COMPARISON BETWEEN CRITERIA FOR SELECTING THE PARAMETERS OF HYSTERETIC ENERGY DISSIPATORS FOR SEISMIC PROTECTION OF STEEL BUILDING STRUCTURES

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Abstract. *This work deals with seismic protection of steel building structures using hysteretic energy dissipators. The sensitivity of the performance of these systems to the design parameters of the dissipative devices is numerically investigated. Particular/y, the influence of the vertical distribution of the yielding forces of the dissipators is deeply examined; two major approaches are compared. Comparison is established in ternlS of the response of a 15 story steel frame subjected to the Lorca earthquake. Initial results seem to indicate little sensitivity of that response to the design yielding forces of the dissipators.*

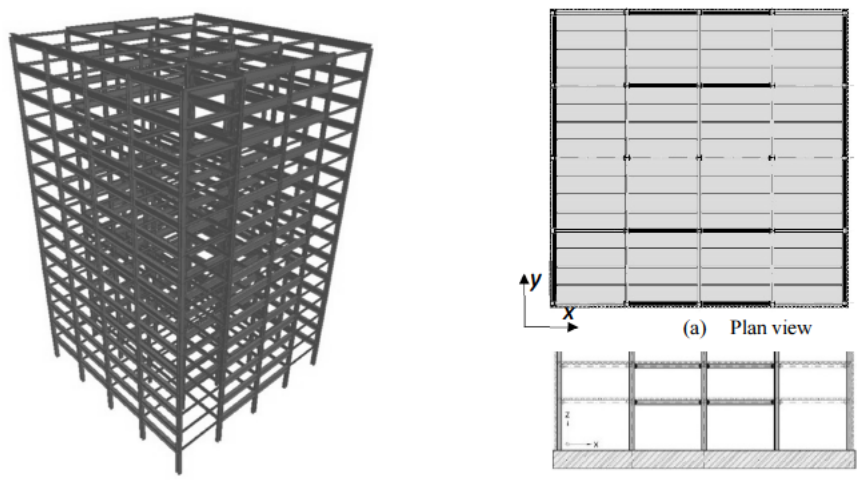
1. INTRODUCTION

The 2011 earthquake in Lorca (11-05-2011) is the most destructive event ever recorded in Spain, causing nine fatalities and other severe consequences. Its importan!intensity was rather unexpected, and a serious concern regarding the vulnerability of the building stock in Spain therefrom arose. This paper analyzes the performance, under the Lorca earthquake, of a 15-story steel frame huilding. This construction type has been chosen for being vulnerable to earthquakes and being vastly widespread in Spain. The seismic performance under the Lorca earthquake is munerically investigated through nonlinear time-history analyses. Three cases are considered: unbraced fTame (bare fTame), concentrically braced rrame with chevron braces (braced frame), and frame protected with hysteretic energy dissipators (protected rrame). Energy dissipative devices are installed connecting chevron braces with the top floor beam. Hysteretic devices (e.g. based on plastification of metals) have been chosen because their satisfactory performance, simplicity, moderate cost, robustness and low maintenance requirements. The design yielding forces of the dissipators have been obtained after the complex formulation described in the works [Benavent-Climent 20 JI, 2014] and the simpler strategy presented in [Foti et al. 1998].

Initial results highlight the capacity of dissipators to reduce the dynamic response, and the little sensitivity of that performance to the design parameters of the dissipators. Research to confirm and extend these preliminary conclusions is nowadays in progress. This work is a part of a wider research effort aiming to reduce the seismic risk in Spain by extensive use ofenergy dissipators.

2. CONSIDERED BUILDING

Figure 1~~Figure 1~~ displays overall views of the 15-story selected prototype building.



(b) 3-D representation (c) Elevation (x-z)

Figure 1 Selected 15-story prototype building

Figure 1~~Figure 1~~ shows that the considered building has 15 levels and possess frame structures in both directions. Figure 1~~Figure 1~~.a displays a plan view; this sketch shows that there are four two-bay seismic frames in x direction (inner and outer frames) and two four-bay seismic frames in y direction (outer frames). The other frames and bays have hinged connections and do not contribute significantly to the lateral resistance. Span-length is both directions is 6 m; first floor height is 4 m; in upper floors, height is 3 m. Spacing between joists is 1.50 m.

