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INFLUENCE OF THE CONSTRUCTIVE FEATURES OF RC EXISTING BUILDINGS IN THEIR DUCTILITY AND SEISMIC PERFORMANCE

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Abstract

This paper aims to analyse the ductility of existing buildings to evaluate its influence in their seismic performance. This has been achieved through the retrospective analysis of an existing building in accordance with current seismic codes. This study has revealed the lack of guidance in the NCSE02 and the EC8 regarding the assessment of the ductility of existing buildings. This manuscript proposes a methodology to assess the ductility of Spanish existing buildings. For this purpose, this paper has implemented the American code procedure combined with the EC8 and the Spanish seismic code (NCSE02) requirements. A two-storey RC frame school located in the Spanish region of Huelva has been selected as a case study. Different versions of this school have been compared: as built, designed according to current best practice, EC8 provisions and NCSE02 provisions. To do so, different constructive features have been varied according to each of the ductility class requirements: geometrical properties and the reinforcement ratio of the structural elements. Nonlinear static analyses have been carried out to obtain the displacement ductility factor (μ) and the behaviour factor (q) of each model. Construction costs and the expected damage index have been determined and compared. Results have shown that the best performance, regarding the ductility and the costs, has been obtained with the models designed with deep beams. Conversely, models with wide beams, and where only the reinforcement ratios have been varied, have merely shown a slight enhancement of the resistant and ductile capacity. It has been concluded that the ductility affects the shear capacity, the seismic performance and the expected damage of RC buildings.

Keywords

Reinforced concrete buildings; nonlinear static analysis; finite element method; ductility; behaviour factor; damage level probability.

Declaration of interest

None

Nomenclature

Geometrical parameters

b	cross-section overall width; subindexes "c" and "b" refer to columns and beams, respectively
b_0	dimension of the concrete core
h	beam depth
ϕ	diameter; subindexes "top", "bot" and "int" refer to the position of the beam rebar i.e. top, bottom or intermediate, respectively. The subindex "cor" refers to the longitudinal rebar in the corner of columns
s	stirrups separation; subindexes "m" and "c" refer to the middle and the corner zone
ρ	reinforcement ratio; superindex " ' " refers to the steel compression ratio; subindexes "cs"

Seismic action parameters

a_{gR}	EC8 reference peak ground acceleration on soil type A
a_c	basic ground acceleration (NCSE02)

Material parameters

σ	stress; subindexes "c" and "t" refer to compression and tensile stress, respectively
ε	strain; subindexes "c" and "t" refer to compression and tensile strain respectively; superindexes "el" and "pl" refer to elastic and plastic strain, respectively
E_0	undamaged modulus of deformation
E_{ci}	tangent modulus of deformation of concrete for zero stress

	and "long" refer to the cross-section and the longitudinal reinforcement ratio for both beams and columns
$\varepsilon_{\phi sy,d}$	design steel strain at yield
ω_{wd}	mechanical volumetric ratio of stirrups within critical regions
Nonlinear analysis parameters	
μ	displacement ductility factor; subindex "ϕ" refers to the curvature ductility factor
q	behaviour factor (EC8); R in American codes
q_0	basic value of the behaviour factor
Δ	displacement; subindexes "y", "u" and "m" refer to yield, ultimate and maximum displacements
V	basal shear force; subindexes "y", "d", "e", "m" and "1y" refer to yielding, design, elastic response, maximum and first significant yield basal shear force, respectively.
α_u/α_1	overstrength ratio (EC8)
K_c	ratio of second stress invariants on tensile and compressive meridians
Ψ	dilatation angle
f_{b0}	biaxial compressive yield strength
f_{c0}	uniaxial compressive yield strength
ϵ	eccentricity of the plastic potential surface
v	viscosity
f_{ck}	characteristic concrete compressive strength
f_{cm}	concrete compressive stress strength
f_{ct}	concrete tensile stress strength
G_{ch}	crushing energy per unit area
G_F	fracture energy per unit area
b	$\varepsilon_c^{pl}/\varepsilon_c^{ch}$ ratio
a_c/a_t/ b_c/b_t	dimensionless coefficient; subindexes "c" refer to compression and "t" to tension
l_{eq}	mesh size (finite element characteristic length)
ν	poisson's ratio
E_s	steel deformation modulus
F_y	steel yield stress

