Shaking table tests of a RC frame structure equipped with hysteretic dampers

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SUMMARY
This paper provides partial results of an on-going research aimed at investigating the seismic response of reinforced concrete (RC) frames equipped with hysteretic-type energy dissipating devices (EDD). From a prototype RC frame structure designed only for gravity loads, a test model scaled in geometry to 2/5 was defined and built in the Laboratory of Structures of the University of Granada. Four EDDs were installed in the test model to provide the same seismic resistance than a conventional RC bare frame designed for sustain gravity and seismic loads following current codes. The test model with EDDs was subjected to several seismic simulations with the shaking table of Laboratory of structures of the University of Granada. The test results provide empirical evidences on the efficiency of the EDDs to prevent damage on the main frame and concentrating the inelastic deformations on the EDDs.

Keywords: RC frame, hysteretic damper, shake table test, passive control

1. INTRODUCTION

The use of passive energy dissipating systems for seismic design of structures has increased exponentially in recent years, for both new and existing buildings. The main objective of energy dissipating devices (EDD) is to concentrate the energy demand in the EDDs, reducing the damage imparted to the framing system (Constantinou and Symans, 1993). EDD's are capable of minimizing inter-story drifts and increasing the overall earthquake resistance of the buildings to achieve performance-based design objectives. Among the different types of passive EDD's the most commonly used in seismic design are viscous fluid dampers, viscoelastic solid dampers, friction dampers and metallic dampers (Symans et al., 2008). EDD's based on the yielding of metals—commonly known as hysteretic dampers—are among the most popular.

This study is aimed at investigating the seismic behaviour of RC frame structures equipped with hysteretic dampers through shaking table tests. A main frame structure was designed to resist only gravity loads while the lateral strength was provided by hysteretic dampers. The mixed system (main frame+dampers) was subjected to several seismic simulations with the shaking table of the Laboratory of Structures of the University of Granada. The results of the tests suggest that the EDDs control satisfactorily the response of the mixed system without need to imposing capacity design criteria to the main frame, that is, without need to design a strong column-weak beam failure mechanism for the main frame.

2. PROTOTYPE

The prototype building considered in this study is the three story structure shown in Figure 1. The structure was designed by applying limit state methods to resist only the gravity loads. The following loads were considered: (i) self-weigh of the floor plus dead loads: 3.22 kN/m^2; (ii) self-weight of the roof plus dead loads: 2.95 kN/m^2; (iii) live loads 2 kn/m^2. A compressive strength of 25 MPa for concrete and a yielding strength of 500MPa for the steel reinforcement was considered in the design of the members. The section of the columns in the prototype structure was 30×30cm and the longitudinal reinforcing ratio was ρ= 0.5%. The cross section of the beams (width×depth) was 30×25cm in all stories. The lateral strength to resist earthquake loads was provided by hysteretic dampers at each level.

