffness and energy dissipation capacity required for the dampers to be installed in each story to achieve a desired building performance level for a given earthquake hazard. The maximum allowed inter-story drift controls the target performance level. The earthquake hazard is characterized in terms of energy input and several seismological parameters used in the literature. With this method, the effect of the hysteretic dampers is recognized directly in terms of hysteretic energy, without having to resort to equivalent viscous damping approximations; further, the cumulative damage induced in the dampers is explicitly evaluated. The validity is assessed experimentally by means of shake table tests.

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REFERENCES

Akiyama H (1985). Earthquake-Resistant Limit-State Design for Buildings, University of Tokyo Press, Tokyo.

Akiyama H (1999). Earthquake-Resistant Design Method for Buildings Based on Energy Balance. Gihodo Shuppan Co. Ltd., Tokyo.

Benavent-Climent A (2007). “An energy-based damage model for seismic response of steel structures”, Earthquake Engineering and Structural Dynamics 36:1049-1064.

Benavent-Climent A (2011). “An energy-based method for seismic retrofit of existing frames using hysteretic dampers”, Soil Dynamics and Earthquake Engineering 31:1385–1396.

Benavent-Climent A, Morillas L and Escolano-Margarit D (2014). “Shake-table tests of a reinforced concrete frame with hysteretic dampers: seismic performance and damage evaluation”. Submitted to Earthquake Engineering and Structural Dynamics. Under second revision.

Benavent-Climent A, Pujades LG, Lopez-Almansa F (2002). “Design energy input spectra for moderate seismicity regions”, *Earthquake Engineering and Structural Dynamics* 31: 1151-1172.

Chou CC, Uang CM (2000). “Establishing absorbed energy spectra — an attenuation approach”, *Earthquake Engineering and Structural Dynamics* 29:1441-1455.

Cosenza E, Manfredi G. (1997). “The improvement of the seismic-resistant design for existing and new structures using damage criteria”, in: P. Fajfar, H. Krawinkler (Eds), Seismic Design Methodologies for the Next Generation of Codes, Balkema, Rotterdam, pp 119-130.

Fajfar P, Vidic T (1994). “Consistent inelastic design spectra: hysteretic and input energy”, *Earthquake Engineering and Structural Dynamics* 23: 523-537.

Housner GW (1956). “Limit design of structures to resist earthquakes”, *Proceedings of the First World Conference on Earthquake Engineering*, Berkeley CA.

Inoue K, Kuwahara S (1998) “Optimum strength ratio of hysteretic damper”, *Earthquake Engng. Struct. Dyn.* 27: 577-588.

Kuwamura H, Galambos TV (1989). “Earthquake load for structural reliability”, *Journal of Structural Engineering* 115:1446-1462.

Manfredi G, Polese M, Cosenza E (2003). “Cumulative demand of the earthquake ground motions in the near source” Earthquake Engineering and Structural Dynamics 32:1853-1865.

Manfredi G. (2001) “Evaluation of seismic energy demand, Earthquake Engineering and Structural Dynamics 30:485-499.

Newmark NM, Hall WJ (1982). Earthquake spectra and design, Earthquake Engineering Research Institute, Berkeley, California.

Oviedo JA, Midorikawa M, Asari T (2010). “Earthquake response of ten-story story-drift-controlled reinforced concrete frames with hysteretic dampers”. *Engng. Struct.*; **32**:1735-1746.

Soong TT and Dargush (1997) Passive energy dissipating systems in structural engineering, Wiley, England.

Uang CM, Bertero VV. (1990). “Use of energy as a design criterion in earthquake-resistant design. ReportNo.UBC/EERC-88/18,University of California at Berkeley.

Zahrah WJ Hall (1984) “Earthquake energy absorption in SDOF systems,” *Journal of Structural Engineerin*g, 110: 1757-1772.