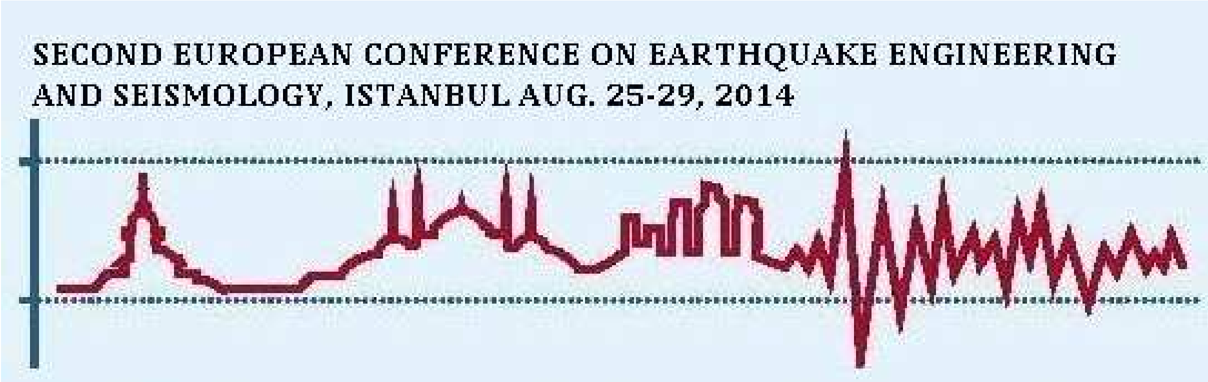

SEISMIC PERFORMANCE OF COMMON REINFORCED CONCRETE MOMENT RESISTING FRAME STRUCTURES

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ABSTRACT

This paper investigates the seismic performance of reinforced concrete (RC) moment resisting frame structures designed according to current codes and located in medium seismicity regions. To this end, a series of experimental and analytical studies are conducted on representative prototypes. First, several prototype buildings are designed and from one of them, a portion of the structure with one bay and a half, and one story and a half is selected. By applying scaling factors of 2/5, 1 and 1, for geometry, stress and acceleration, respectively, a test structure is defined. The test structure is constructed and tested with the 3x3m^2 shaking table of the Laboratory of Dynamics at the University of Granada. It is subjected to several seismic simulations of increasing intensity up to collapse. Results from these tests are used to validate/modify empirical equations proposed in the literature for predicting relevant aspects of the capacity of this type of structures such as the ultimate rotation of the plastic hinges. Second, several numerical models are developed and calibrated with the results from shake table tests. Third, nonlinear time history analysis are performed to evaluate the system seismic response. Finally, a probabilistic methodology is used that takes into account inherent uncertainties in the ground motion, computer models, and damage estimates. As a result of this research a quantitative and qualitative basis is provided for the assessment of the seismic performance under the design earthquake of a good deal of recent low-rise buildings constructed in Mediterranean countries in the last decade. Results serve to compute damage and loss analysis in the building components.

INTRODUCTION

The purpose of this paper is to assess the seismic structural performance of representative prototypes of RC moment resisting frame structures designed following Spanish current seismic code under the action of the design earthquake (DE). Ten years after the introduction of current strength-based seismic code NCSE-02 by Ministerio de Fomento (2002), the number of homes in Spain has increased by 5 millions. However this code applies only to housing buildings in locations where the peak ground acceleration (PGA) of the DE is more than 0.08g. The increase in the number of homes in medium seismicity areas (e.g. expected PGA>0.16g) is unknown by the authors but it must be important as the Spanish Statistical Office reports average growth of 23.1-45.4% (Alacant, Málaga, Murcia) and 16.9-20.4% (Granada) during that period. The use of RC frame structures is widespread in low and medium rise housing buildings as other typologies are penalized with lower ductility performance factors in the code. Common design practice in RC frame structures is to adopt a value of the ductility performance factor μ=3. The application of this factor means that the lateral seismic forces to be resisted by the

