Seismic vulnerability analysis of wide-beam buildings in Spain

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SUMMARY:
A number of short-to-mid height RC buildings with wide beams have been constructed in moderate-seismicity areas of Spain. The seismic behavior in the direction of the wide beams appears to be deficient because of low lateral strength, low ductility of the wide beams, big strut compressive forces inside the column-beam connections, and unreliable contribution of the spandrel zones of the wide beams. In the orthogonal direction, the behavior is worse since only the joists and the façade beams contribute to the lateral resistance. The objective is to assess the seismic capability of these structures; further research will involve proposing retrofit strategies. The research approach consists of selecting a number of representative buildings and evaluating their vulnerability by code-type, push-over and dynamic analyses. The cooperation of the masonry infill walls is accounted for. The main conclusion is that the seismic behavior of these buildings is inadequate in most of the situations.

Keywords: seismic vulnerability, concrete buildings, wide beams, push-over analysis, Spain

1. INTRODUCTION

A significant number of RC with wide beams are located in areas of medium and moderate seismicity of Spain. Most of those buildings are intended for dwelling and administrative use. These buildings have a concrete framed structure with one-way slabs as primary system; the wide beams constitute the distinctive characteristic, their width is higher than the one of the supporting columns and their depth is equal to the one of the rest of the slab, thus providing a flat lower surface, convenient to ease the construction of the slabs and the layout of the facilities. Figure 1 displays a sketch of a one-way slab with wide beams.

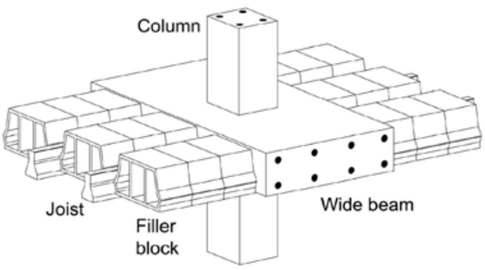


Figure 1. One-way slab with wide beams

This type of floor is also common in other countries in Europe as France and Italy. As well, the wide-beam technology is used in Australia and the US and Canada, where it is also known as banded-floor or slab-band system; in these countries, wide beams are utilized for car park buildings, with span-lengths that are significantly longer than those considered in the European buildings that have wide beams. A common feature to all these wide beams is that the reinforcement amount has to be high (commonly ranging in between 2 and 6%), to compensate the insufficiency of the effective depth.

The seismic behavior of buildings with wide beams appears to be deficient. In the direction of the wide beams, the following weaknesses can be presumed:

- The lateral strength and stiffness of the building are low, mainly because the effective depth of the beams is small (compared to the one of conventional beams).
- The ductility of the wide beams is low since their reinforcement amount is high.
- The strut compressive forces developed inside the column-beam connections are big, due to the low height of the beams.
- Since the beams are wider than the columns, a relevant part of the longitudinal reinforcement of the beams lies out of the vertical projection of the columns (Figure 1). Therefore, the contribution of such spandrel zones of the beams to the bending resistance of the beam-column connections is unreliable because the beams do not have any torsion reinforcement.

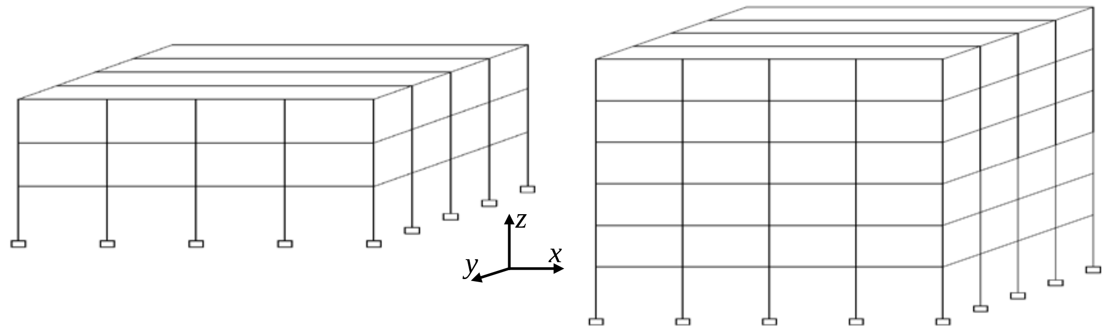
In the orthogonal direction, the lateral seismic behavior might be still worse since the only members of the slabs that contribute to the lateral resistance of the buildings are the joists and the façade beams.

Furthermore, before 1994 these buildings were designed with codes that did not include ductility demands and that required only a low lateral resistance; even, in the regions with moderate seismicity, commonly the seismic action was not taken into account. All these considerations seem to indicate that the seismic capacity of these structures is not reliable. The objective of this research is to assess numerically the seismic capability of wide-beam buildings that are situated in moderate seismicity areas of Spain and that were constructed mainly prior to 1994. The research approach consists of: (i) study of the main features of these buildings and selection of a number of representative edifices and (ii) analysis of the vulnerability of the selected buildings.

