A Shake table test of typical Mediterranean reinforced concrete structures

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SUMMARY:
This research investigates the ultimate earthquake resistance of typical RC moment resisting frames designed accordingly to current standards, in terms of ultimate energy absorption/dissipation capacity. Shake table test of a 2/5 scale model, under several intensities of ground motion, are carried out. The loading effect of the earthquake is expressed as the total energy that the quake inputs to the structure, and the seismic resistance is interpreted as the amount of energy that the structure dissipates in terms of cumulative inelastic strain energy.

Keywords: shake table tests, reinforced concrete frame, energy dissipation capacity, seismic assessment

1. INTRODUCTION

During the last two decades, a great number of buildings have been constructed in low-to-medium seismic areas of Spain following the provisions of the recent Spanish codes Ministerio de Fomento (2002) and Ministerio de Obras Públicas Transportes y Medio Ambiente (1994). One of the most economic and common technologies has been moment resisting RC frames, which have especially been used for buildings with short spans, as in the case of housing. The Spanish code not only provides guidelines for the determination of lateral seismic forces and for the estimation of torsional effects but also offers general prescriptions for the building and the necessary structural requirements that are necessary in order to achieve a certain level of ductility and hence lateral force reduction factors. Finally, although the code requires that RC structures collapse under a weak beam-strong column pattern, no criterion is given in order to fulfill this requirement, unlike other codes. Recent seismic events and the latests advances in earthquake engineering have cast doubts on the performance of these buildings, which haven't been experimentally tested to date. For instance, the structural behavior in case that the expected seismic hazard is exceeded or the economic impact of low-intensity ground motions might produce is yet to be determined.
The purposes of this ongoing investigation are (i) to assess the performance of this type of structures under increasing levels of seismic excitation (ii) to evaluate the adequacy of code criteria regarding today's standards (iii) to determine the ultimate energy absorption capacity of these structures and (iv) to determine the lateral collapse pattern of the structure.

2. PROTOTYPE STRUCTURE

The considered prototype is a three-story 3x3 bay moment resisting RC frame shown in fig. 1. The slab is constructed with one-way joists carried by rectangular beams which depth, *h*, is larger than the width *b*. In figure 1a) the direction of the joists is indicated with arrows. The structure is designed to withstand both gravity loads (dead loads: floors 3.22kN/m^2; roof 2.95kN/m^2 and live loads: floors 2kN/m^2; roof 1kN/m^2) and lateral loading within the scope of European Committee for Standardization (2004) and the current Spanish seismic code for a moderate seismic area such as Granada (Soil Class C; v_s<200m/s; base acceleration α=0.23g (here g is the acceleration of gravity); behaviour factor q=3.0).