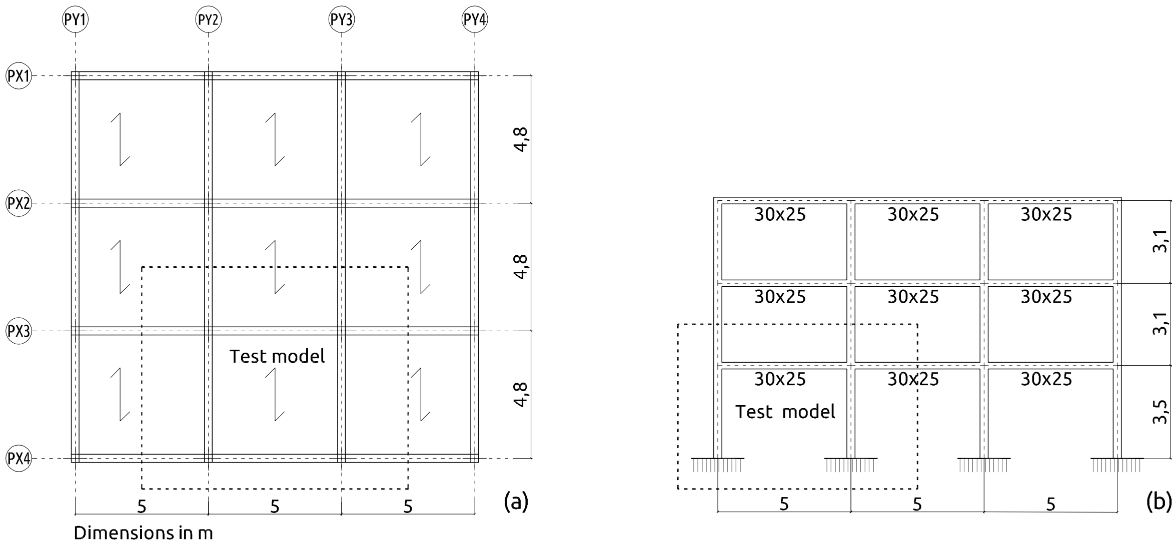

Figure 1. Prototype structure: (a) plan; (b) elevation

3. TEST SPECIMEN

The test specimen was a portion of the prototype structure delimitated by the dash lines in Figure 1, consisting of one and half spans, and one and half stories. The test specimen structure was extracted from the prototype building by cutting at mid span of the beams, and at the middle height of the columns, where the bending moment due to lateral loads is approximately zero. To simulate the gravity loads of the upper part of the building and to satisfy similitude laws, additional masses in form of steel blocks were added on the top of the test specimen. The boundary conditions at mid span of the beams and at mid height of the columns were simulated with pin connections and vertical struts. Scale factors for stress λ_σ and acceleration λ_a were unity, while length scale factor λ_L was 2/5. The scale factors indicated in Table 1 were used so as to satisfy similitude requirements for dynamic loading (Harris & Sabnis, 1999). Figure 2 shows the geometry and reinforcement details of the RC frame.

The seismic resistance of the test model was provided by installing two hysteretic dampers at each story. The hysteretic damper used in the tests has the form of a conventional brace and it is installed in the main structure as a standard diagonal bar. Each hysteretic damper is constructed by assembling several short length segments of I-shaped steel sections which constitute the energy dissipating device, and two U-shaped steel bars that function as auxiliary elements. The seismic damper dissipates the energy through plastic strains on the web of the I-shaped sections under out-of-plane flexure. The auxiliary elements are designed to remain elastic. A detailed description of the hysteretic damper can be found in (Benavent-Climent et al. 2011). The mechanical properties of the dampers (i.e. yield strength and stiffness) were determined using an energy based method proposed by Benavent-Climent (Benavent-Climent, 2011). The dampers were designed so they can resist a design earthquake with an energy input representative to the seismic hazard in Granada (Benavent-Climent et al. 2002).

Table 1. Scaling factors

Physical quantity	Scaling law	Units	Scaling factor
Length	λ_L	L	2/5
Stress	λ_σ	FL^{-2}	1
Acceleration	λ_a	LT^{-2}	1
Force	$\lambda_{F=}(\lambda_L)^2\ \lambda_\sigma$	F	0.16
Surface	$\lambda_V=(\lambda_L)^2$	L^2	0.16
Volume	$\lambda_V=(\lambda_L)^3$	L^3	0.064
Moment	$\lambda_M=\lambda_F\ \lambda_L$	FL	0.064
Time	$\lambda_{T=}(\lambda_L)^{-2}/\lambda_a$	T	0.63
Strain	λ_ε	L/L	1