A relevant part of this study consists of investigating the hysteretic behavior of the beam-column connections; several researchers have investigated experimentally the seismic performance of connections between wide beams and columns. Some of these studies refer to the US and Canada [Hatamoto et al. 1991; Popov et al. 1992; Gentry, Wight 1994; La Fave, Wight 2001; Quintero-Febres, Wight 2001] while other works correspond rather to Australia [Stehle et al. 2001; Siah et al. 2003; Goldsworthy, Abdouka 2012]; most of the works conclude that the seismic capacity of the tested connections is limitted. Since the conditions of the wide-beam slabs in these countries differ significantly from those in Spain, these results cannot be applied directly to this study. Given that there is an important number of potentially vulnerable wide-beam buidlings in Spain and that there is a lack of reliable information, a research project aiming to assess the seismic capacity of potentially vulnerable buildings located in seismic prone areas of Spain and to propose retrofit strategies was launched a few years ago; this work is a part of such research activity. Two construction typologies were considered: waffle slabs and wide-beam slabs. About buildings with wide beams, the previous research has consisted mainly of cycling testing on connections between wide beams and columns [Benavent-Climent 2007; Benavent-Climent et al. 2009] and of investigating numerically the seismic vulnerability of buildings with wide beams located in regions of Spain with medium seismicity [Benavent-Climent, Zahran 2010]. Conversely, this work focuses on buildings located in regions of Spain with lower seismicity; the obtained experimental results are used in both numerical studies. Further research will aim to propose retrofit strategies for RC buildings with wide beams situated in Spain. This research might be also useful for similar constructions located in close countries, as France and Italy.

2. CONSIDERED BUILDINGS

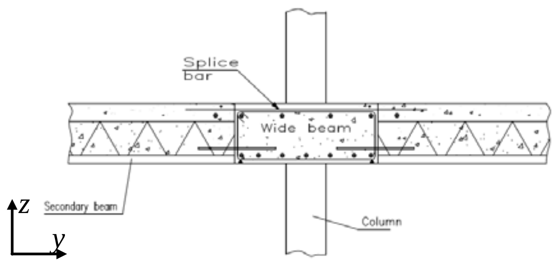
Six prototype buildings are chosen to represent the vast majority of the edifices with wide beams located in low-to-mid seismicity areas of Spain; all the buildings have four bays in both directions, two of them have three floors while the four other have six floors. Those constructions are globally described in Figure 2 while Figure 3 displays a plan view of a slab and two cross-sections of a wide beam and of secondary beams, respectively. Figure 2 and Figure 3 show that the considered buildings are both regular and rather symmetric; hence, no relevant twisting effects are expectable. Figure 3.a shows that in the *x* direction every one-way slab contains five wide beams while in the *y* direction there are two (outer) façade beams and three (inner) joists that are coplanar with columns. Figure 3.b and Figure 3.c show that the wide beams are wider than the columns while the width of the façade beams is equal to the one of the columns. As well, Figure 3.b and Figure 3.c also show that the joists are semi-prefabricated, being composed of a lower "sole" and a "truss-type" naked reinforcement; since prestressed prefabricated beams are also commonly employed as joists, have been also considered in the analyses. The top concrete layer is 4 cm thick and is not reinforced.



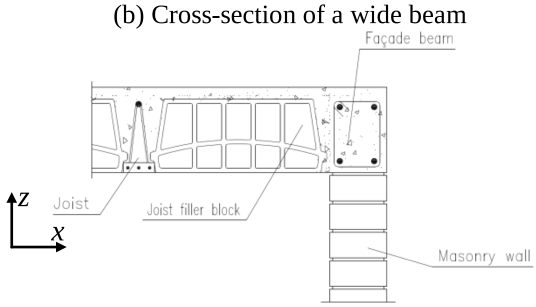
(a) Three-story building (b) Six-story building

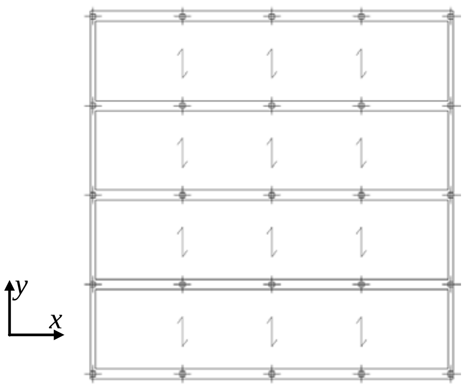
Figure 2. Selected representative buildings

Table 1 describes the main characteristics of the considered buildings; in 3 – 5 – ■, "3" refers to the number of floors, "5" corresponds to the span-length in both directions and "■" means that the columns have square cross-section. The fundamental periods correspond to the direction of the wide beams and to the orthogonal one, respectively; such directions are indicated as *x* and *y* in Figure 2 and in Figure 3. Such periods are determined from the numerical models of the buildings that are described in section 3, to be considered for the push-over and dynamic analyses (sections 4 and 5, respectively).