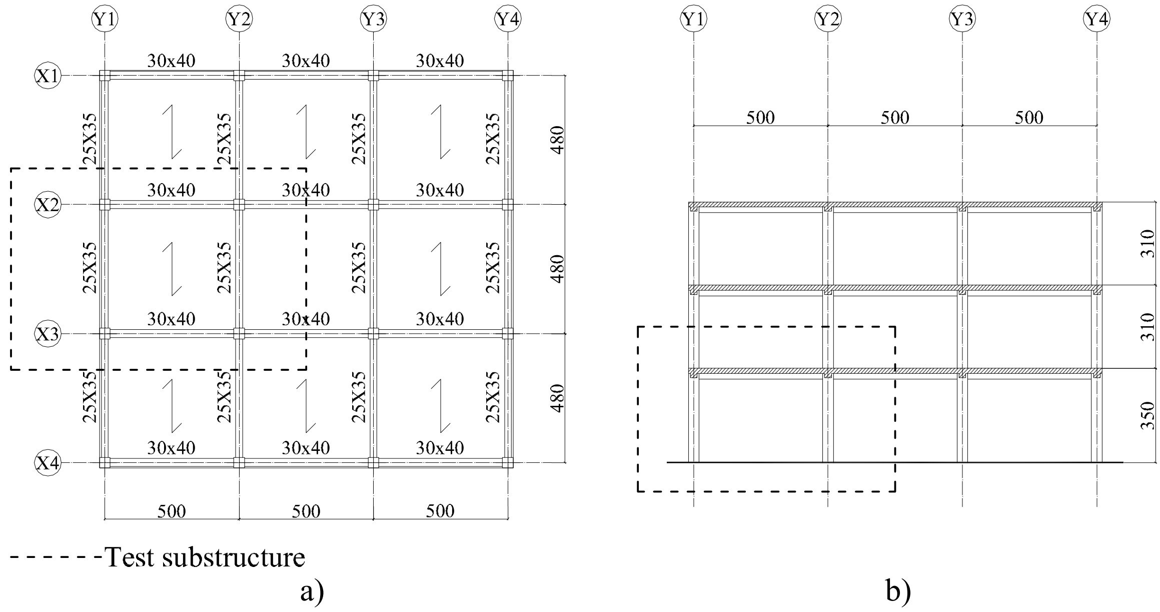

Figure 1. Prototype: a) plan; b) elevation

The concrete compressive strength assumed in calculations is 25MPa and the yield strength for steel is 500MPa. This prototype is representative of buildings designed in low to medium seismic areas in Spain in the latest decades (from 1994 until today), as it follows the provisions of the most recent Spanish codes Ministerio de Fomento (2002) and the previous code Ministerio de Obras Públicas Transportes y Medio Ambiente (1994). As a result of the application of the limit state design method, the following structural dimensions are obtained: RC columns 40x40cm, beams supporting the joists (*b*x*h*=) 30x40cm, perpendicular beams (*b*x*h*=) 25x35cm. The columns were then designed so that, under lateral loads, the frame develops a weak beam-strong column collapse mechanism as shown in fig. 2. For this purpose, an over-strength factor of 1.25 was adopted on the ultimate flexural moments of beams M_{ub} to columns M_{uc}, i.e. $1.25=\Sigma M_{uc}/\Sigma M_{ub}$. As a consequence of this capacity design, the detailing of RC sections shown in fig. 3 was obtained.

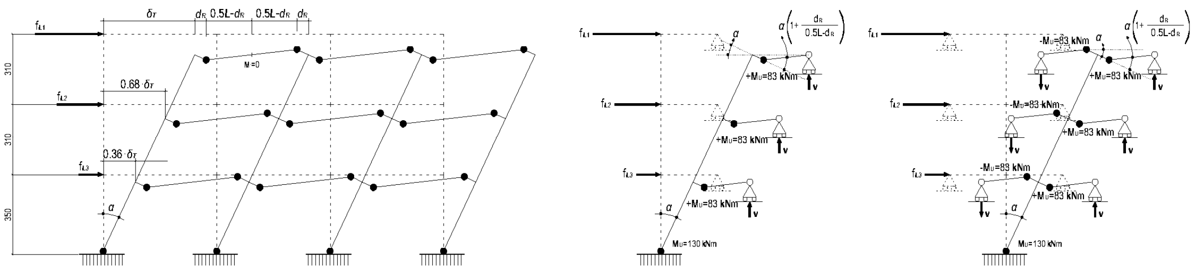

Figure 2. Capacity design

3. TEST SPECIMEN

Test specimen is a 2/5 scale substructure extracted from the prototype in fig.1. The connections to the rest of the structure are simulated by using pin joints at column mid-height and beam mid-span, corresponding to the inflection points of flexure under lateral loading. Gravity loads are simulated with added steel blocks. Scale factors for stress λ_σ and acceleration λ_a were unity, while length scale factor λ_L was 2/5. Scale factors in table 3.1 were applied so as to satisfy similitude requirements for dynamic loading Harris and Sabnis (1999).

Table 3.1. Scale 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	$\lambda_S=(\lambda_L)^2$	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	$\lambda\varepsilon$	L/L	1

The specimen was built in the Laboratory of Dynamics of Structures of the University of Granada. The test structure was constructed in 4 stages (foundation, first story columns, slab, and 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. For each batch of concrete, compressive strength tests were realized on the 28th day and the first test day. Summarized results from average material properties are displayed in table 3.2.

