DESIGN CRITERIA FOR SEISMIC RETROFIT OF VULNERABLE CONCRETE FRAMES USING ENERGY DISSIPATORS

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ABSTRACT

Many buildings in Spain are vulnerable to the design earthquakes mainly because the former Spanish code required only low lateral strength and did not contain any ductility demand. Further, a large number of these structures are reinforced concrete moment resisting frames with wide beams and one-way joist slabs. Such frames are highly vulnerable due to low lateral stiffness and strength, low ductility and deficient beam-to-column moment transmission. This study is concerned with the seismic upgrading of reinforced concrete moment resisting frames with wide beams designed only for gravity loads and located in low seismicity regions, by using hysteretic energy dissipators. The objective of this study is to clarify the influence of the spatial, strength and stiffness distributions of the dissipators in the seismic response of the frame. To this end, a numerical parametric assessment is carried out. The main issues and parameters are: (1) number of floors (1 to 6), (2) plan symmetry, (3) distribution of stiffness and yielding force of the dissipators along the height of the building, (4) plan and vertical layout of the devices in the frame and (5) seismic input. The seismic performance of the frames is assessed by nonlinear static and time history analyses. The nonlinear behavior of both the main frame and the dissipators is accounted for. Performance indices are: base shear coefficient, overturning moment, interstory drifts, damage index and plastic deformation energy per floor.

KEYWORDS: Energy dissipators, Passive control, Wide beams, Spain, Concrete frames

1. INTRODUCTION

An important number of buildings in Spain are vulnerable to the current design seismic inputs because, apart from poor construction and design practices, the fact that the former Spanish code required only low lateral strength and did not contain any ductility demand. For instance, most of the structures built during the seventies, eighties and nineties in regions of Spain where the current code prescribes PGA up to 0.11g (corresponding to stiff soil and to a return period 500 years) were designed merely for gravity loads. Moreover, a large number of these structures are reinforced concrete moment resisting frames with wide beams and one-way joist slabs. Such frames are highly vulnerable due to their relevant seismic deficiencies: (1) low lateral stiffness and strength, (2) low ductility and (2) deficient beam-to-column moment transmission. Although the seismic hazard is rather low, since the vulnerability is high, these structures have a noticeable risk of suffering severe damage or even collapse when are subjected to the current design earthquake.

This study is concerned with the seismic upgrading of buildings located in low seismicity regions by using hysteretic energy dissipators. Such buildings possess reinforced concrete moment resisting frames with wide beams designed only for gravity loads. The objective of this study is to clarify the influence of the spatial, strength and stiffness distributions of the dissipators in the seismic response of the frame. To this end, a numerical parametric assessment is carried out. The main parameters and issues are:

- Geometry of the building frames: number of floors (1-6), plan symmetry.
- Stiffness and yielding force of the dissipators along the height of the building.
- Plan and vertical layout of the devices in the frame.
- Seismic input. Historic accelerograms registered in Spain and in close or similar regions are considered; will correspond to very soft, soft, rigid or rock soils. These inputs are scaled to provide similar damage potential.

The seismic performance of the frames is assessed by nonlinear static analyses (push-over) and by nonlinear time history analyses. The nonlinear behaviors of both the main frame and the dissipators are accounted for. Performance indices are: base shear coefficient, overturning moment, interstory drifts, damage index and plastic deformation energy per floor.

This work belongs to a bigger research project (funded by the Spanish government) aiming at proposing advanced seismic retrofit strategies for vulnerable buildings located in low-to-moderate seismicity regions in Spain.

2. CONSIDERED BUILDINGS

One of the first objectives of the research is to study the main features of the existing buildings. In order to do this, the seismic area of Spain has been divided in two zones termed A (high seismicity, according to Spanish standards) and B (mid seismicity). In both zones the consideration of design codes PDS-74 (Comisión Permanente de Normas Sismorresistentes, 1974), PDS-94 (Comisión Permanente de Normas Sismorresistentes, 1994) and NCSE-02 (Comisión Permanente de Normas Sismorresistentes, 2002) is compulsory. Zone A corresponds basically to the Granada province (where the current code NCSE-02 states a seismic acceleration for stiff soil a_c ranging in between 0.16 g and 0.25 g) while zone B corresponds to a small part of Catalonia close to the French border (Puigcerdà, Castefollit de la Roca, Montagut, etc.) where the current code NCSE-02 states a seismic acceleration a_c ranging in between 0.08 g and 0.11g. This paper focuses in zone B.