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structural system can be divided by μ=3 on the grounds that the design meets some requirements regarding minimum dimensions, reinforcement and detailing. It is understood that the meeting of these requirements leads to a lateral collapse pattern of the strong column-weak beam type and to collapse prevention in the event of the design earthquake. It is also implicit in the adoption of a ductility factor above μ=1 that structural damage will occur under the design earthquake but no guidance is given, as the main purpose of seismic codes from last decade (also known as first generation codes) was to reduce casualties by preventing a collapse. Little notice is given as well to the damage to nonstructural components as they are out of scope. The seismic performance of sub-assemblies of these specific concrete moment resisting frames is discussed after shake table tests by Benavent-Climent et al. (2013). Empirical results suggest that minimum design provisions are adequate for collapse prevention.

Earthquakes do not only pose a threat to human lives but also raise concern over their economic and social impact. As a consequence the tendency in seismic design is changing now towards the paradigm of Performance Based Design and Assessment. FEMA (2012) states that "these approaches explicitly evaluate how a building is likely to perform, given the potential hazard it is likely to experience, considering uncertainties inherent in the quantification of potential hazard and uncertainties in assessment of the actual building response. To complete a performance assessment, statistical relationships between earthquake hazard, building response, damage, and then loss are required. In a general sense, the process involves the formation of four types of probability functions, respectively termed: hazard functions, response functions, damage functions, and loss functions, and manipulating these functions to assess probable losses". This paper is limited to facilitating response functions under the DE.

Response is addressed by analysis of shake table tests and numerical models representing regular concrete moment resisting frame structures in housing buildings. Non-linear time history analysis are performed with eight historical ground motion accelerograms that are selected and scaled to match the expected hazard level produced by the DE. Two demand parameters are extracted that characterise damage to structural and non-structural components in the prototype building: chord rotation demand in plastic hinges and inter-storey drift. Statistical functions are fitted to the distribution of demand parameters and compared to well-known operational ranges used in the assessment of concrete moment resisting frames.

PROTOTYPE STRUCTURES

Two regular concrete moment resisting frame structures (3 and 6 stories) are taken as prototypes of typical housing buildings located in medium seismicity regions in Spain. These structures are designed using commercial software Arktec Tricalc to meet the design requirements in current Spanish codes. Gravity loads in Table.1 are prescribed by Ministerio de Vivienda (2006) and are the usual in housing buildings. The seismic hazard of the DE is characterized by the normalized response spectrum in Fig.1 considering soft soil and PGA=0.23g. The intensity of this DE corresponds to a return period of 500 years which is a probability of exceedance of 10% in 50 years. Ductility performance factor is set to μ=3. Structural elements are designed by limit state analysis, and the effects of the design ground motion on the structure are determined through modal analysis. The failure mechanism of the frames was determined to be of the weak beam-strong column type by a capacity design approach.

Table 1. Loading of the prototypes

Live loads		Dead loads	
Housing	2.0 kN/m2	Concrete	24.5 kN/m3
Roof	1.0 kN/m2	Concrete slab	2.15 kN/m2
		Partitions	1.0 kN/m2
		Pavement	1.0 kN/m2
		Roof	3.0 kN/m2
		Walls	7.0 kN/m