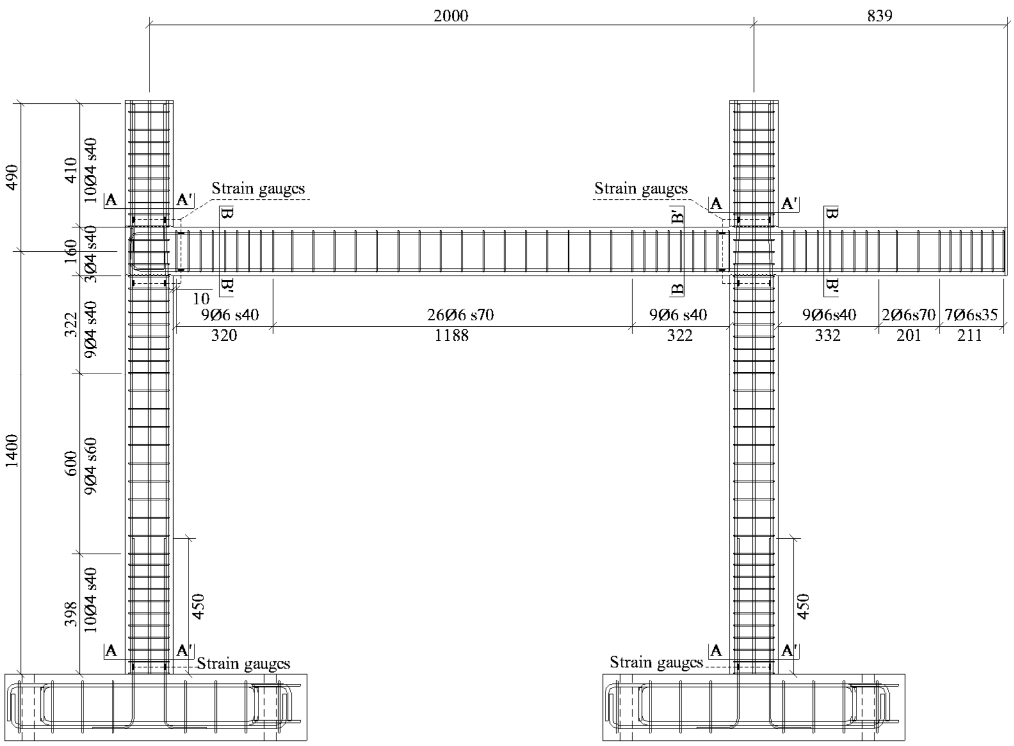

Figure 3. Specimen details

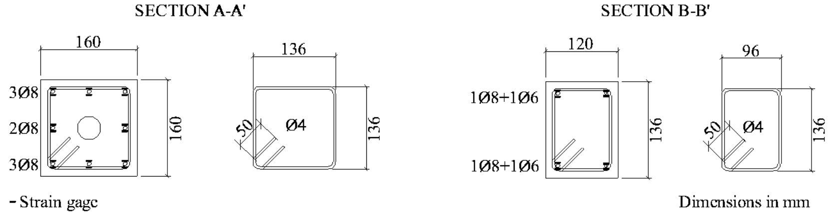

Figure 4. Section details

Table 3.2. Average material properties

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

4. TEST CONFIGURATION

The specimen was subjected to several uniaxial dynamic tests, referred to hereafter as "seismic simulations", with the shaking table of the Laboratory of Dynamics of Structures of the University of Granada. Before the seismic simulations, several low intensity white-noise and ground motion tests were conducted for calibration of the shaking table. In addition, free vibration tests were performed before and after each seismic simulation so as to obtain modal properties of the specimen.

4.1. Instrumentation and measurement

The instrumentation used in these tests included 192 strain gauges, 10 uniaxial accelerometers and 9 displacement transducers. Strain gages were glued on the longitudinal reinforcement at sections close to the top and bottom of the columns, and also at beam sections close to the beam-to-column joints. Two strain gauges were mounted on each rebar. Uniaxial accelerometers were placed on each story in both the actuator and the perpendicular directions and also on the shaking table. The story drifts and out of plane deformations were measured with linear variable differential transformers (LVDT) connected to an external reference frame fixed to the shaking table. Relative displacements of the slabs were obtained from measured story drifts. Each one of the frames is instrumented individually to account for torsional effects. Scan frequency in the data acquisition system was set to 200Hz. Figure 5 shows the set-up and instrumentation.