(b) Cross-section of a wide beam





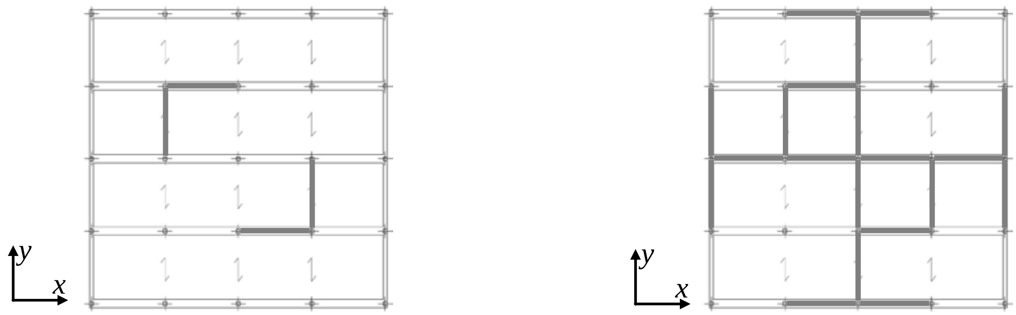
(a) Floor slab with wide beams (c) Cross-section of two secondary beams

Figure 3. Building slab with wide beams

Since the selected buildings possess only low lateral resistance, the cooperation of the infill walls cannot be neglected; however, in this study only the contribution of walls that are made with "Group 2" brick units [EN 1996 2005] being 12 cm thick has been accounted for. For each of the six representative buildings three wall densities have been considered: no walls, low density and high density. The first and second cases correspond to commercial buildings with light claddings while the third case corresponds to house buildings. Figure 4 depicts typical layouts of the walls for the second and third cases; since the infill walls are placed symmetrically in both directions, the horizontal behavior will be also symmetric. All these walls are assumed to be continuous down to the foundation.

Table 1. Representative buildings

Building	Stories / height (m)	Span-lengths of the beams (m)	Plan size (m)	Wide beams (cm)	First floor columns (cm)	Top floor columns (cm)	Fundamental periods w/o walls (*x* / *y*) (s)	Weight (kN)
3 – 5 – ■	3 / 10	5 × 5	20 × 20	25 × 60	40 × 40	30 × 30	0.585 / 1.037	9770
3 – 5.5 – ■	3 / 10	5.50 × 5.50	22 × 22	29 × 75	40 × 40	30 × 30	0.783 / 1.309	10825
6 – 5 – ■	6 / 19	5 × 5	20 × 20	25 × 60	50 × 50	30 × 30	1.333 / 2.630	20310
6 – 5.5 – ■	6 / 19	5.50 × 5.50	22 × 22	29 × 75	50 × 50	30 × 30	1.364 / 2.989	25640
6 – 5 – ▮	6 / 19	5 × 5	20 × 20	25 × 60	60 × 50	40 × 30	1.206 / 2.480	20875
6 – 5.5 – ▮	6 / 19	5.50 × 5.50	22 × 22	29 × 90	60 × 50	40 × 30	1.241 / 2.836	26115



(a) Low wall density (b) High wall density

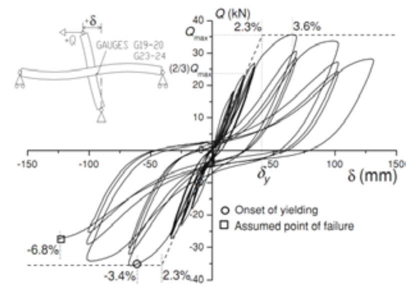
Figure 4. Layout of the infill walls in the considered buildings

The characteristic value of the concrete compressive strength is 17.5 MPa and the steel yielding point is 410 MPa; these are common values for RC structures in Spain before 1994. In the infill walls, the characteristic values of the brick and mortar strengths are 12 and 8 MPa, respectively.

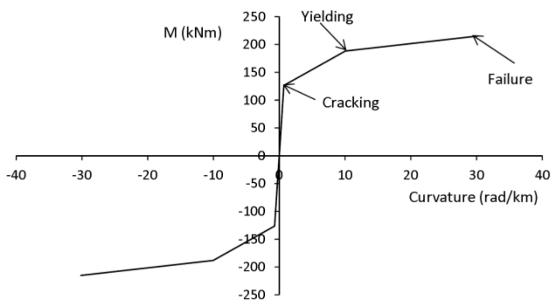
3. NUMERICAL MODELLING OF THE STRUCTURAL BEHAVIOR

The nonlinear static and dynamic structural behavior of the buildings in each direction is described with 2D frame finite element models; the infill walls are considered as compression-only bars joining adjacent floors. The diaphragm effect of the floor slabs is accounted for by rigid fictitious pin-ended bars connecting the outer nodes of the frames. Since the top concrete layer is not reinforced, the cooperation of the effective width of the slab with the beams and joists is not accounted for. The connections between the columns and the wide beams (*x* direction) and between the columns and the façade beams (*y* direction) are modeled as rigid since the reinforcement is assumed to be satisfactorily anchored (Domínguez 2012). Conversely, the connections between the columns and the joists (*y* direction) are only modeled as rigid for negative bending moments while are considered as ordinary hinges for positive bending moments since Figure 3.b shows that the lower reinforcement bars are not adequately anchored.