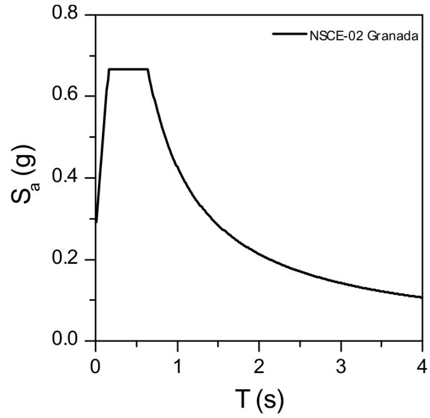

Figure 1. Acceleration response spectra

Prototype 1 is a 3×3 spans and 3 storeys concrete moment resisting structure with a total height of 9.7m. The dimensions of the typical floor are 15 × 14.4m. Prototype 2 has 3 × 4 spans and 6 storeys with a total height of 19m. The dimensions of the typical floor are 20 × 14.4m. Storey height is 3.5m for ground floor and 3.1m between other floors. An elevation of representative frames of the prototypes are shown in Fig.2. In prototype frame 1, the dimensions of interior columns are 0.40×0.40m while exterior columns are 0.30×0.30m; all beams are 0.30×0.40m. In prototype frame 2, the dimensions of all columns in the first 3 stories are 0.40×0.40m, columns in stories 4 to 6 are 0.30×0.30m, and all beams are either 0.30×0.40m or 0.40×0.40m. Table.2 summarizes the sizes and reinforcement of RC sections as numbered in Fig.2.

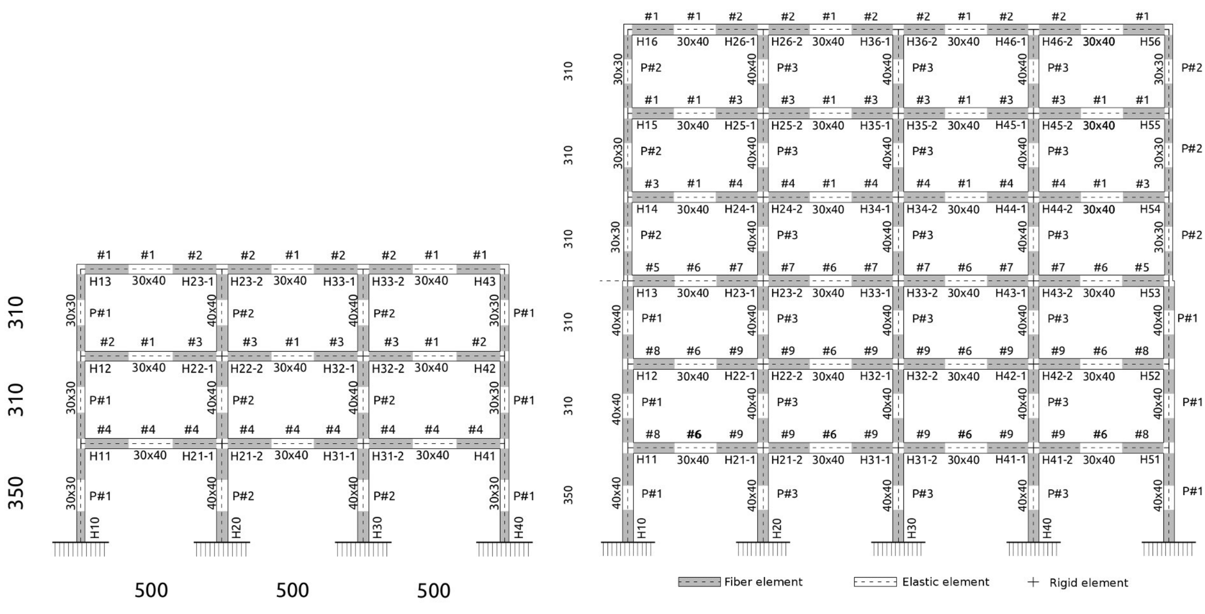

Figure 2. Schematic of prototype frames 1 and 2

Table 2. Description of the prototype members sizes and reinforcement

	Frame 1 (3 storey)						Frame 2 (6 storey)											
Section number	#1	#2	#3	#4	P#1	P#2	#1	#2	#3	#4	#5	#6	#7	#8	#9	P#1	P#2	P#3
Width (cm)	30	30	30	30	30	40	30	30	30	30	40	40	40	40	40	40	30	40
Height (cm)	40	40	40	40	30	40	40	40	40	40	40	40	40	40	40	40	30	40
Top rebars	4	4+1	4+2	4+2	5	6	2	3	2+1	2+2	3	2	4	2+2	2+3	5	5	6
Diameter (mm)	12	12,16	12,16	12,20	16	16	20	20	20,16	20,16	20	20	20	20,16	20,16	20	16	20
Bottom rebars	5	5	5	5	5	6	2	2	2	2	3	3	3	3	3	5	5	6
Diameter (mm)	12	12	12	12	16	16	20	20	20	20	20	20	20	20	20	20	16	20