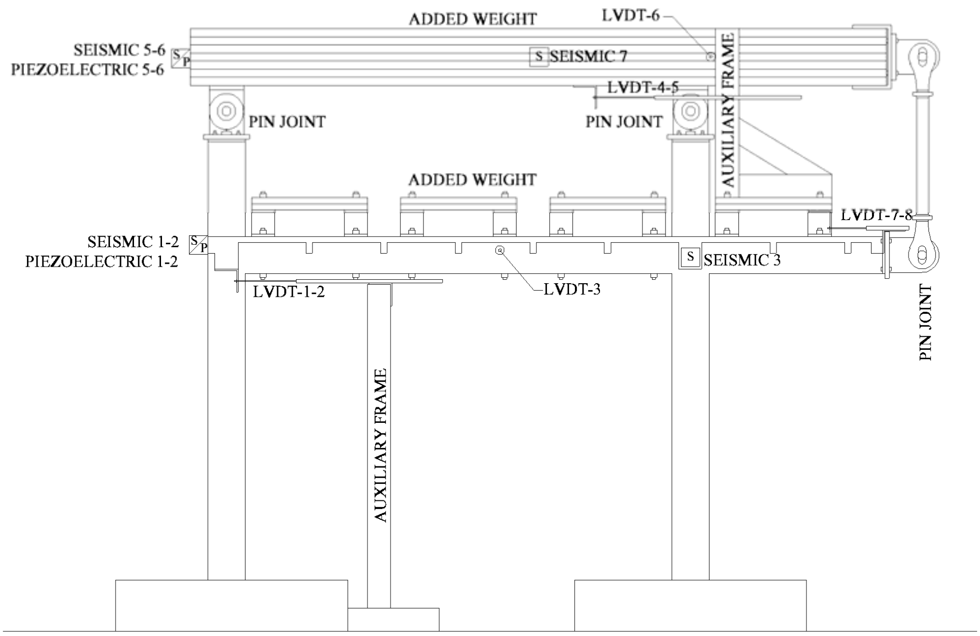

Figure 5. Experimental setup

4.2. Seismic simulations

Main tests consisted in four seismic simulations, referred to as C50, C100, C200 and C300, in which the original ground acceleration recorded at Calitri (Italy) during the Campano Lucano earthquake (1980) (Ambrasseys et al. 2001) was scaled to 50%, 100%, 200% and 300% of the original record. The corresponding peak ground accelerations (PGA) are shown in Table 4.1. These amplitudes represent different hazard levels at the building site (Granada, Spain). Fig. 6 shows the history of acceleration of the original ground motion. Table 4.1 shows also the expected structural performance level (SPL) and the ranges of return period of the quake in the site (Granada, Spain) according to the Spanish seismic code by Ministerio de Fomento (2007) where a modification factor of the expected PGA at the building site can be calculated as a function of the return period.

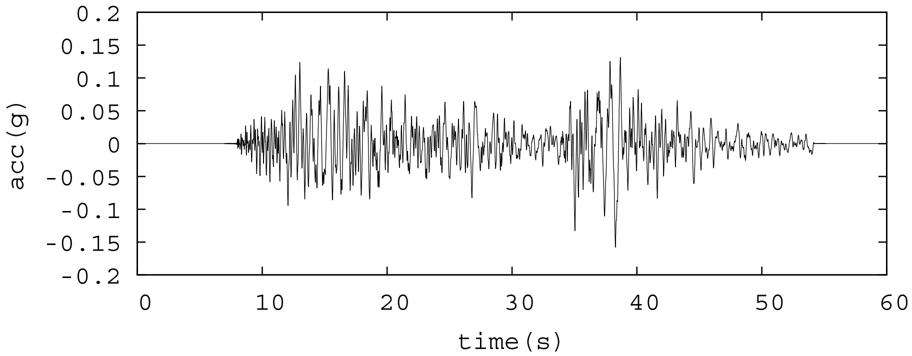

Figure 6. Applied ground motion

Table 4.1. Seismic simulations

Test name	PGA	Return period	Expected SPL
C50	0.08g	59 years	Immediate occupancy (IO)
C100	0.15g	81 years	Life Safety (LS)
C200	0.31g	500 years	Collapse prevention (CP)
C300	0.47g	1428 years	Total collapse (TC)

5. SEISMIC PERFORMANCE

Seismic simulation C50 represents a low design-level earthquake for the building site. Under this seismic simulation no visible damage was observed on the specimen. All rebars remained elastic and no remanent deformation could be detected. The lateral displacements of the two parallel frames were not exactly the same due to torsional effects. Such torsional behavior was observed in all tests.