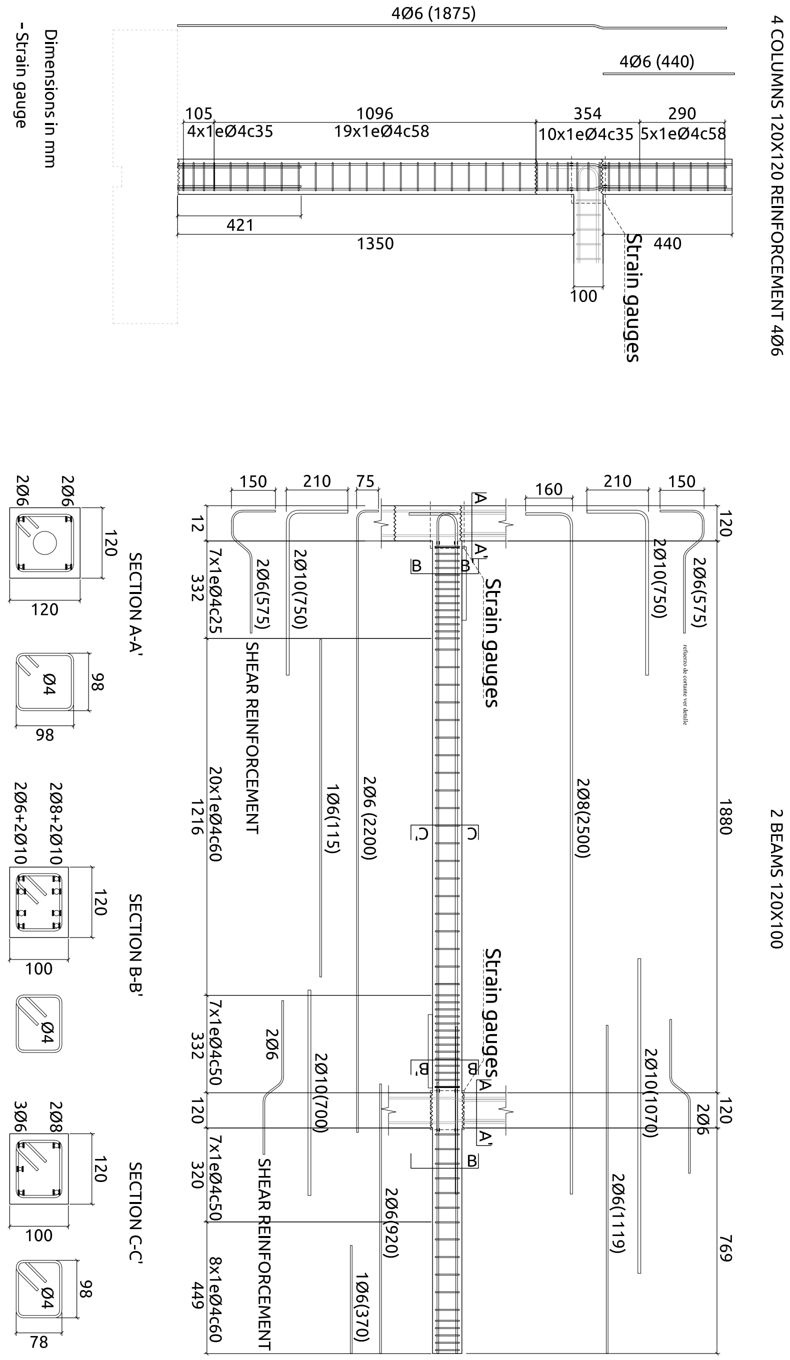

Figure 2. Geometry and reinforcement details

The specimen was constructed in 4 stages (foundation, first story columns, slab, second story columns). Tension tests were conducted on samples of reinforcing bars of each lot and size. Compression tests were conducted on normalized concrete cylinders. Table 2 summarizes the results for the material strength tests.

Table 2. Mechanical properties of materials

Material	Yield strength (MPa)
Concrete in columns (28th day)	-34.9
Concrete in slab (28th day)	-34.7
Concrete in columns (test day)	-40.9
Concrete in slab (test day)	-39.2
Longitudinal reinforcement	551.1
Stirrups	636.2

4. SHAKING TABLE TESTS

4.1. Test set-up

The test model was clamped to the 3mx3m shaking table test at University of Granada as shown in Figure 3. The shaking table was calibrated by moving it with the specimen mounted on it under acceleration control as follows. First a flat-shape random signal of root mean square (RMS) amplitude of about 0.05g was applied to adjust the parameters required by the TVC (Three Variable Control) controller. Second the AIC (Adaptive Inverse Control) controller was trained by moving the shaking table with a random signal with RMS amplitude of about 0.05g. Third the OLI (On-line Iteration) controller was trained through an iterative process in which the shaking table was subjected to the desired accelerogram scaled to a low intensity of 0.1g. All this process was conducted until the errors between the desired acceleration and the actual acceleration measured on the table reduced to acceptable values.