The building has been designed [López Almansa, Montaña 2014; Montaña 2014] for a seismic acceleration 0.15 g and soft soil type (with soil coefficient 1.67); corner period of design spectrum is 3 s. The importance is "normal" (dwelling, administrative or commercial use). Dead load has been assumed as 2.5 kN / m^2 (slab self-weight) + 1.5 kN / m^2 (partitioning walls) + 1 kN / m^2 (facilities) + 1.5 kN / m^2 (cladding system, distributed along the whole surface of the façade). Live load is L = 2 kN / m^2; 50% of this load is considered to act simultaneously with the seismic action. In spite that the building is symmetric, 5% accidental eccentricity is considered. Damping factor is 5%. Response reduction factor is 4.5. Design inter-story drift is 1%; this condition is the most restrictive.

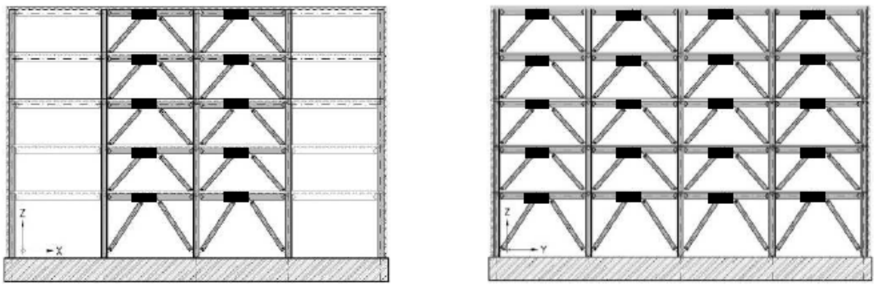
Columns, beams and joists are made of W sections and braces are made with square hollow sections (HSS). Joists are W10×15. In seismic frames, beam-column connections are pre-qualified according to [FEMA-350 2000]; the chosen type is "Welded Unreinforced Flange Bolted Web" (WUF-B), commonly known as "California post-Northridge" connection. Shear studs connecting the steel deck with the supporting horizontal members are placed only in the non-seismic elements, e.g. the joists (y direction) and those beams that do not belong to the seismic frames (x and y directions). This solution is basically intended for guaranteeing the diaphragm effect of the slabs (under lateral loading) rather than for increasing the bending stiffness and strength of beams and joists (under gravity loads). Columns are made of A-572 steel (f_y = 342 MPa) while beams and joists are made of A-36 steel (f_y = 248 MPa); this difference attempts getting earlier failures in the beams than in the columns. The compressive strength of the topping concrete is f_c' = 21 MPa; the depth of the steel deck is 50 + 70 mm (120 mm concrete depth) and its thickness is 0.75 mm. Table 1~~Table 1~~ displays the structural steel profiles for main structural members. The weight of the building is 51707 kN (dead load alone). Fundamental periods in x and y directions are 1.321 and 1.303 s, respectively.

Table 1. Structural steel members of the selected building

Floor No.	Columns	Beams	Braces
1-3	W 14 × 605	W 36 × 330	HSS 12” × 5/8”
4-5	W 14 × 550	W 36 × 300	HSS 12” × 1/2”
6-7	W 14 × 500	W 36 × 280	HSS 12” × 3/8”
8-9	W 14 × 455	W 36 × 260	HSS 12” × 5/16”
10-11	W 14 × 426	W 33 × 241	HSS 12” × 1/4”
12-13	W 14 × 398	W 33 × 221	HSS 12” × 3/16”
14-15	W 14 × 370	W 30 × 211	HSS 10” × 1/4”

3 PROPOSED PROTECTION SYSTEM

The proposed strategy consists of incorporating chevron steel bracing members and to installing energy dissipators in the connections between each bracing and the above beam; with this arrangement, the dissipative devices experience relevant strains under interstory drift motions since each bracing unit consists of the series combination of two braces and one energy dissipator. Figure 2~~Figure 2~~ displays a solution where braces and dissipators are placed on each seismic frame (to obtain plan symmetry and torsion strength) and on each story (to obtain vertical uniformity); energy dissipators are represented as ▬. Although any type of dissipative device might be employed, only hysteretic dissipators (e.g. its dissipative behavior is based on plastification of metals, commonly steel) are considered in this research. This decision has been taken since those devices are cheap, robust, simple, and have repeatedly proven its efficiency and reliability; as well, many devices have been proposed and a number of experiments, numerical simulations and applications have been reported.