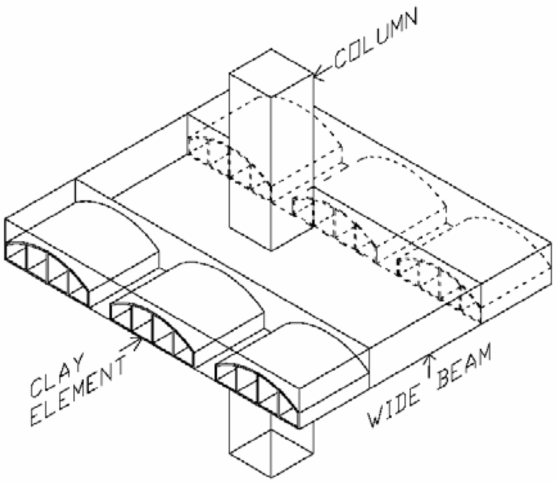

Figure 1. Wide beam-column connection

All the considered buildings possess reinforced concrete structures with columns, continuous down to the foundation, and one-way joist slabs. Such slabs are flat, both at the upper and lower sides. They are composed of (1) beams, (2) any type of concrete joist (prestressed, cast in situ, semi prefabricated, among other solutions), (3) nonstructural clay hollow elements placed between the joists and (4) a concrete topping layer (such layer is reinforced to guarantee the continuity among the joists for negative bending moments). Obviously, the remaining space is filled with concrete. The beams are wide; their width can reach up to 90 cm for inner frames and up to 60 cm for outer ones. It is remarkable that in most of the cases the beams are significantly wider than the columns, i.e., their sides are cantilevered (spandrel beams). Figure 1 describes a rigid connection among a column and a wide beam; some joists and nonstructural clay hollow elements can also be seen. The outer joists had to withstand the cladding but they had the same depth and width than the inner ones and only the reinforcement is different.

The basic features of the buildings erected between 1974 and 1994 have been obtained by asking highly experienced designers and supervisors (most of them being also University professors) and by field work. The features that are relevant for seismic behavior are summarized in Table 1. It is noteworthy that the vast majority of the consulted designers have stated that the seismic loads were irrelevant compared to the gravity ones and, hence, were commonly disregarded.

Table 1 Buildings with wide beams in zone B

Number of floors	In between 1 and 6
Number of spans	High variation; frequently 3 to 4 spans per direction
Span-length	In between 5 and 5.5 m
Story height	About 3 m; the first floor is commonly 4 m
Beams depth and width	Depth: 22-25 cm (up to 29 cm); it includes the concrete topping layer (5 cm). Width: 60-75 cm (up to 90 cm)
Joists	Any type of concrete joist (prestressed, cast in situ, semi prefabricated, among other solutions). The separation among the axles of contiguous joists is 60 cm.
Columns' size	Usually the top floor columns were 30 × 30 cm^2; the size increased 5 or 10 cm every two or three floors. In the lower floor could be 30 × 60 cm^2
Shear walls	Up to 6-storey buildings, there were no structural walls
Plan symmetry	Isolated buildings have regular plan layout while town buildings have irregular configurations. Cladding non-structural walls generate eccentricity between the centres of mass and of rigidity
Torsion strength	Columns are uniformly distributed; hence, the torsion strength is acceptable. The façade columns have their biggest size in the façade plan
Mechanical parameters of the materials	Concrete compressive characteristic strength: f_{ck} = 17.5 MPa; steel yielding point: f_{yk} = 400 MPa. After 1988, f_{ck} = 20-40 MPa and f_{yk} = 500 MPa
Beam reinforcement detailing	The top and bottom longitudinal reinforcements were laid in single layers. For widths bigger than about 50 cm, the (continuous) construction reinforcement consisted of 4 (bottom) and 3 (top) bars; their diameter were 16 or 20 mm. The transversal reinforcement consisted basically of equally distributed stirrups; for higher shear forces, additional stirrups were laid near the connections. The stirrups were not properly closed (90º, instead of 135º). In extremely wide beams (90 cm for inner beams and 60 cm for outer ones), additional transversal reinforcement was laid to avoid shear punching.
Column reinforcement detailing	The longitudinal reinforcement amount ranged between 1% and 6%. The longitudinal bars were placed mainly at the corners; separations bigger than 15 cm were frequent. The overlapping was situated in the sections situated immediate above the connections (slabs). The stirrups were not continued inside the connections.