Seismic simulation C100 corresponds to a moderate level earthquake at the building site. Yielding of the rebars at the base of all columns and at a beam in the exterior joint occurred. Minor cracks developed at the beam ends and slab.

Seismic simulation C200, which represents a strong quake at the building site, almost produced the collapse of the specimen. Deformations at rebars reached over 18000 μm/m and severe cracking was observed at the base of the columns, beam joints and the slab.

During the seismic simulation C300, the specimen totally collapsed, with lateral displacements larger than 7% of story height.

5.1. Structural response

Figure 7 shows the condition of the structure after the seismic simulation C200. Maximum response acceleration AC, maximum lateral displacement DISP and interstory drift ratios ID are summarized in table 5.1. Figures 8 and 9 show the histories of absolute response acceleration and relative displacement (in terms of interstory drift ratio) of the slab, for each seismic simulation.

Figure 7. Experimental setup

Table 5.1. Structural response parameters

	FIRST STORY				SECOND STORY			
	AC	DISP	ROT	ID	AC	DISP	ROT	ID
Units	g	mm	deg	%	g	mm	deg	%
C50	0.13	3.47	0.053	0.24	0.18	1.19	0.017	0.22
C100	0.28	6.78	0.085	0.5	0.33	2.42	0.032	0.44
C200	0.40	16.71	0.149	1.19	0.47	5.67	0.073	1.05

The values of ID are compared with the reference values proposed by different standards and authors in table 5.2. It can be concluded that the limiting values of ID from ATC-40 (1996) and FEMA 356 (2000) would overestimate the lateral capacity of the structure while values from Fardis (2009) are similar to the experimental results.

Table 5.2. Displacement based performance levels

	ID	Fardis	ATC-40	FEMA	SPL
Test	%	%	%	%	
C50	0.24-0.22	0.2<ID<0.5	1	1	Immediate occupancy
C100	0.5-0.44	0.5<ID<1.5	1-2	2	Life safety
C200	1.19-1.05	1.5<ID<3.0	2	4	Collapse prevention

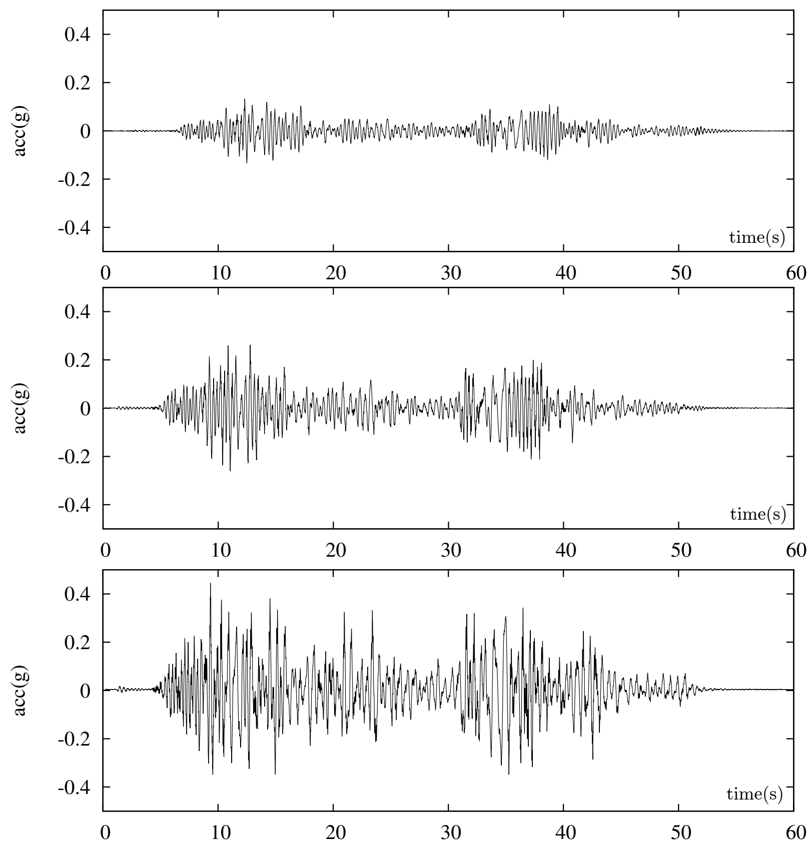

Figure 8. Response acceleration during tests C50, C100 and C200.