(a) *x* direction (b) *y* direction

Figure 2 Proposed seismic protection using energy dissipators

The solution described in Figure 2~~Figure 2~~ can be used either for seismic design of new buildings or for retrofit of existing ones.

4 DESIGN CRITERIA FOR THE HYSTERETIC DISSIPATIVE DEVICES

The energy that can be dissipated in the whole building in a given direction cannot be obtained by merely adding the capacities of each story; it depends on the distribution, among the different stories, of the dissipated energy and on the accidental eccentricities between their centers of mass and rigidity. To cope with this issue, a number of formulations to select the variation, along the building height, of the design initial (elastic) stiffness and yielding forces of the dissipative and bracing members have been proposed; in this paper, the approach in the works [Benavent-Climent 2011, 2014] is considered. This formulation is an energy-based procedure in which the required base shear force to be provided by the dampers is obtained by establishing the energy balance of the system, and the total lateral strength distribution (i.e. frame + dampers) is determined to provide a rather uniform distribution of the cumulative ductility η in each level along the building height. The latter is achieved by adopting as lateral strength distribution the maximum shear-force distribution in an equivalent elastic undamped shear strut with similar lateral stiffness distribution along its height, subjected to a bilinear energy input spectrum. In this procedure, the earthquake hazard is characterized in terms of input energy and several seismological parameters (predominant period of the soil T_G, I_D index [Manfredi 2001], etc.) that take into account the proximity of the earthquake to the source. The study has been carried out for near-fault inputs with dimensionless index $I_D = 7.5$, medium stiffness soil with predominant period $T_G = 0.52$ s, input energy in terms of equivalent velocity $V_D = 64.6$ cm/s and ratio between the hysteretic and input energies $V_E / V_D = 0.7$ [Akiyama 1985]. Value of V_D has been obtained from the reference [Benavent-Climent et al. 2002] for moderate seismicity regions of Spain, like Lorca. The obtained values of the yielding forces of the dissipators are compared with those arising from the simpler formulation in [Foti et al. 1998]. This approach relies on representing the effect of the expected seismic action in terms of equivalent static forces; then, the yielding force at each story is selected as a given percentage of the corresponding internal shear forces in each set of dissipators in a given story and direction.

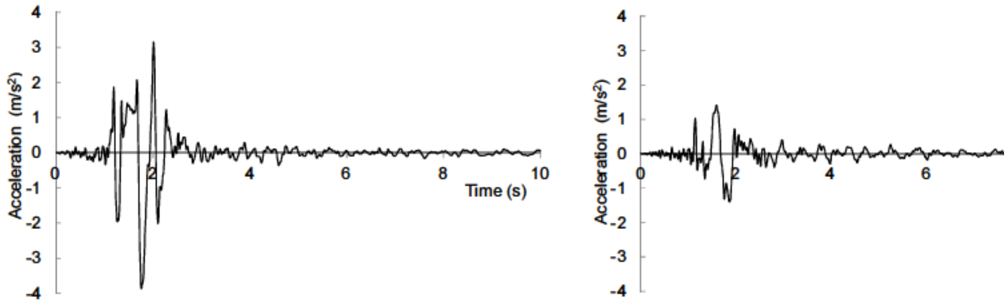
Table 2~~Table 2~~ displays the design yielding forces and initial stiffness of the dissipative devices in two major cases: in the complex approach described in [Benavent-Climent 2011, 2014] and in the simplified method in [Foti et al. 1998]. In this last case, three criteria are considered for the vertical variation of the pushing forces: triangular, uniform and sinusoidal. In the complex approach, corner period $T_G = 0.52$ s. As shown in Table 2~~Table 2~~, the same base shear yielding force (2920 kN) is assumed in the four cases. Figures in Table 2~~Table 2~~ correspond to *x* direction; yielding forces in a single floor and in a single two-bay frame. Since the simplified method does not determine initial stiffness, the same ratio between yielding forces and initial stiffness has been assumed.