After the information in Table 1, 12 representative prototype buildings have been defined, as described in Table 2. All the prototypes have 4 spans in both directions. The first floor is 4.5 m high while the other floors are 3 m high. In buildings PB1, PB2 and PB3 the (square) columns are 40 × 40 cm^2 in the first floor and 30 × 30 cm^2 in the two upper floors. In buildings PB4, PB5, PB6, PB7 and PB8 the (square) columns are 40 × 40 cm^2 in the four lower floors and 30 × 30 cm^2 in the two upper floors. In buildings PB9, PB10, PB11 and PB12 the (rectangular) columns are 45 × 35 cm^2 in the three lower floors and 40 × 30 cm^2 in the three upper floors; the biggest size belongs to the frames plane except in the outer columns (have their biggest size in the façade plan). It is remarkable that the concrete compressive characteristic strength corresponds to 28 days; the value for more aged concrete has been estimated (CEB-FIP 1990) by multiplying by the factor 1.284: $f_{ck}(\infty)$ = 17.5 × 1.284 = 22.47

MPa. The secant concrete deformation modulus is calculated for this strength according to the prescriptions of the former Spanish design code (Comisión Permanente del Hormigón, 1980): E_c = 28481 MPa. In the last column in Table 2 "Overall torsion" refers to the existence of stiff cladding walls that generate relevant eccentricities among the centers of rigidity and of mass.

Table 2 Prototypes of buildings in zone B

Prototype	Number of floors (N)	Span-length for main beams (L) (m)	Span-length for joists (L') (m)	Slab depth and width	Joists ($h \times b$) (cm)	Columns ($h \times b$) (cm)	Overall torsion
PB1	3	5	5	25 × 60	25 × 10	Square	Yes
PB2	3	5	5	25 × 60	25 × 10	Square	No
PB3	3	5.50	5.50	29 × 75	29 × 10	Square	Yes
PB4	3	5.50	5.50	29 × 75	29 × 10	Square	No
PB5	6	5	5	25 × 60	25 × 10	Square	Yes
PB6	6	5	5	25 × 60	25 × 10	Square	No
PB7	6	5.50	5.50	29 × 75	29 × 10	Square	Yes
PB8	6	5.50	5.50	29 × 75	29 × 10	Square	No
PB9	6	5	5	25 × 60	25 × 10	Rectangular	Yes
PB10	6	5	5	25 × 60	25 × 10	Rectangular	No
PB11	6	5.50	5.50	29 × 90	29 × 10	Rectangular	Yes
PB12	6	5.50	5.50	29 × 90	29 × 10	Rectangular	No

The fundamental periods of the prototype buildings (it corresponds to the direction of the beams) are computed by a simplified Rayleigh-Ritz approach by representing each frame by an equivalent SDOF system; the loading pattern is constant along the height of the building in the three storey buildings and is triangular (proportional to the height) in the six storey ones. The mass corresponds to all the dead loads plus half of the live ones ($D + 0.5\,L$). For building PB3 the fundamental period is T_1 = 1.11 s while for building PB6 is T_1 = 2.12 s. These values correspond to extremely flexible frames; it is due to the following reasons: (1) the first floor is highly flexible because of its important height (4.5 m), (2) the beams are wide (flat) and (3) the concrete deformation modulus is low because of its low compressive strength. The reduction of stiffness of the beams due to cracking generated by the gravity loads has not been accounted for; it would have further lengthened the periods.

2. SEISMIC INPUTS

A number of registered seismic inputs have been selected; given the scarcity of the available information for the considered area, other Mediterranean locations are used instead. The processing procedure consisted of a linear baseline and an eighth-order elliptical band-pass filter with cut-off frequencies of 0.25 and 25.00 Hz. No instrument correction was applied because many of the records were not associated with reliable instrument characteristics; this affects only the high frequencies. The accelerograms have been sorted out in accordance with the four soil types defined by the current Spanish seismic design code [Comisión Permanente de Normas Sismorresistentes, 2002]; such norm classifies the soil in four categories according to the shear wave velocity: I (more than 750 m/s, rock), II (in between 400 and 750 m/s, stiff), III (in between 200 and 400 m/s, soft) and IV (less than 200 m/s, very soft). Tables 3, 4, 5 and 6 display the main features of the selected inputs for each of the abovementioned soil types, respectively. All these inputs are scaled to fit the design seismic acceleration of 0.11g.