EMPIRICAL STUDY: SHAKE TABLE TESTS

Shake table tests of a 2/5 scale sub-assembly of a frame prototype similar to those described were realized at the University of Granada as described by Benavent-Climent et al. (2013). Scaling factors are 2/5, 1 and 1, for geometry, stress and acceleration. According to similarity requirements, the fact that the scaling for factor for stress is 1 means that stress in the materials must be the same in the prototype and in the scaled model. Dimensionless quantities as inter-storey drift and chord rotations are also the same in prototypes and scaled model. Four tests are performed by increasing the shaking intensity until collapse. The shaking intensity in these tests corresponds to approximate return periods of 17, 97, 500, and 1435 years at the described location. For the DE with 500 years return period, it is observed that the sub-assembly develops a strong column-weak beam mechanism and that the earthquake has done serious damage: strains of longitudinal reinforcement reach 7 times the yield strain, the maximum chord rotations are 28% of their ultimate capacity, and the inter-storey drift id reaches 1.19% of the story height. An important loss of lateral stiffness causes the fundamental period to lengthen from 0.32s to 0.54s. The seismic performance of the sub-assembly is near the upper limit between life safety (LS) and collapse prevention (CP) performance levels according to SEAOC (1995) guidelines for RC moment resisting frame structures. Significant damage and near collapse are the counterparts of LS and CP in Eurocode 8 by European Committee for Standardization (2006). Visual inspection of the sub-assembly reported extensive cracking and crushing in concrete being related to the development of plastic hinges in the lateral load carrying system.

ANALYTICAL STUDY

Prototype frames 1 and 2 are modelled with non-linear uni-dimensional finite elements in Forum8 Engineer's Studio software to have the dynamic response of both frames reproduced by non-linear time history analysis. In modelling the frames, structural members are idealised as fibre elements and each fibre element is divided into a 30x30 grid of cells. The RC constitutive model used in this study is one developed by the Concrete Laboratory at University of Tokyo and is implemented in the software. Concrete and rebar's nominal strength are 25MPa and 500MPa. The centre portion of beams and columns (white areas in Fig.2) are assumed to remain elastic, unlike the grey areas, where the mesh of fibre elements is more dense. Shear deformation of the panel is neglected by assuming that beam-column connections are rigid (crosses in Fig.2).

The lateral capacity of the prototypes is characterised by a force controlled pushover analysis which results (top displacement *d* vs base shear *Q*) are shown in Fig.3. The dashed lines in these figures are idealizations of the capacity curves according to FEMA (2000). Initial fundamental periods of prototype frames 1 and 2 are 0.54s and 0.97s as estimated by elastic modal analysis.