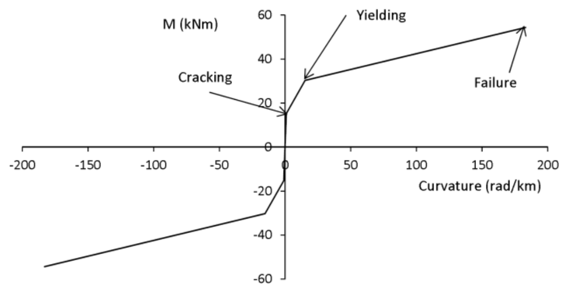
The behavior of concrete and steel is described by classical uniaxial constitutive laws; the stress-strain diagram for steel is bilinear with strain hardening, while the one of concrete is a parabola-rectangle model where the tension strength is neglected [EN 1992 2003]. In both the *x* and *y* frames, the nonlinear behavior is concentrated in plastic hinges located at both ends of each member; the length of each plastic hinge is estimated as half of the depth of the cross-section of the member [Reinhorn et al. 2009]. The hysteretic behavior of the plastic hinges of the columns, of the wide beams (*x* direction, see Figure 2 and Figure 3) and of the façade beams and the joists (*y* direction) is described by trilinear laws. Such laws are characterized by the cracking, yielding and failure moments and curvatures; these moments are determined according to [ACI 318-08 2008] and the obtained results are compared to those provided by the program Response 2000 [Bentz, Collins 1992], the agreement is satisfactory. The cracking curvature is determined from the initial sectional stiffness, calculated by classical linear analyses accounting for the contribution of the reinforcement bars. The yielding curvature is determined as suggested in [Sugano 1968]. The ultimate curvature of the wide beams and of the columns is determined according to the experimental results described in [Benavent-Climent 2007; Benavent-Climent et al. 2009]; these authors suggest estimating such curvature by multiplying the yielding curvature by a ductility factor equal to 12 for the wide beams belonging to outer connections, to 21 for the wide beams belonging to inner connections, and to 3 for the columns. The ductility factor of the façade beams (*y* direction) is assumed to be equal to the one of the outer wide beams. Given the lack of experimental results about the joists, their ultimate ductility curvature is conservatively estimated as 4; remarkably, since the contribution of the joists to the transversal lateral resistance is rather low, it is expectable that the overall behavior of the buildings in transversal direction (*y*) is not very sensitive to this parameter. In the columns, the interaction with the compressive axial force is taken into account [Reinhorn et al. 2009]. Figure 5.a shows experimental hysteresis loops [Benavent-Climent et al. 2009] of a connection between a wide beam and a column, and Figure 5.b, Figure 5.c and Figure 5.d show back-bone envelopes adopted for the numerical modeling of a column and an outer and an inner wide beam, respectively.



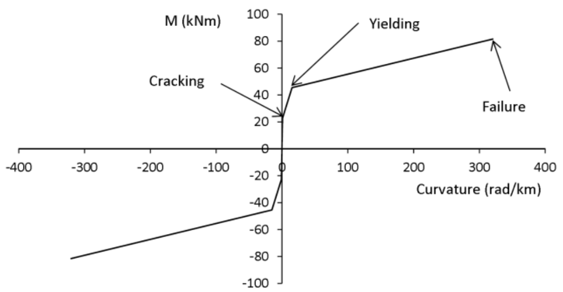
(a) Experimental loops [Benavent et al. 2009]



(b) Back-bone envelope for a column



(c) Back-bone envelope for an outer wide beam



(d) Back-bone envelope for an inner wide beam

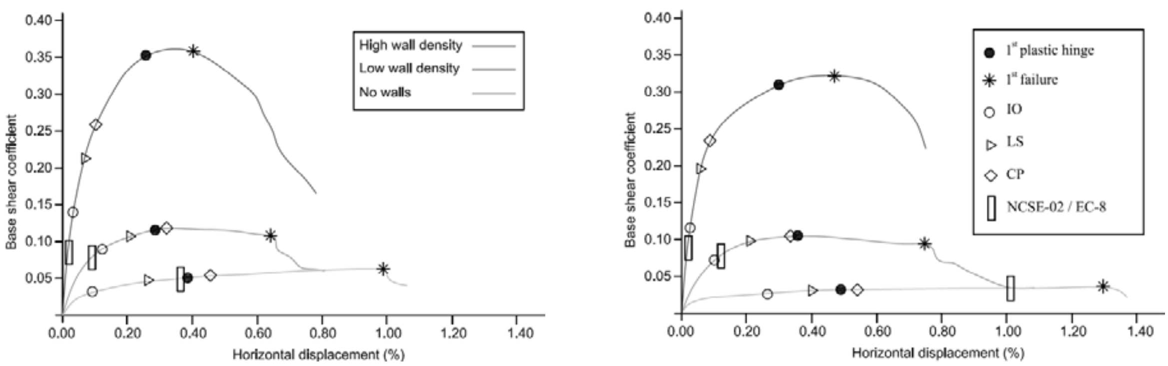
Figure 5. Hysteretic model of a connection between a column and beams and joists