Table 3 Seismic inputs for rock (v_s > 750 m/s)

Name	Station	Date	Time [UTC]	Magnitude	Fault distance (km)	PGA (m/s^2)
Friuli	Tolmezzo-Diga Ambiesta	06/05/1976	20:00:13	6.5 Mw	27	3.499
Ardal	Naghan 1	06/04/1977	13:36:37	6 Mw	5	8.907
Tabas	Dayhook	16/09/1978	15:35:57	7.4 Mw	11	3.779
Montenegro	Hercegnovi Novi-O.S.D. Pavicic School	15/04/1979	06:19:41	6.9 Mw	65	2.509
Campano Lucano	Sturno	23/11/1980	18:34:52	6.9 Mw	32	3.166
Timfristos	Karpenisi-Prefecture	14/06/1986	07:40:39	3.7 ML	9	3.019
Timfristos	Karpenisi-Prefecture		09:49:18	3.3 ML	8	2.845
Izmit	Sakarya-Bayindirlik ve Iskan Mudurlugu	17/08/1999	00:01:40	7.6 Mw	34	3.542

Table 4 Seismic inputs for stiff soil (400 m/s < v_s < 750 m/s)

Name	Station	Date	Time [UTC]	Magnitude	Fault distance (km)	PGA (m/s^2)
Friuli (aftershock)	Forgaria-Cornio	11/05/1976	22:44:01	4.9 Mw	3	3.001
Friuli (aftershock)	Forgaria-Cornio	11/09/1976	16:35:03	5.5 Mw	16	2.273
Friuli (aftershock)	Breginj-Fabrika IGLI	15/09/1976	03:15:19	6 Mw	18	4.956
	Forgaria-Cornio				17	2.586
Friuli (aftershock)	Breginj-Fabrika IGLI		09:21:19	6 Mw	22	4.136
	Forgaria-Cornio				17	3.395
	San Rocco				17	2.319
Friuli (aftershock)	Forgaria-Cornio	16/09/1977	23:48:08	5.4 Mw	5	2.365
Tabas	Tabas	16/09/1978	15:35:57	7.4 Mw	52	10.805
Montenegro	Bar-Skupstina Opstine	15/04/1979	06:19:41	6.9 Mw	16	3.68
	Petrovac-Hotel Oliva				25	4.453
Montenegro (aftershock)	Bar-Skupstina Opstine	24/05/1979	17:23:18	6.2Mw	33	2.652
	Budva-PTT				8	2.624

3. STRUCTURAL MODELING OF THE BUILDINGS

For both the static and the dynamic analyses, the buildings have been modeled by a finite element code. The columns, joists and beams are described by frame elements with rigid connections among them. The joists and the topping concrete layer are considered together as T-section bars where the web is the joist and the upper flange is the concrete layer; the effective width encompasses the whole width of the topping layer (60 cm). The joists-topping concrete assemblies are considered as continuous, thanks to the contribution of the top layer. Both the dead and live loads are applied to the joists as distributed forces along their length. The contribution of the top concrete layer to the bending stiffness of the slab in the direction orthogonal to the joists is neglected and is only contributed by the beams. The columns are considered clamped in the foundation.

Table 5 Seismic inputs for soft soil (200 m/s < v_s < 400 m/s)

Name	Station	Date	Time [UTC]	Magnitude	Fault distance (km)	PGA (m/s^2)
Ionian	Lefkada-OTE Building	04/11/1973	15:52:12	5.78 Ms	5.146	1.095
Friuli (aftershock)	Buia	11/09/1976	16:35:03	5.5 Mw	2.26	0.862
Montenegro	Ulcinj-Hotel Olimpic	15/04/1979	06:19:41	6.9 Mw	2.88	4.49
Alkion	Korinthos-OTE Building	24/02/1981	20:53:37	6.6 Mw	3.036	1.138
Kalamata	Kalamata-OTE Building	13/09/1986	17:24:34	5.9 Mw	2.67	1.965
Kalamata (aftershock)	Kalamata-OTE Building	15/09/1986	11:41:28	4.9 Mw	2.355	1.017
Ionian	Lefkada-Hospital	24/04/1988	10:10:33	4.8 Mw	2.705	0.376
Off coast of Levkas island	Lefkada-Hospital	24/08/1988	10:10:33	4.5 ML	2.369	0.253
Sicilia-Orientale	Catania-Piana	13/12/1990	00:24:26	5.6 Mw	2.483	0.682
Pyrgos	Pyrgos-Agriculture Bank	26/03/1993	11:58:15	5.4 Mw	4.256	1.186
Patras	Patra-San Dimitrios Church	14/07/1993	12:31:50	5.6 Mw	3.337	1.199
Izmit	Ambarli-Termik Santrali	17/08/1999	00:01:40	7.6 Mw	2.58	0.779
	Duzce-Meteoroloji Mudurlugu				3.542	2.014