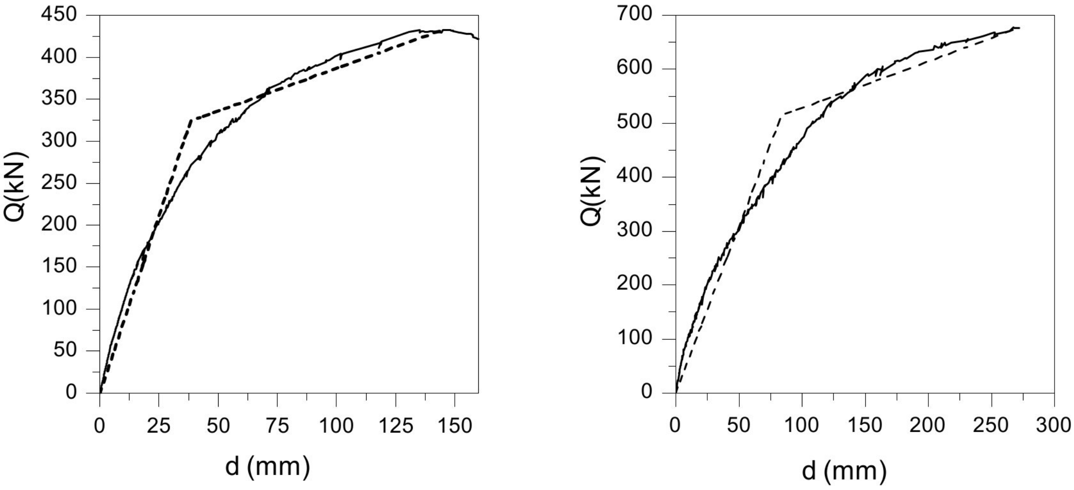

Figure 3. Lateral force displacement Q-d capacity curves of frames 1 and 2

To study the dynamic response of the idealised frames, eight historical ground motions are selected and modified to match the expected seismic hazard level for the DE. Modification of the ground motion consisted in scaling the accelerograms in amplitude so that the total energy input in the structure, normalized by the total mass of the building M and expressed in terms of an equivalent velocity $V_E=(2E/M)^{0.5}$ was the same in every analysis. Total energy input corresponding to the DE is V_E=112cm/s as estimated by Benavent et al. (2002). Use of energy input in terms of equivalent velocity V_E as an intensity measure for seismic hazard is supported by early studies by Matsumura (1992) and Akiyama (1985). Table.3 summarizes the ground motion accelerograms used and the scaling factor needed to tally the design earthquake's input energy. Further details on these analyses and accelerograms can be found in Escolano-Margarit (2013). These results are used to populate a database of engineering demand parameters (edp) for assessment.

Table 3. Scaling factors used in nonlinear time history analysis

Ground motion	Frame 1	Frame 2
Alkion (Korinthos)	1.63	1.45
Friuli (Tolmezzo)	2.40	3.00
El centro	1.27	1.30
Kobe	0.39	0.53
Montenegro (Petrovac)	0.72	0.96
Taft	1.90	2.12
Campano-Lucano (Calitri)	1.05	1.16
Montebello (Northridge)	3.65	4.27

PERFORMANCE BASED SEISMIC ASSESSMENT

Performance of the prototypes is evaluated following the Performance Based Earthquake Engineering framework. The implementation of this framework is decomposed into four steps as described by Moehle and Deierlein (2004). First step is to describe the seismic hazard for the structure. This study focuses on a 500 years return period at the building site and uses total energy input as intensity measure (im). The probability (annual rate λ) that the DE is exceeded is λ (V_E>112cm/s)=0.002.

The second step in the framework is response analysis for the given intensity measure. Results from the nonlinear time history analysis discussed in the preceding section are used to characterize the response of structural and nonstructural components in the prototype buildings. The outcomes of response analysis are statistical functions that relate demand parameters to the seismic hazard expected by the structure. Two demand parameters (edp) are selected to characterize damage to structural and nonstructural components. The first edp is chord rotation demand in the frame plastic hinges. Chord rotation demand is defined as the ratio of peak chord rotation to the ultimate chord rotation θ_m/θ_u experienced by the frames plastic hinges following formulate in Eurocode 8 and is useful in assessing damage to the RC frame. Limiting value of the chord rotation demand between No Damage (ND) and Damage Limitation (DL) performance levels according to Eurocode 8 is calculated to be around 0.15. This limiting value is about 0.75 for significant damage (SD). Hinges with negligible values of the ratio (e.g. hinges not involved in the weak beam-strong column lateral collapse mechanism) are excluded from the calculation. The second edp is the peak inter-storey drift id (as % of storey height). Inter-storey drift is useful in assessing damage to partitions and all drift-sensitive components. Limiting values of the inter-story drift between performance levels suggested by SEAOC (1995) are: 0.50% between immediate occupancy (IO) and life safety (LS), and 1.50% between LS and near collapse (NC).