The hysteretic behavior of the masonry infill walls is represented by Bouc-Wen models [Baber, Noori 1985]. Such models are characterized by two major parameters, i.e. the resistance and the initial stiffness. The resistance is obtained from tie-and-strut models. In such models, two major failure modes are considered: diagonal strut compression and horizontal sliding along a course; in all the analyzed cases, the resistance for the first failure mode is significantly smaller. The parameters for the tie-and-strut models are estimated as indicated by the Eurocode 6 [EN 1996 2005]. As suggested in [Mostafaei, Kabeyasawa 2004], the initial stiffness is estimated as two times the ratio between the ultimate resistance and displacement. The finite element models of the frames and of the infill walls are jointly implemented in the IDARC-2D version 7.0 code [Reinhorn et al. 2009]; the hysteretic behavior of the plastic hinges of beams, columns and joists is modeled as discussed previously (Figure 5). As well, hysteretic curves based on sectional fiber models (derived from the abovementioned material constitutive laws) are also implemented in that code. Since the agreement between both types of analyses is satisfactory [Domínguez 2012], only calculations based on the hysteretic curves shown in Figure 5 are considered next; such choice relies on the higher simplicity of such analyses.

4. PUSH-OVER ANALYSES

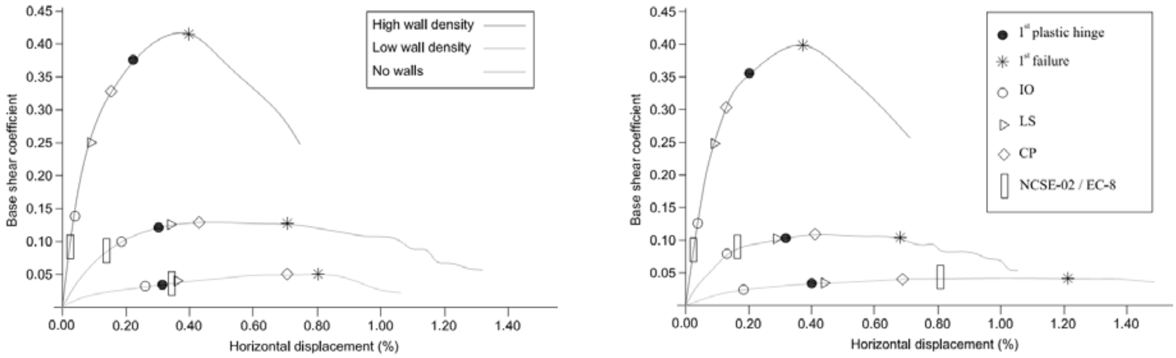
Two-dimensional push-over analyses are carried out on the six buildings described in Table 1; for each building and each horizontal direction, the three aforementioned wall densities are considered. Since the behavior of the infill walls is described with dynamic models, the capacity curves are not obtained by classical static nonlinear analyses but with incremental nonlinear dynamic analyses using a given register that is scaled with different factors. The considered input is the NS component of the Tolmezzo-Diga Ambiesta register of the Friuli earthquake (06/05/1976) [ESD 2012]. The results obtained with that register are compared with those calculated with other accelerograms belonging to the same data base; the agreement is satisfactory. As well, for the buildings without walls, conventional incremental static analyses are carried out; no relevant differences among them are observed. Given the high lateral flexibility of the considered buildings, second-order analyses are performed; in most of the cases the differences with the first-order analyses are small.

Figure 6 to Figure 11 show the capacity curves of the six considered buildings, respectively. Each Figure contains two sets of curves, the left one corresponds to the direction of the wide beams (*x*) while the right one corresponds to the transversal direction (*y*); each set is composed of three curves that are associated with the case without walls, with low wall density and with high wall density, respectively. In each capacity curve the performance points corresponding to the less demanding situation (soil A and design acceleration 0.08 g) are also represented. The target drifts (performance points) are determined, for each performance objective (IO, LS and CP) according to [FEMA 356 2000] by an iterative procedure aiming to obtain the intersection between the capacity curve and the demanding spectrum that corresponds to the damping that is equivalent to the achieved damage. Given that the use of the buildings corresponds mainly to normal importance, these performance objectives are assigned to return periods 100, 500 and 1000 years, respectively. For 500 years the design spectrum from the Spanish code [NCSE-02 2002] is adopted; for 100 and 1000 years that spectrum is modified according to the empirical expression suggested in that code. The performance points for IO, LS and CP are symbolized as "○", "▷" and "◇", respectively. In some capacity curves point "◇" is not present because the iterative procedure has not converged; this means that CP corresponds to collapse. The points "▯" indicate the design base shear coefficient prescribed by the most demanding design code (among the Spanish and the European ones). In certain capacity curves corresponding to low wall density, point "▯" is not present because the equivalent forces prescribed by the codes are too high. Points "•" and "✳" stand for the formation of the first plastic hinge and for the first failure, respectively.