Figure 3. Test set-up

3.2. Instrumentation

Displacements, strains and accelerations were collected simultaneously by a HBM MGC Plus data acquisition system, using a sampling rate of 200Hz. The test model was instrumented with the following sensors that are shown in Figure 4:

a) Two displacement transducers (LVDTs) measured the relative displacement, v, in the direction of the seismic loading at each level. LVDTs 1-2 measured the displacement between the shaking table and the first story and LVDTs 4-5 measured the relative displacement between the first story and the added weight of second story
b) Two additional LVDTs measured the relative displacement in the direction perpendicular to the seismic loading at each level. LVDT 3 measured the displacement between in the first story an LVDT-6 in the second story.
c) Two pairs of seismic and piezoelectric accelerometers measured the absolute acceleration, $\ddot{v}^t$, at each level. The pairs labelled as seismic 1-2 and piezoelectric 1-2 sensors measured the acceleration in the first story while the pair formed by seismic 5-6 and piezoelectric 5-6 sensors measured the acceleration at the second story.
d) Two seismic accelerometers measured also the absolute acceleration in the direction perpendicular to the seismic loading. The accelerometer referred to as seismic-3 in Figure 4 measured the accelerations at the first story while accelerometer seismic-7 measured the accelerations at the second story.
e) One piezoelectric accelerometer labelled as piezoelectric 4 measured the acceleration of the shaking table $\ddot{v}_g$.
f) 218 strain gauges attached to the steel reinforcement prior to casting measured the strains in the vicinity of the plastic hinge regions located at beam and column ends.

Each damper was instrumented with the following sensors:

a) One LVDT measured the axial deformations of each brace damper. LVDTs 9-10 measured the displacements of the dampers of the first story while LVDTs 11-12 did in the second.
b) 4 strain gauges attained at each end of the brace dampers measured the strains that allowed to calculate the axil force sustained by the brace damper.