Fig.4 shows the cumulative distribution of the prototypes' edp. Circles and squares correspond to results from time history analysis of frames 1 and 2. These data is fitted with lognormal distribution and plotted with lines. The probability that a certain value x of an edp is not exceeded by the prototype response is P(x<edp) and can be estimated with Eq.1. The probability that a certain value x of an edp is exceeded is P(x>edp)=1- P(x<edp).

$$P(x < edp) = \phi\left(\frac{\ln x - \mu(\ln edp)}{\sigma(\ln edp)}\right) \qquad (1)$$

where ϕ is the cumulative distribution function of the standard normal distribution, μ(ln edp) is the mean of the logarithms of the edp, and σ(ln edp) is the standard deviation of the logarithms of the edp. Table4 summarizes the values of μ(ln edp) and σ(ln edp) fitting the distribution of edp in Fig.4. The results of the response analyses are shown with symbols and the solid lines represents the fitting lognormal distribution. Results from shake table tests are shown with a vertical dashed line and are in upper medium range of analytical results. On one hand, peak chord rotation demands are higher in frame 2 (up to 0.50) than in frame 1 (up to 0.28). This may suggest a trend for higher chord rotation demands in taller buildings. Considering that the limiting value of the chord rotation demand θ_m/θ_u between performance levels ND and DL is 0.15, the likelihood that the frames enter the DL performance level, $P(\theta_m/\theta_u)>0.15$ is 70% and 85% for frames 1 and 2. On the other hand, the distributions of the displacement demand (inter-storey drift) are remarkably alike. The minimum peak inter-storey drift found in these analyses is 0.60%. About 70% of the peak inter-storey drifts are above 1%.

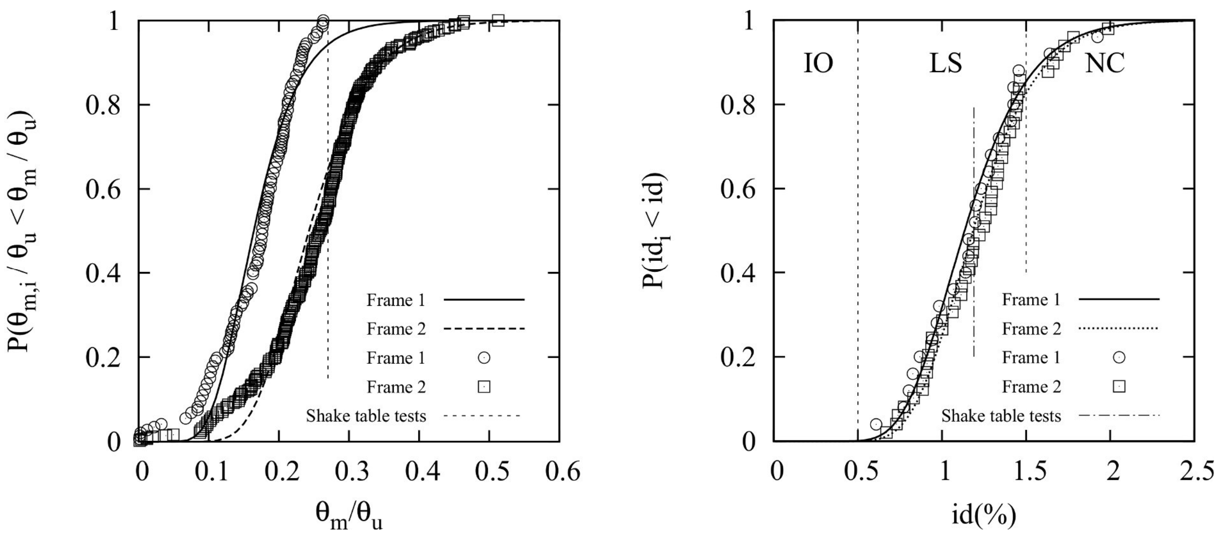

Figure 4: Cumulative distribution and approximate functions of the demand parameters

Table 4: Parameters for statistical fitting of the demand parameters

	Frame 1 θ_m/θ_u	Frame 2 θ_m/θ_u	Frame 1 (%)id	Frame 2 (%)id
μ(ln edp)	1.80-	1.41-	0.13	0.17
σ(ln edp)	0.31	0.28	0.26	0.25