1. Introduction

In Spain, the vast majority of the existing reinforced concrete (RC) buildings has been constructed prior to current seismic codes. Consequently, no seismic considerations were implemented in their design procedure (Manfredi y Masi 2017). Even if considered, those requirements were not as restrictive as those in the provisions of the current code. Therefore, the seismic strengthening of these buildings has been the focus of several works over the last few years (O'Reilly y Sullivan 2018). In fact, the seismic retrofitting of buildings requires a preliminary structural evaluation of their seismic performance as pointed out in Part 3 of the Eurocode-8 (EC8-3) (European Union 2005).

One of the most important seismic structural parameters is ductility. Ductility considerations were first introduced in the 1960's and 70's as it was observed that some structures behaved better than predicted (Calvi et al. 2008). This parameter assesses the ability of a structure to undergo large deformations in the inelastic range without a substantial reduction in strength (Park 1998). Furthermore, the Eurocode-8 Part 1 (EC8-1) (European Union 2004a) establishes that it shall be verified that structures possess an adequate level of ductility.

Ductility is a critical parameter in the seismic assessment of new and existing buildings (Alam Shahria et al. 2012). The effects of ductility are commonly considered by reducing the elastic response spectrum using a behaviour factor and performing an elastic analysis of the structure (Ferraioli et al. 2014). The behaviour factor (q in the EC8-1 and R in the American code NEHRP) depends on the structural system of the building (Kappos 1999). Thus, a correct evaluation of the behaviour factor of a structure plays a key role in the seismic safety assessment. This assumption was highlighted by the experimental results observed during past seismic events (Vona y Mastroberti 2018).

Ductility also depends on the overstrength ratio (α_u/α_1), which is defined as the ratio between the ultimate strength of a structure and its first yielding strength. The importance of overstrength has been proven by experimental and numerical research on the performance of buildings during severe earthquakes (Zahid et al. 2013). The EC8-1 states that the overstrength ratio of existing buildings must be verified.

It should be noted that most seismic evaluations of existing buildings check whether life safety is ensured, without making thorough considerations of their ductile behaviour (Alam Shahria et al. 2012), while, in fact, the severity and significance of the seismic damage depends heavily on the ductility as shown in Fig. 1 (Applied Technology Council (ATC) 1998). Therefore, structural verifications should be made considering the ductility of a building in order to provide adequate retrofitting schemes (Zerbin et al. 2019).

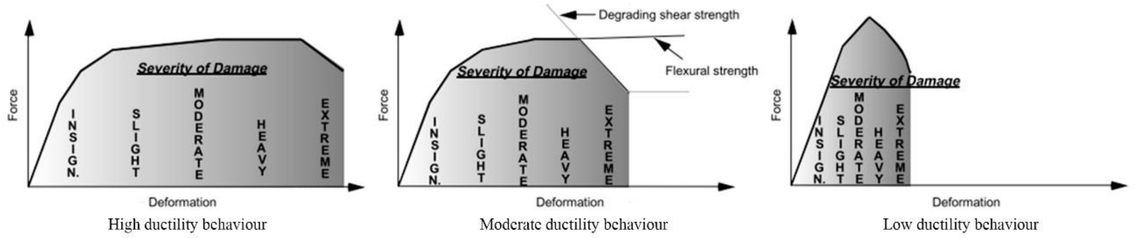


Fig. 1. Component force-deformation behaviour, ductility, and severity of damage - FEMA 306 (Applied Technology Council (ATC) 1998).

This paper aims to analyse the ductility of existing buildings to evaluate its influence in their seismic performance. This has been achieved by means of the retrospective analysis of an existing building in accordance with current seismic codes. Different versions of a case study building have been compared: as built, designed according to current best practice, EC8 provisions and NCSE02 (Spanish Ministry of Public Works [Ministerio de Fomento de España] 2002) provisions. To do so, different constructive features (the geometrical properties and the reinforcement ratio of the structural elements) have been varied according to each of the ductility class requirements. Nonlinear static analyses have been carried out to obtain the displacement ductility factor (μ) and the behaviour factor (q) of each model. Construction costs and the expected damage index have been determined and compared.