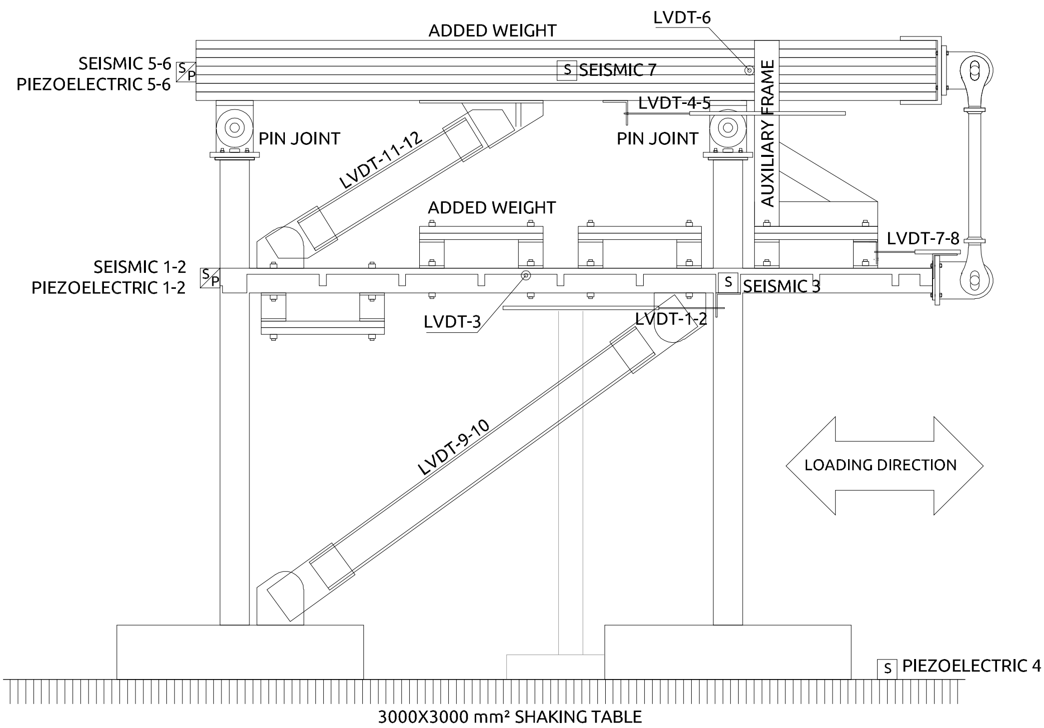

Figure 4. Instrumentation

3.3. Loading history.

The models were tested by applying to the shake table the acceleration record measured in Calitri (Italy) during the 1980 NS Campano-Lucano earthquake. The original acccelerogram shown in figure 5 was scaled in time by $\lambda_t = \sqrt{1/2}$ and in amplitude to different values in order to simulated different levels of shaking. Figure 6 shows the 5% damped elastic response spectra in terms of the absolute response acceleration S_{pa}, the relative response velocity, S_{pv} and the relative displacement S_{pd}. Each specimen was subjected to a series of consecutive seismic simulations. In each seismic simulation the acceleration, $\ddot{v}_g$, of the original record was scaled by multiplying by factor of 0.5, 1, 2, 3 and 3.5 to meet different seismic hazard levels at the building site. Table 3 shows the peak ground acceleration, PGA, the expected structural performance level SPL, and the return period associated to each seismic simulation according to the Spanish seismic code for Granada (Spain) and soft soil conditions (MFOM, 2002).

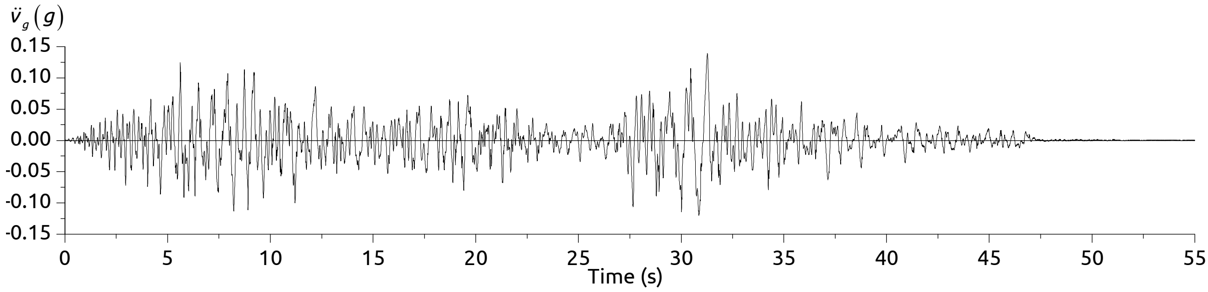

Figure 5. Accelerogram used during the test

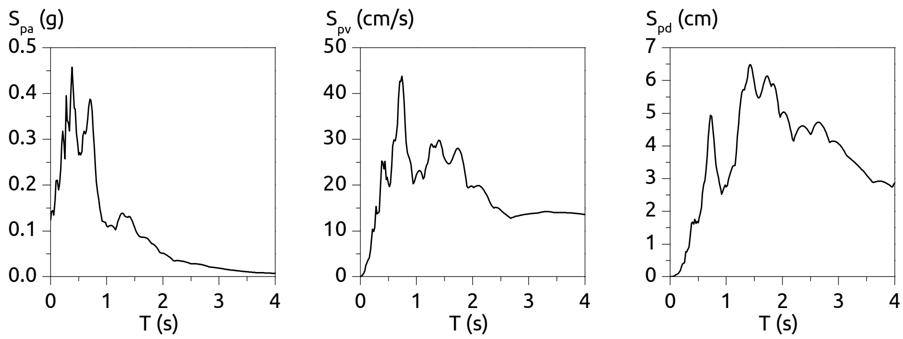

Figure 6. Elastic response spectra

Table 3. Seismic Simulations

Seismic simulation	PGA (g)	Return period (years)	Expected SPL
c50	0.08	59	Immediate occupancy (IO)
c100	0.15	81	Immediate occupancy (IO)
c200	0.31	500	Immediate occupancy (IO)
c300	0.47	1428	Life Safety (LS)
c350	0.54	4223	Collapse prevention (CP)