Table 2. Design yielding forces (kN) / initial stiffness (kN/mm) of the hysteretic dissipators

Floor No.	Design criterion			
	[Benavent Climent 2011, 2014]	**Triangular**	**Uniform**	**Sinus**
1	2920 / 2229	2920 / 2229	2920 / 2229	2920 / 2229
2	2830 / 1338	2895 / 1369	2725 / 1288	2065 / 976
3	2737 / 836	2847 / 870	2531 / 773	1460 / 446
4	2640 / 669	2774 / 703	2336 / 592	1117 / 283
5	2537 / 446	2676 / 470	2141 / 376	902 / 159
6	2426 / 393	2555 / 414	1947 / 315	756 / 122
7	2304 / 334	2409 / 349	1752 / 254	650 / 94
8	2169 / 268	2238 / 277	1557 / 192	570 / 70
9	2016 / 239	2044 / 242	1363 / 162	507 / 60
10	1842 / 191	1825 / 189	1168 / 121	457 / 47
11	1642 / 191	1582 / 184	973 / 113	416 / 48
12	1409 / 178	1314 / 166	779 / 98	381 / 48
13	1138 / 177	1022 / 159	584 / 91	352 / 55
14	819 / 177	706 / 153	389 / 84	327 / 71
15	444 / 172	365 / 141	195 / 76	305 / 118

5 LORCA EARTHQUAKE

2011 Lorca earthquake (11-05-2011) is the most damaging seismic event ever recorded in Spain [IGN 2011]. Its magnitude is rather moderate (M_w = 5.1; [IGME 2011]), therefore, the intensity is mostly contributed by other circumstances as the extremely shallow hypocenter (the hypocentral depth is estimated as 2 km), the high proximity between the epicenter and the city center (2.9 km until the seismologic station) and the ensuing strong near-fault effects. Figure 3~~Figure 3~~ displays the most severe accelerograms [IGN 2011]; those inputs were recorded in a stiff soil site, almost rock-type. The maximum acceleration of the NS component is approximately 0.37 g, more than three times the PGA prescribed for Lorca by the current Spanish design code [NSCE-02 2002] (0.12 g).



(a) NS component (b) EW component

Figure 3 Accelerograms of the Lorca earthquake (11-05-2011)

Table 3~~Table 3~~ depicts the most relevant characteristics of the considered records. I_A is the Arias Intensity [Arias 1970] given by $I_A = \frac{\pi}{2\,g}\int \ddot{x}_g^2\,dt$ where $\ddot{x}_g$ is the input ground acceleration; the Arias intensity is an estimator of the input severity. I_D is the dimensionless seismic index [Manfredi 2001] given by $I_D = \frac{\int \ddot{x}_g^2\,dt}{PGA\,PGV}$. The dimensionless index accounts broadly for the velocity pulses content; small/big values of I_D correspond to records with/without pulses. *PI* is the pulse index [Baker 2007], which takes values between 0 and 1; records with scores above 0.85 and below 0.15 are classified as pulses and non-pulses, respectively. E_p is the relative pulse energy [Zhai et al. 2013], representing the portion of the total energy of the ground motion that corresponds to the pulse. The pulse is extracted by the peak-point method [Dickinson, Gavin 2011]. Values of E_p greater than 0.3 correspond to pulse-like records and values equal to or below 0.3 are ambiguous. The Triffunac duration is defined as the time between the 5% and the 95% of the Arias Intensity I_A [Triffunac, Brady 1975]. The bracket duration [Kempton, Stewart 2006] is comprised in between the instants when the 5% of the maximum acceleration is exceeded for the first and last time, respectively. Table 3~~Table 3~~ shows that the Lorca accelerograms are clearly pulse-like.