Finally, it should be mentioned that the works described in this paper are framed within the PERSISTAH project (*Projetos de Escolas Resilientes aos SISmos no Território do Algarve e de Huelva*, in Portuguese). The project is focused on the seismic retrofitting and vulnerability reduction of primary school buildings located in Algarve (Portugal) and Huelva (Spain). Schools are some of the most vulnerable buildings to earthquakes as concluded in (O'Reilly et al. 2018). In addition, the seismic hazard of the region is considerable due to the proximity of the Eurasian-African tectonic plates boundary (Amaro-Mellado et al. 2017). Moreover, it has been affected by some of the most remarkable historical earthquakes suffered in Europe, such as the 1755 Lisbon earthquake (M_w=8.5) and the 1969 earthquake (M_w=8) (Sá et al. 2018).

2. State of the art

Research related to the ductility of buildings is of great interest and has been the focus of considerable attention over the years (Kappos 1999). Even so, the vast majority of works are based on analysing the behaviour and overstrength factors in the seismic design of new buildings (Žižmond y Dolšek 2016). These factors are quite related since the global ductility of buildings increases as the overstrength factor does (Taieb y Sofiane 2014). A major part of the studies is focused on analysing the behaviour of buildings by means of the American *R*-factor. It has a similar purpose as the behaviour factor introduced by the EC8-1 (Mondal et al. 2013). In (Elnashai y Mwafy 2002), a conservative overstrength value of medium and low period RC buildings of 2.0 was proposed. Even the *R*-factor for steel moment-resisting frames was analysed in (Ferraioli et al. 2014) by means of nonlinear static and dynamic analyses. Also, the response modification factor of RC moment-resistant beams adding steel slit panels was calculated in (Zerbin et al. 2019).

Some other works are based on proposing new approaches to obtain the behaviour factor. In (Costa et al. 2010), a new probabilistic methodology for the calibration of the behaviour factor according to the EC8-1 procedure was presented. The authors assessed a set of regular and irregular structures that comprised different RC frame structures. Considering different element configurations to represent typical RC buildings is the common approach followed in these studies. Also, in (Zahid et al. 2013), the authors obtained both the behaviour and overstrength factors for regular and irregular buildings designated according to different European codes. However, they did not vary the geometry nor the reinforcement ratio of the elements.

Likewise, some works can be found on the analysis of the ductility of buildings located in Spain. In (Vielma et al. 2010), nonlinear static and dynamic analyses were carried out to estimate the ductility and overstrength factors of typical RC frame typologies in Spain. In (Gómez-Martínez et al. 2016a), a comparison between the seismic performance and the behaviour factor of RC Spanish-code-designed wide beam and deep beam frames was carried out. Results showed that wide beams had lower local ductility than deep beams. However, ductility factor results were slightly higher than those established in the Spanish code for wide beams.

Few studies can be found on the analysis of the ductility of existing buildings, despite the importance of such analyses. In (Vona y Mastroberti 2018), several existing RC buildings were assessed to obtain their behaviour factor through a new approach. Results pointed out the lack of guidance available in seismic codes to assess the behaviour factor of existing buildings.

Although nonlinear dynamic analyses have been gaining considerable interest over the past few years, nonlinear static analyses provide reliable results concerning the seismic behaviour of structures. In (Ferraioli et al. 2014), nonlinear static and dynamic analyses were carried out concluding that realistic overstrength ratios could be obtained from pushover analyses. Moreover, in (Anagnostopoulou et al. 2015), similar results concerning the inelastic response of a structure were found using either static pushover or dynamic analysis. Furthermore, the EC8-1 establishes that verifying the overstrength ratio values can be performed by means of nonlinear static analyses (European Union 2004a).