5. TEST RESULTS

Figure 7 shows the displacement histories at the first and second stories during the different seismic simulations. Figure 8 shows the acceleration histories during the tests. Based on the experimental results and the onsite observations after each test, the following appreciations can be done. After the seismic simulation c50 the whole structure remained elastic. After the seismic simulation c100 the RC frame remained elastic while the dampers of the first story reached the yield axial displacement. After the seismic simulation c200 the reinforcement at the base of the columns of the first story yielded with maximum strains of ε=3160μm/m. The dampers in both stories suffered moderate inelastic deformations. After the seismic simulation c300 the reinforcement at the base of the columns suffered moderate inelastic deformations with maximum strains on the longitudinal reinforcement of ε=9000 μm/m. The reinforcement at the top section of the columns of the first story yielded reaching maximum strains of ε=3055 μm/m. The dampers of the first story suffered important inelastic deformations while the dampers of the second story remained with moderate inelastic deformations. Following the seismic simulation c350 the reinforcement at the bottom end of the columns of the first story suffered important inelastic deformation with maximum strains of ε=16782 μm/m. The longitudinal reinforced of the top end of the columns of the first story suffered slight inelastic deformations. The dampers at the first story were near to collapse, while the dampers in the second story suffered moderate inelastic deformations.

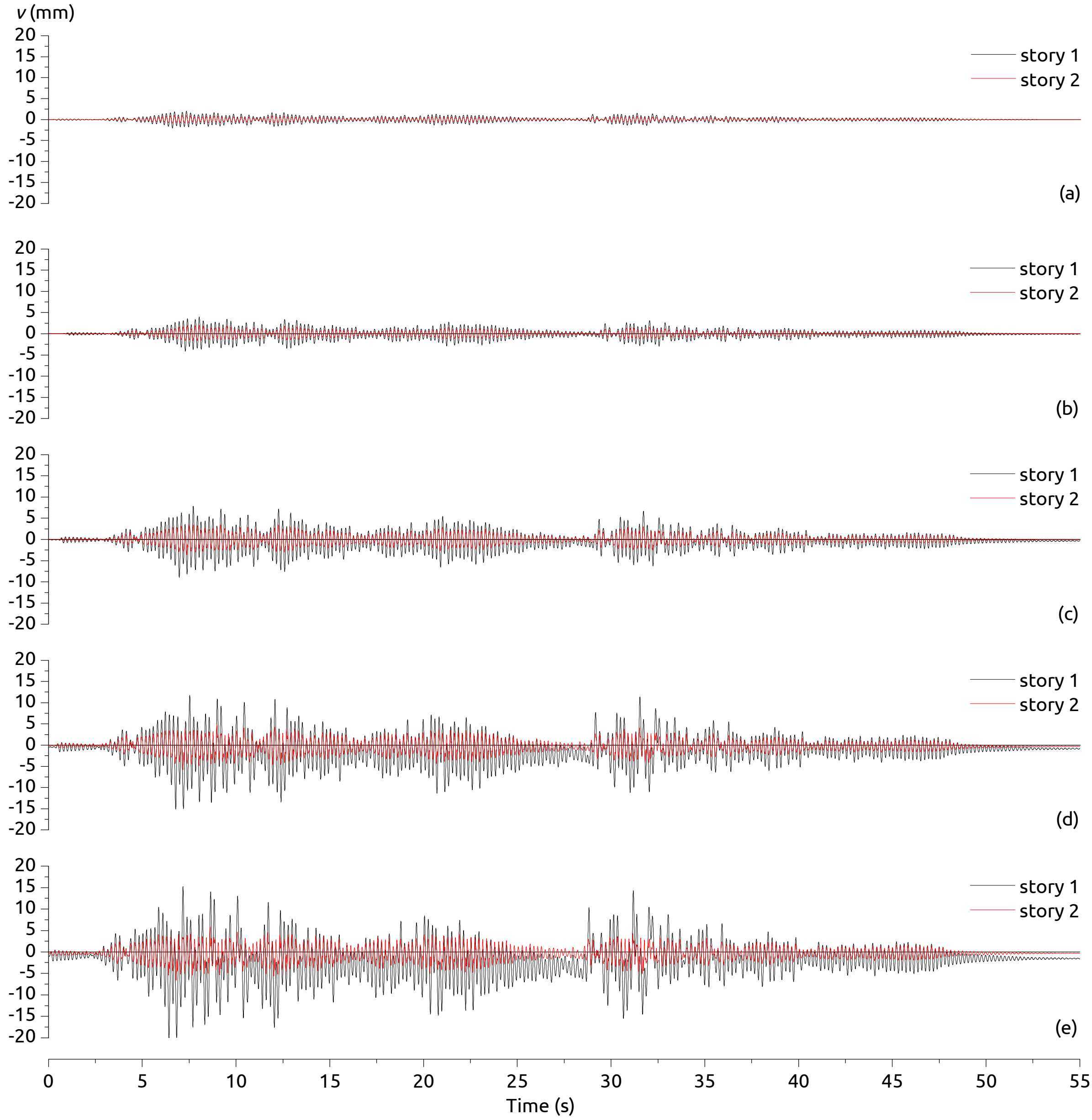

Figure 7. Displacement time histories (a) c50 test (b) c100 test (c) c200 test (d) c300 test (e) c350 test.