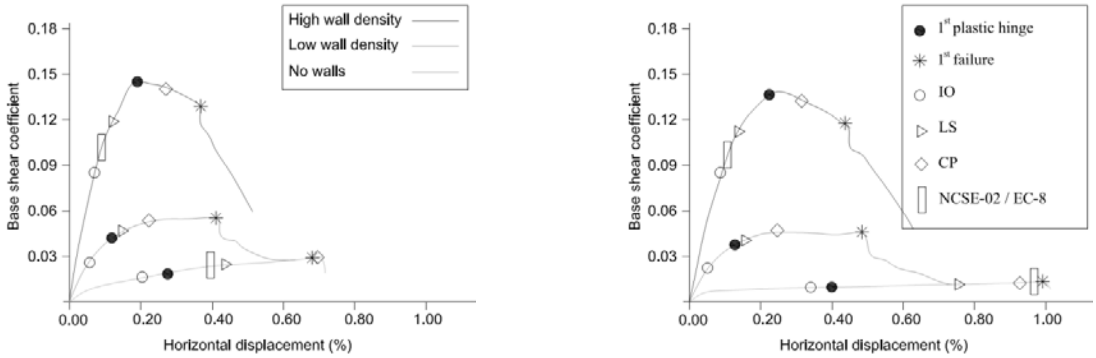
(a) Direction of the wide beams (*x*)
(b) Transversal direction (*y*)

Figure 6. Capacity curves and Target Drifts of building 3 – 5 – ■. Soil A and design acceleration 0.08 g



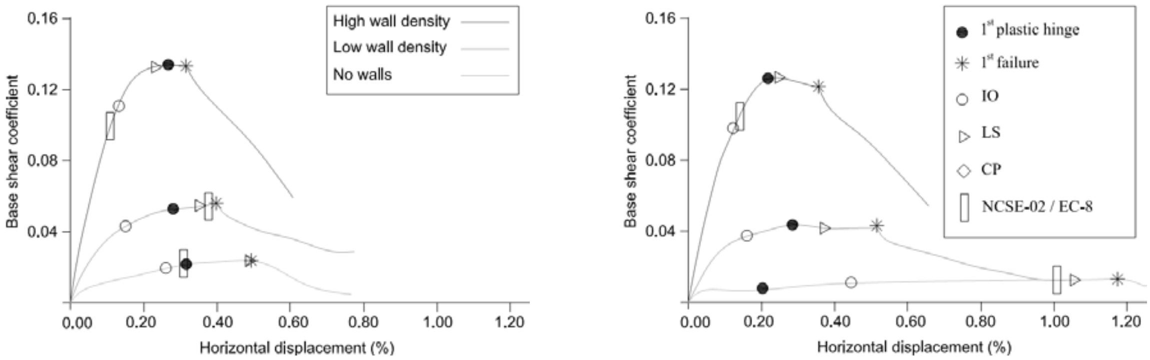
(a) Direction of the wide beams (*x*) (b) Transversal direction (*y*)

Figure 7. Capacity curves and Target Drifts of building 3 – 5.5 – ■. Soil A and design acceleration 0.08 g



(a) Direction of the wide beams (*x*) (b) Transversal direction (*y*)

Figure 8. Capacity curves and Target Drifts of building 6 – 5 – ■. Soil A and design acceleration 0.08 g

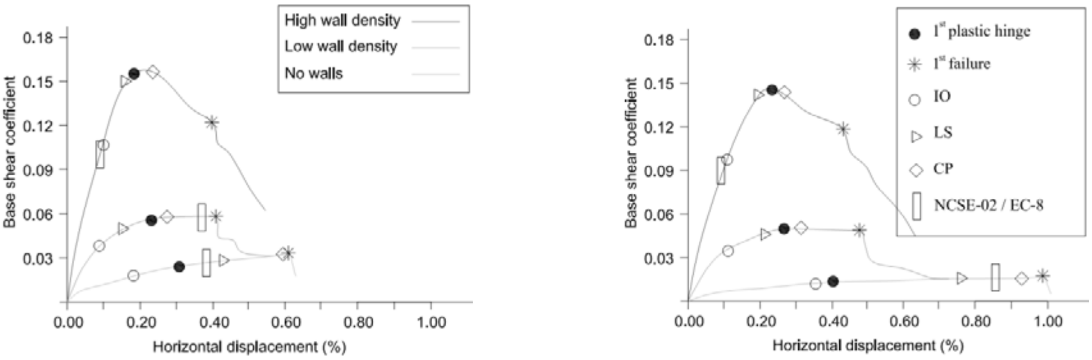


(a) Direction of the wide beams (*x*) (b) Transversal direction (*y*)