Taking into account the ductility of buildings requires accurate analysis methods. The nonlinear seismic response of existing building could be evaluated through macro-element methods. Although the computational stress is reduced, these analyses are not exhaustive. Finite element methods (FEM) have become a very effective tool to analyse the seismic behaviour of buildings. They can provide detailed results of the structural response as well as an accurate implementation of the structural element configuration. There are several works based on the use of FEM to analyse the ductility of buildings. In (Reza Azadi Kakavand et al. 2018), a comparison between FE models and experimental results was performed with regards to the ductility of an RC frame. In (Demir et al. 2016a), numerical FE calculations were used to analyse the performance of a new shear reinforcement configuration in the ductility of RC beams. The authors pointed out that decreasing the spacing of stirrups would lead to insufficient bond between concrete and reinforcement. In (Abou-Elfath y Elhout 2018), several RC frame configurations were defined by varying the number of storeys, element dimensions, and the reinforcement ratio. FE calculations were performed concluding that the R-factor is sensitive to both the number of storeys and the storey height.

The state of the art reveals that focus has been made so far on analysing the ductility in the seismic design of new buildings. Moreover, it has been found that there is a lack of guidance available in seismic codes to assess the behaviour and overstrength factors of existing buildings. Therefore, this paper aims to provide guidance to engineers assessing the ductility of existing structures. Research on the ductility of such buildings is mainly focused on artificial models. Contrariwise, in this work, the method proposed is applied to a real case-study building.

3. Method

The present section is divided into the following parts: seismic code requirements, nonlinear static analysis, finite element model technique, determination of material properties, building configuration and construction costs and damage determination. The method proposed in this study is shown in the graphical framework below (Fig. 2).

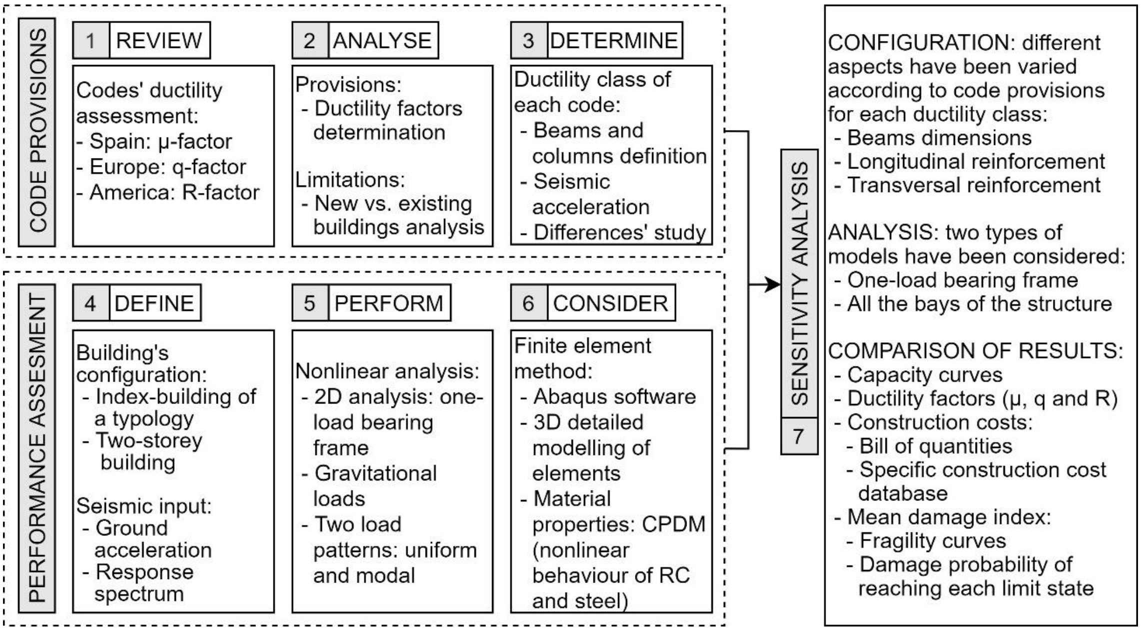


Fig. 2. Graphical framework of the study.