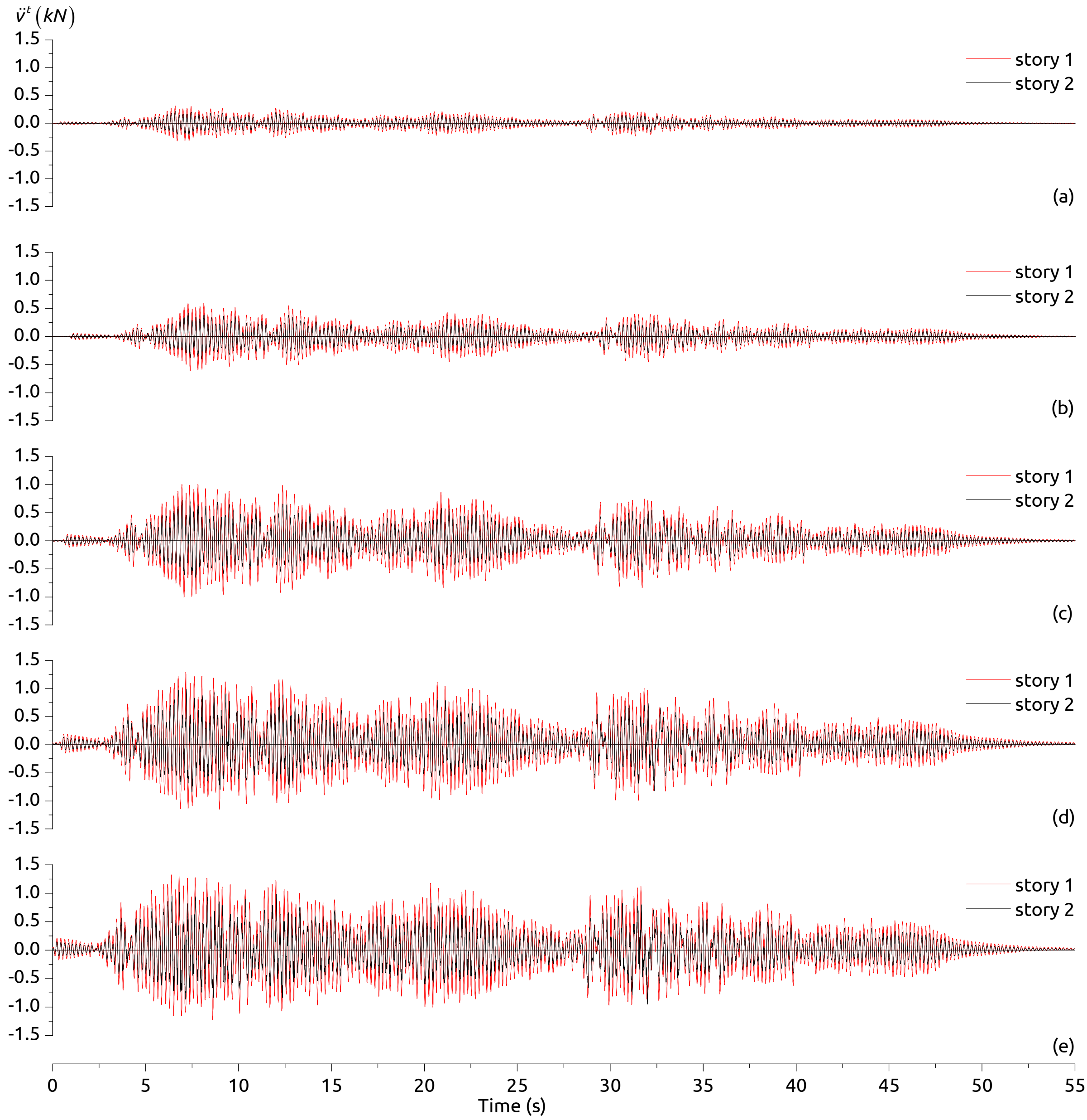

Figure 8. Acceleration time histories (a) c50 test (b) c100 test (c) c200 test (d) c300 test (e) c350 test.

Table 4 summarizes the maximum values of the relevant response parameters measured during the tests. In Table 4 T is the fundamental period of the structure, ξ is the damping ratio, $\ddot{v}^t_{\max}$ is the maximum absolute response acceleration, $v_{\max}$ is the maximum relative displacement and ID is the maximum inter-story drift. The inter-story drift can be used as a simple damage index for the RC frame. Comparing the experimental IDs recorded during the tests to the limiting values currently available in the literature, the structural performance level SPL associated with each seismic simulation can be determined as follows. According to FEMA-356 (Federal Emergency Management Agency, 2000) and ATC-40 (Applied Technology Council, 1996) during the seismic simulations c50 to c300 the structure exhibited an Immediate Occupancy SPL (ID<1%) while during the test c350 the structure exhibited a Life Safety SPL (1%<ID<2%). According to (Elnashai & Di Sarno, 2008) during the seismic simulations c50-c100 the structure exhibited a Immediate Occupancy SPL (0.2%<ID<0.5%) while during the seismic simulations c200 to c350 the structure exhibited a Life Safety SPL. Comparing the expected SPL in Table 3 with the SPL observed during the test, it can be concluded that with the inclusion of hysteretic damper all the SPL expected in design (Table 3) were fully satisfied, and even enhanced.