Figure 9. Capacity curves and Target Drifts of building 6 – 5.5 – ■. Soil A and design acceleration 0.08 g

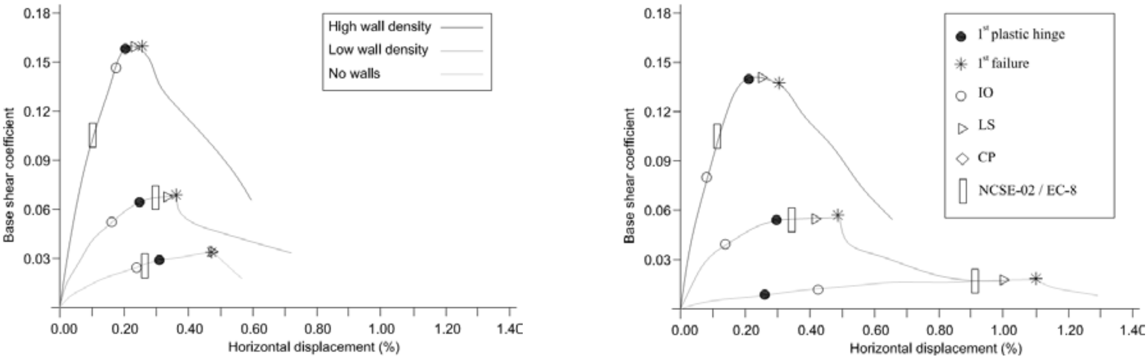
Plots from Figure 6 to Figure 11 show that the infill walls increase significantly the seismic strength in both directions. Comparison among left (a) and right (b) curves shows that in the transversal direction (*y*) the resistance of the buildings without walls is significantly smaller than in the longitudinal one (*x*), while this difference is strongly attenuated by the cooperation of the walls. In the initial segments, the curves for the buildings with walls are markedly above those of the buildings without walls; conversely, after than these top curves reach their peaks, they descend abruptly and tend to converge with the bottom curves. Since those peaks appear to correspond to the collapse of the walls, this trend shows that the walls fail prior to the main structure; remarkably, points "•" are either coincident or slightly earlier than those peaks. These global inferences apply for all the considered cases; the observation of the Target Drifts and the other points shown in [Domínguez 2012] provide the following conclusions applicable to Soil A and to design acceleration 0.08 g:

- Comparison among points “▯” and “▷” indicates that the requirements of the considered design codes (assuming the aforementioned values of the response reduction factors) are often inadequate. For the 3-story buildings without walls the design codes tend to be slightly more demanding while this situation clearly inverts for the cases with walls. For the 6-story buildings without walls the agreement between the seismic forces derived from push-over analyses and those prescribed by the codes is reasonable. For the 6-story buildings with low wall density the requirements of the design codes are clearly excessive; they are beyond the building capacity, particularly in the transversal direction (*y*). For the 6-story buildings with high wall density the requirements of the design codes are commonly too low. These conclusions can be broadly generalized for the design acceleration 0.11 g and for the other soil types [Domínguez 2012].
- Comparison among points “▷” and “•” shows that in the high-wall-density buildings the LS Target Drift is either earlier than the first plastic hinge or rather simultaneous; this indicates a highly proper behavior. In buildings with low wall density or with no walls this situation attenuates or reverses, mainly in the transverse direction (*y*) of the 6-story buildings without walls.
- Comparison among points “◇” and “✳” shows that in buildings with 5 m span-length the CP Target Drift is slightly earlier than the first failure; this indicates a proper behavior. In 6-story buildings with 5.5 m span-length, points “◇” are not present; this means collapse.
- Comparison among the onset of yielding and points “▯” shows that the buildings without walls do not fulfill the seismic design codes. The cooperation of the infill walls improves this situation but still the design codes are not fulfilled in all the cases. Given that Figure 6 to Figure 11 correspond to the less demanding situation, in the other cases the degree of fulfillment is significantly lower.



(a) Direction of the wide beams (*x*) (b) Transversal direction (*y*)

Figure 10. Capacity curves and Target Drifts of building 6 – 5 – ▌. Soil A and design acceleration 0.08 g



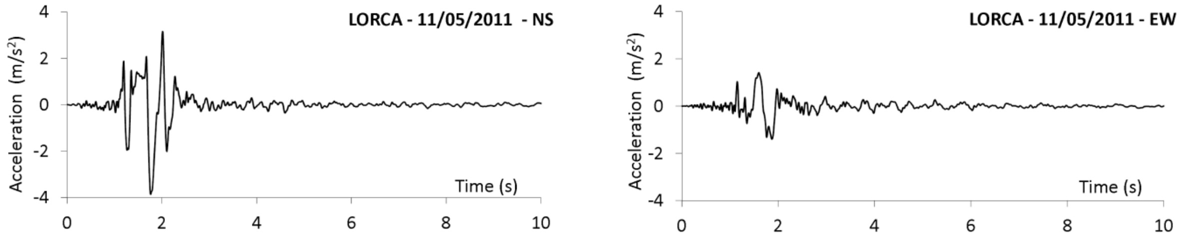
(a) Direction of the wide beams (*x*) (b) Transversal direction (*y*)