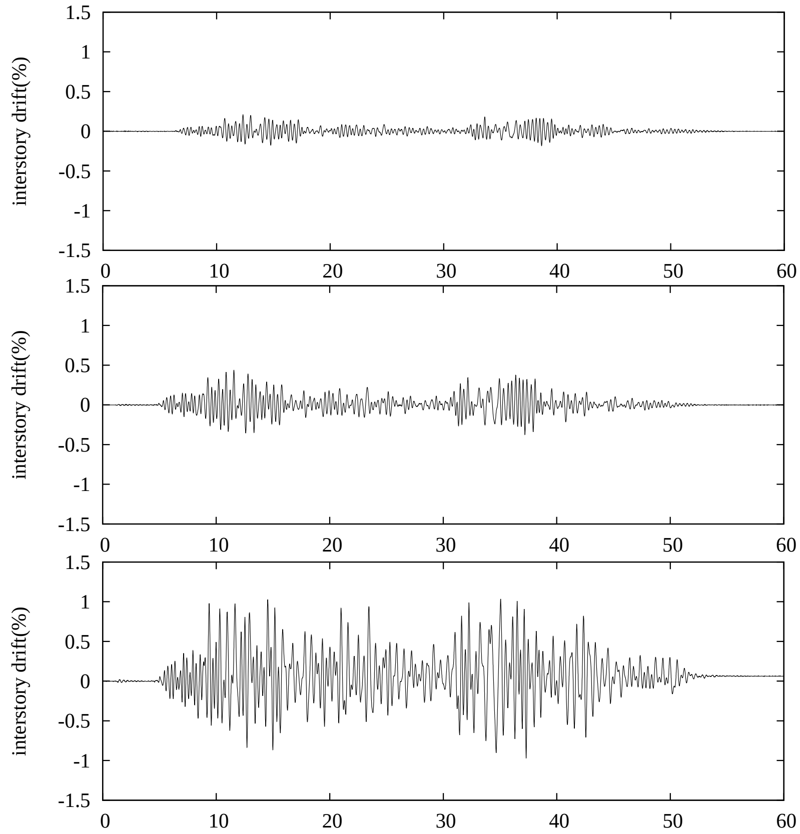

Figure 9. Interstory drift ratio during tests C50, C100 and C200

5.2. Assesment of input energy

Energy balance equation 5.1 states that input seismic energy E_I must be absorbed by means of plastic strain energy W_p, elastic strain energy W_{el}, kinetic energy W_k, and the damping energy W_ξ dissipated by the damping mechanism of the system. It has been shown by Akiyama (1985) that input energy is a very stable quantity that depends on the total mass of the structure M and the fundamental period T_1.

$$E_I = W_\xi + W_p + W_{el} + W_k \tag{5.1}$$

Following the original formulae by Uang and Bertero (1990) relative input energy can be calculated by integrating the terms for all the degrees of freedom as in eq 5.2.

$$E_I = \int \sum_{j=1}^{NDOF} m_j \, \ddot{u}_g \, \mathrm{d}u_j \tag{5.2}$$

Input energy during the seismic tests is summarized in table 5.3. For convenience, input energy is expressed in terms of equivalent velocity $V_E=\sqrt{(2E_I/M)}$.

Table 5.3. Seismic input energy

Test	C50	C100	200
V_E (cm/s)	25.6	55.8	124.9

6. CONCLUSIONS

A series of shake table tests (seismic simulations) have been conducted on a 2/5 scale specimen representing a typical Mediterranean reinforced concrete frame structure designed according to current seismic codes, in order to assess its performance and to determine its energy absorption capacity and collapse pattern. Structural performance proved to be adequate regarding the criteria of the Spanish Code, which aim is to prevent collapse in case of a major earthquake at the building site. The structure had no or minor damage after the seismic simulation C50 (associated with a return period of 59 years). Severe damage and deterioration of structural stiffness was already noticed during the seismic simulation C100 (associated with a return period of 81 years). Under the seismic simulation C200 (associated with a return period of 500 years) the structure was very close to collapse. The specimen developed a strong column-weak beam plastic mechanism as the current code requires, and damage concentrated at the bases of columns and at ends of beams.

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