Table 4. Overall response parameters

		Dynamic characterization		Story 1			Story 2		
Test	PGA	T	ξ	$\ddot{v}^t_{\max}$	$v_{\max}$	ID	$\ddot{v}^t_{\max}$	$v_{\max}$	ID
	g	s	%	g	mm	%	g	mm	%
c50	0.08	0.197	2.6	0.13	2.05	0,14	0.31	0.96	0,16
c100	0.15	0.195	2.6	0.27	4.10	0,29	0.56	1.97	0,33
c200	0.31	0.205	2.6	0.52	8.93	0,64	0.83	3.49	0,58
c300	0.47	0.205	2.6	0.71	15.10	1,07	1.1	5.77	0,96
c350	0.54	0.205	2.6	0.81	20.0	1,43	1.1	6.62	1,1

6. CONCLUSIONS

This paper presented the results of shaking table test conducted on a RC frame structure with hysteretic dampers at the Laboratory of Structures of the University of Granada (Spain). The frame structure was designed to resist only gravity loads, leaving to the dampers the responsibility of the whole earthquake resistance. The main findings of this experimental study may be summarized as follows:

(1) The inclusion of the energy dissipation devices in the RC frame structure provided the necessary lateral strength to resist the maximum design earthquake.
(2) The inclusion of hysteretic dampers satisfied and even enhanced, the expected SPL considered in design for all the seismic hazard levels considered.

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