Figure 11. Capacity curves and Target Drifts of building 6 – 5.5 – ▌. Soil A and design acceleration 0.08 g

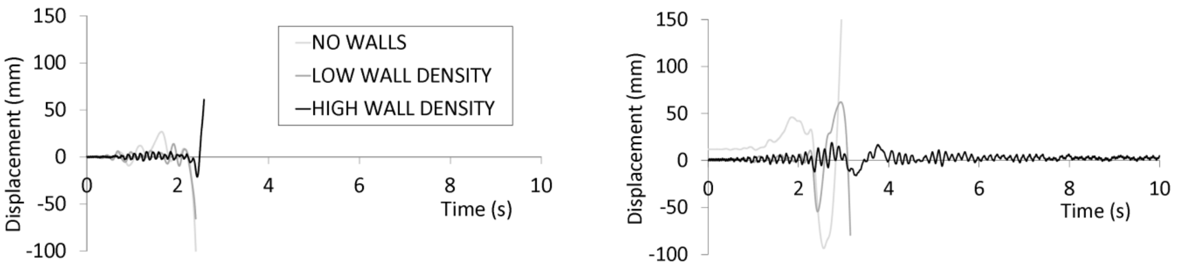
5. DYNAMIC ANALYSES

A number of nonlinear dynamic analyses have been carried out on the considered buildings (Table 1).

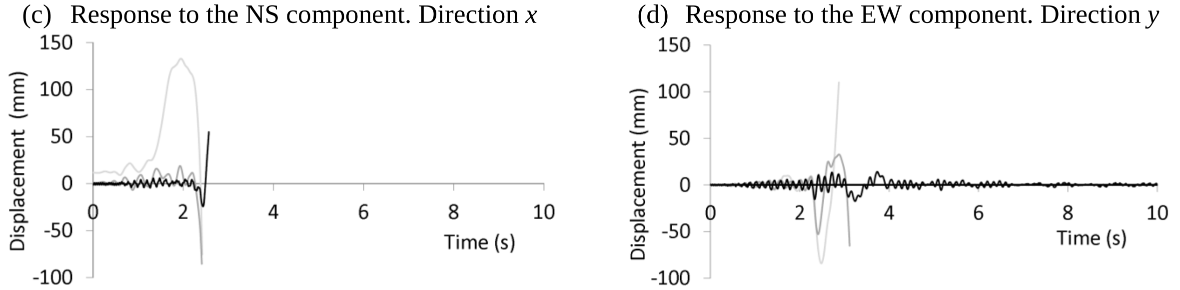
The input accelerograms have been taken from the European Strong-Motion data base [ESD 2012]; only registers whose response spectra roughly match the shape of the design one [NCSE-02 2002] along the range of periods of interest, are selected. The considered accelerograms are scaled to the design spectrum that corresponds to return period 500 years. As well, the strongest input that has been recently registered in Spain (Lorca earthquake, 11/05/2011) has been also considered [IGN 2011]. Figure 12 shows the NS and EW components of the Lorca accelerogram and the time-history displacement responses of the top floor of building 3 – 5 – ■ to such registers. Plots from Figure 12 show that the building 3 – 5 – ■ collapses in most of the cases. These conclusions can be broadly generalized to the other 3-story buildings while the corresponding performance of the 6-story buildings is more deficient [Domínguez 2012].



(a) Input North-South component (NS) (b) Input East-West component (EW)



(c) Response to the NS component. Direction *x* (d) Response to the EW component. Direction *y*



(e) Response to the NS component. Direction *y* (f) Response to the EW component. Direction *x*

Figure 12. Lorca register (11/05/2011) and response of building 3 – 5 – ■

6. CONCLUSIONS

This work presents a numerical study of the seismic vulnerability of RC buildings with one-way wide-beam slabs located in low-to-moderate seismicity regions of Spain; those buildings were designed without any seismic consideration. Two 3-story and four 6-story buildings are selected to represent the majority of the existing ones; such buildings differ in their span-length (5 and 5.5 m) and in the cross-section of the columns (rectangular and square). The cooperation of the infill walls is considered; three wall densities are considered: no walls, low density and high density. The vulnerability is investigated by code-type analyses, by push-over analyses and by nonlinear dynamic analyses.

The overall conclusion of this work is that the considered buildings show an inadequate seismic behavior in most of the analyzed situations. The seismic performance, relative to the design requirements, tends to worsen as the return period of the design input decreases. Since these buildings are selected to be representative of the vast majority of buildings with wide beams that were constructed in Spain prior to 1994 without accounting for any seismic consideration, these conclusions can be generalized to them.

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