Table 6 Seismic inputs for very soft soil (v_s < 200 m/s)

Name	Station	Date	Time [UTC]	Magnitude	Fault distance (km)	PGA (m/s^2)
Gazli	Karakyr Point	17/05/1976	02:58:42	6.7 Mw	11	7.065
NE of Banja Luka	Banja Luka-Borik 2	13/08/1981	02:58:12	5.7 Mw	7	2.555
	Banja Luka-Borik 9				7	3.551
	Banja Luka-Institut za Ispitivanje Materijala				7	4.34

4. NONLINEAR STATIC ANALYSES

The seismic performance of the considered buildings is numerically assessed by nonlinear static analyses (push-over) following the prescriptions of the Applied Technology Council (ATC 1996). The second-order effects have not been considered. The lateral loads patterns correspond to the first mode multiplied by the mass of each floor. The vertical forces correspond to all the dead loads plus half of the live ones ($D + 0.5\ L$). The elastic demand design spectrum is determined according to the requirements of the current Spanish seismic design code (Comisión Permanente de Normas Sismorresistentes, 2002) assuming normal importance of the buildings (housing occupancy) and a seismic acceleration $a_c = 0.11$g. Figure 2 shows the capacity curve (base shear vs. top floor horizontal displacement in the direction of beams) of building PB3. Figures 3 and 4 display views of the plastic hinges formation in an inner and outer frame, respectively. Figures 3 and 4 show that the behaviour of the inner frames is particularly not adequate since the hinges appear in the columns instead of in the beams.