The third step in the framework is damage analysis. Damage to the building components can be characterized in terms of fragility curves showing the probability that a damage state is reached given a quantitative measure of any edp. Looking at Fig.4 it’s observed that the strongest likelihood is that few buildings with a structure of these characteristics remain in immediate occupancy (IO) or no damage (ND) performance levels after the DE. According to SEAOC (1995), performance levels are LS in 85% of the analyzed cases and NC in the remaining 15%. This seems to suggest that life safety requirements in code NCSE-02 are met but damage is extensive to structural and nonstructural components. Structural performance of the concrete frames can be classified as DL by Eurocode 8 formula. No plastic hinges are found that reach significant damage (SD) but at least 70% of the analyzed cases show chord rotation demands above 0.15 that may involve deterioration and require repairing. A more accurate damage analysis requires better knowledge of the buildings components, but it’s likely that an average inter-storey drift of 1.20% implies serious damage to most drift-sensitive components such as partition walls, doors, glazing, piping, etc. The likelihood of damage to drift-sensitive components is similar in the 3 storey and 6 storey prototype frames.

The last step is loss analysis. Loss analysis is a translation of the damage analysis into decision variables (mainly economic) such as the probability of exceeding a certain threshold repair cost in a given time span. This requires estimation of the repair costs and is beyond the scope of this paper. It seems clear that occupation and activity are likely to be interrupted in a good deal of buildings either for replacement works or technical judgment on the building's health.

CONCLUSIONS

The purpose of this paper is to assess the seismic performance of reinforced concrete moment resisting frames built in medium seismicity areas following current Spanish seismic code NCSE-02. Empirical and analytics studies are used to evaluate the seismic performance of two prototype frames corresponding to common housing buildings under the design earthquake. FEMA-P58-1 and PEER probabilistic framework is used as a basis for the assessment. Performance assessment is divided into four steps: hazard analysis, response analysis, damage analysis and loss analysis. A response analysis is carried out that provides a mathematical expression of the conditional probability of exceeding some demand parameters, given that the design earthquake is experienced.

Response analysis is based on the results of shake table tests and nonlinear time history analysis of representative models. Shake table tests of a 2/5 scale of a sub-assembly of a frame similar to the prototypes are realized at the University of Granada. Evaluation of the sub-assembly after the design earthquake shows an important drop in lateral stiffness, extensive concrete cracking and crushing and the development of plastic hinges in a weak beam-strong column pattern. The performance of the sub-assembly is in the upper limit between life safety and collapse prevention levels. Analytical studies discussed in this paper are state-of-the-art nonlinear time history analysis of the idealized prototypes. Eight historical far-field ground motion accelerograms are selected and scaled in amplitude to match the expected seismic hazard level. Two demands parameters are extracted from the results to characterize seismic demand in the prototypes: peak inter-story drift and peak chord rotation demand in RC plastic hinges.

Statistical functions are given that serve to estimate the probability that the demand parameters are exceeded for later probabilistic damage and loss analysis. These statistical functions are obtained from the fitting of analytical response parameters. Results are compared to well-known performance levels in the literature. It is likely that no building with a RC structure of these characteristics remains in immediate occupancy or no damage performance levels after the design earthquake. Expected performance levels are life safety in 85% of the analyzed cases and near collapse in the remaining 15%. This supports that life safety requirements in code NCSE-02 are met but suggests that damage to structural and nonstructural components may be extensive and serious. It is observed that demand parameters from empirical and analytical models are similar so damage in the analytical models must be similar to damage observed in shake table tests.

Results depict a post-earthquake scenario where a great deal of buildings using this technology may experience nonstructural and structural damage that need repairing or replacement. This is relevant for any drift-sensitive components as average peak inter-storey drift is 1.2% of the storey height. As many as 70% of the analysis show chord rotation demands that imply repairing and judgment on the building’s structural health. The interpretation of these results must be careful because of limitations in modeling the response and uncertainties in the level of earthquake hazard. It is unknown if analyses underestimate or overestimate the seismic response of a real building.

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