Table 3. Major characteristics of the records of the Lorca earthquake (11-05-2011)

Component	PGA [m/s^2]	PGV [m/s]	I_A [m/s]	I_D	*PI*	E_p	Triffunac duration [s]	Bracket duration [s]
NS	3.920	0.331	0.527	2.57	0.9995	0.72	1.005	4.035
EW	1.409	0.147	0.117	3.53	0.912	0.63	3.825	14.215

6 MODELLING OF THE DYNAMIC BEHAVIOR

The dynamic behavior of the selected building equipped with the energy dissipators is described with SeismoStruct software [Seismosoft 2013]. The structural behavior of the building is linear, while nonlinearities are concentrated in the dissipators. The monotonic nonlinear behavior of dissipative devices is described with a bilinear model; strain hardening is represented with the slope of the plastic branch being 0.05 times the elastic one. Cyclic hysteretic behavior takes into account the Bauschinger effect in the reloading branches.

Time integration is carried out with Newmark average acceleration method ($\gamma = 0.5$, $\beta = 0.25$). Time increment is 0.005 s and convergence criterion is based on displacement and rotation; displacement tolerance is 10^{-5} m and rotation tolerance is 10^{-5} rad. In each time step, maximum number of iterations is 300. Damping matrix is generated from a Rayleigh model with 5% in 1^{st} and 4^{th} modes.

7 NUMERICAL RESULTS

Figure 4~~Figure 4~~ displays the top floor time history responses in the *x* direction (Figure 1~~Figure 1~~.a) of the considered building under the NS component of the Lorca record (Figure 3~~Figure 3~~.a). Figure 4~~Figure 4~~ represents three responses: frame without any bracing (bare frame), frame with conventional concentric chevron bracing (braced frame), and frame with energy dissipators installed as indicated in Figure 2~~Figure 2~~ (protected frame). Comparison among plots in Figure 4~~Figure 4~~ confirms the capacity of energy dissipators to reduce the structural response far beyond the reduction provided by conventional concentric bracing.

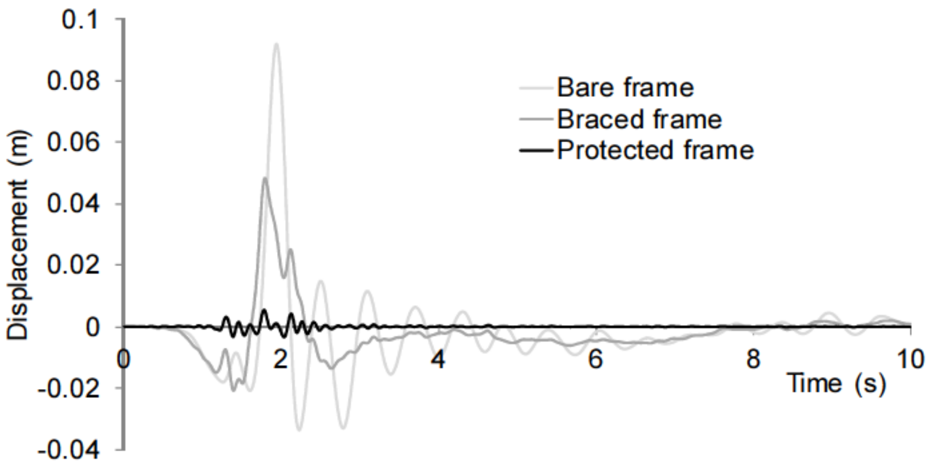


Figure 4 Time-history responses in *x* direction

In Figure 4~~Figure 4~~, the design yielding forces of the dissipators have been determined according the four criteria listed in Table 2~~Table 2~~. Only minor differences have been found among the obtained time-history responses. This big similarity seems to indicate little sensitivity of the performance of the dissipation system to the vertical distribution of the yielding forces.

8 CONCLUSIONS

This paper presents a numerical study on the capacity of hysteretic energy dissipators to reduce the time-history response of a 15-story steel building under the strongest component of the Lorca earthquake (Spain, 11-05-2011). Preliminary results provide two major conclusions:

- Dissipators reduce significantly the maximum relative displacement. This reduction clearly exceeds the one supplied by conventional concentric bracing.
- Results are highly insensitive to distribution of the yielding forces of the dissipators.

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