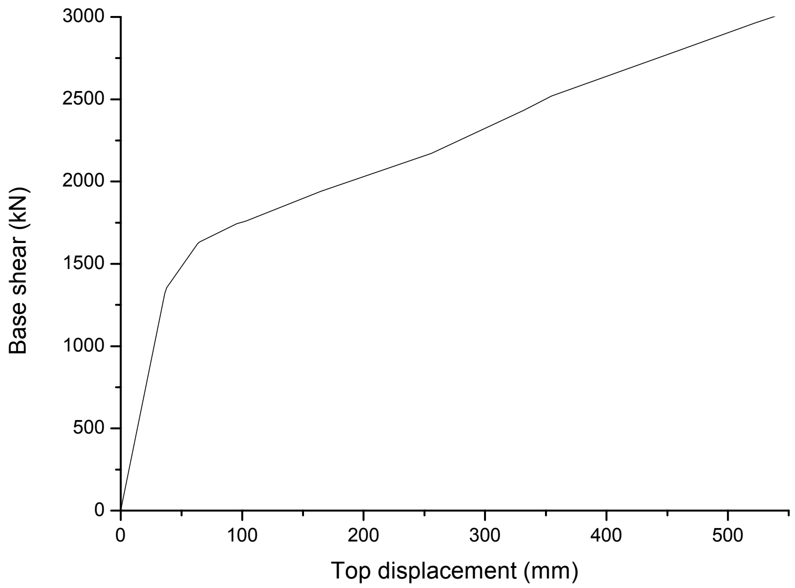

Figure 2. Push-over analysis of building PB3

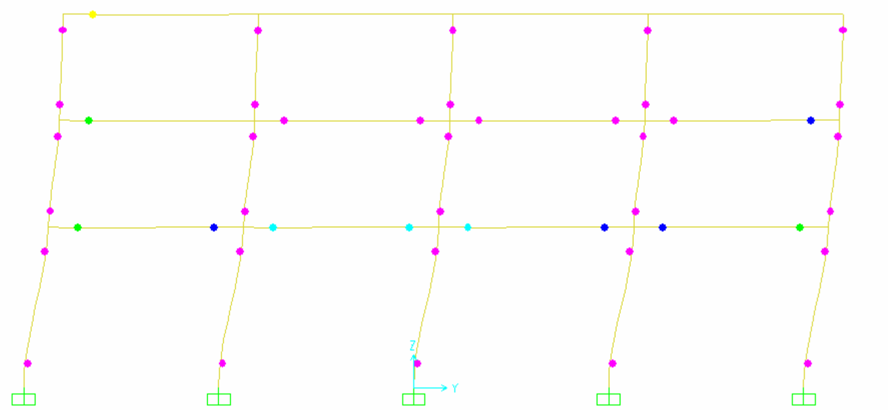

Figure 3. Plastic hinges near collapse in building PB3 for an outer frame

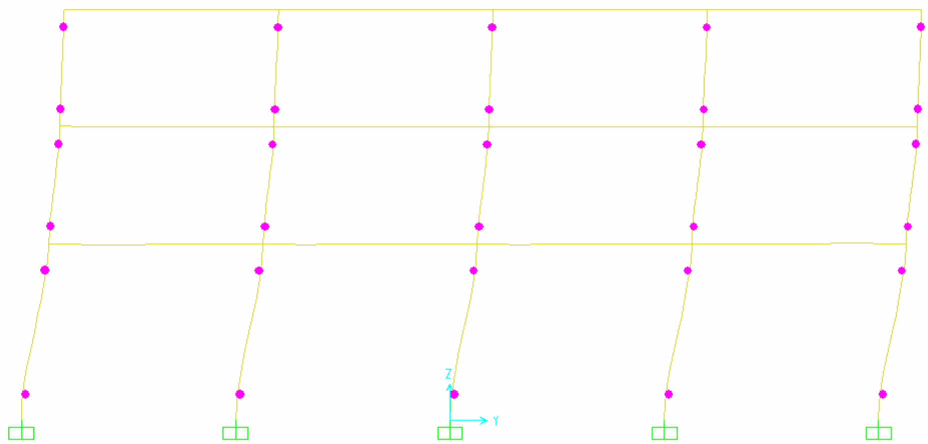

Figure 4. Plastic hinges near collapse in building PB3 for an inner frame

5. NONLINEAR TIME HISTORY ANALYSES

The seismic performance of the considered buildings is further numerically assessed by nonlinear dynamic analyses for the selected earthquakes. The assumed viscous damping follows the Rayleigh model and corresponds to 5% of the critical. The hysteretic properties are derived according to (Benavent-Climent 2005; Benavent-Climent 2007). No results are available in the moment this paper is written.

6. SEISMIC RETROFIT

The objective of this study is to proposed seismic retrofit strategies based in the incorporation of energy dissipators to the main frame. Several devices, mainly hysteretic, will be considered. A numerical parametric assessment will be carried out to clarify the influence of the spatial, strength and stiffness distributions of the dissipators in the seismic response of the frame. The main issues and parameters are: (1) number of floors (1 to 6), (2) plan symmetry, (3) distribution of stiffness and yielding force of the dissipators along the height of the building, (4) plan and vertical layout of the devices in the frame and (5) seismic input. No results are available in the moment this paper is written.

7. CONCLUSIONS

This paper presents the general approach, methodology and preliminary results of a research project now in progress, which involves four Spanish Universities, one Japanese University and one American University. This project is sponsored by the Spanish Ministry of Education. The research is aimed at: (1) evaluating the seismic vulnerability of reinforced concrete frame structures with wide beams, built between 1974 and 1994 in moderate seismicity areas of Spain; and (2) upgrading seismically these frames by using energy dissipators. Both experimental and numerical approaches are applied. For the experimental part, six wide beam-column connections were recently tested under static cyclic loading up to collapse, in order to clarify their hysteretic behavior and quantify their ultimate energy dissipation capacity. For the numerical part, a series of dynamic response analyses of frames are currently in progress. The numerical models, calibrated with the test results, represent typical reinforced concrete frames with wide beams, designed for the northern part of Spain by applying the old Spanish seismic code, which was in force until 1994. This paper focuses on these numerical simulations. The main expected outputs of this research are: (1) simple algorithms for quantifying the earthquake resistance of this type of structures, (2) a strategy for seismic upgrading the frames with energy dissipators and (3) a methodology for designing the dissipators.

ACKNOWLEDGEMENTS

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