3.1. Seismic code requirements

Clear procedures to analyse the ductility of existing buildings are hard to find due to the lack of studies (Vona y Mastroberti 2018). Regarding the ductility, the EC8 is mainly focused on new buildings. It classifies the ductility of RC buildings into three classes: low (DCL), medium (DCM) and high (DCH) dissipative capacity. The EC8 q-factor is used to reduce the elastic response spectrum in seismic design (Ferraioli et al. 2014). It is obtained according to Eq. (1) for each ductility class.

$$q = q_0 k_w \geq 1.5 \tag{1}$$

where:
q_0 is the basic behaviour factor, which depends on the structural type, the ductility class, and the overstrength ratio.
k_w is the factor reflecting the prevailing failure mode in structural systems with walls.

In this work, the building selected is characterized by a multi-storey and multi-bay frame system. For that system, the EC8 specifies an overstrength α_u/α_1 ratio of 1.30 and a k_w of 1.00. Therefore, the q-factor for DCM and DCH is 3.60 and 5.85, respectively. In the case of DCL structures, the q-factor is 1.50. Furthermore, the q-factor must be modified for irregular buildings in elevation and in plan, which is not the case.

In the case of existing buildings, the EC8 does not provide guidance to obtain the q-factor. However, in (Kappos 1999), a procedure related to the EC8 philosophy was proposed. The q-factor is obtained through Eq. (2), which is similar to the American R-factor definition. In this work, this procedure for obtaining q-factor has been implemented.

$$q = q_{\mu} q_{\Omega} q_{\xi} q_{S} \tag{2}$$

where:

- q_{μ} is the ductility factor. Its value varies according to the period (T) of the structure to be analysed. It is used to determine the acceleration spectrum (S_a) and the displacement spectrum (S_d) of an inelastic single-degree-of-freedom system (SDOF). It is then used to obtain the target displacement of the performance point as in (Fajfar 1999).
- q_{Ω} is the behaviour factor which depends on the effects of the overstrength (Khose et al. 2012). It depends on the material properties and the sizes and dimensions of the structural elements (Alam Shahria et al. 2012).
- q_{ξ} is the damping-dependent factor which takes into account the effect of "added" viscous damping. It must be considered in structures with supplementary devices (Mondal et al. 2013) which is not the case.
- q_{S} is the redundancy factor which takes into account plastic hinge redistribution. In (Vona and Mastroberti 2018) and (Mondal et al. 2013), the authors considered a unitary value indicating correct plastic hinge distribution. This assumption has also been considered in this work.

Hence, in the case under study, the behaviour factor depends only on q_{μ} and q_{Ω} (Mwafy et al. 2002).

The Spanish NCSE02 also establishes five ductility classes. However, the three main classes can be assimilated to the EC8 classification. In this work, for the sake of simplicity, the same nomenclature will be used for all of them: DCL, DCM, and DCH. In NCSE02, the classes are limited by displacement ductility factor (μ) values; i.e. 2 for DCL, 3 for DCM, and 4 for DCH.

Coherently with the above statements, both the NCSE02 and the EC8 establish different specific provisions to satisfy the appropriate ductility capacity for each class. In Tables 1 and 2, these configuration requirements are shown for beams and columns, respectively.

Table 1. Specific provisions for beams according to each seismic code and ductility classes.

	NCSE02		**EC8**	
	DCM	**DCH**	**DCM**	**DCH**
General properties			Concrete ≥ C16/20 Only ribbed bars Steel class B or C	Concrete ≥ C16/20 Only ribbed bars Only steel class C
Geometrical properties	b_b≥200 mm	b_b≥250 mm	$b_b \leq \min\{b_c+h_w;2b_c\}$ $h_b/b_b \leq 3.5$ (EC2)	b_b≥200 mm $b_b \leq \min\{b_c+h_w;2b_c\}$ $h_b/b_b \leq 3.5$ (EC2)
Reinforcement ratio	In the top: At least 2ϕ14 mm In the bottom: At least 2ϕ14 mm and 4‰ ≥A/3 in the ends (Being A the maximum tension rebar area at the top ends) Both in the top and bottom: ≥A/4 in all length (Being A the maximum negative rebar area at the ends)	In the top: ≥A/3 in all length (Being A the maximum negative rebar area at the ends) In the bottom: ≥A/2 in all length (Being A the maximum tension rebar area at the ends)	Maximum and minimum reinforcement ratio in the tension zone of beams: $\rho_{max} = \rho' + \frac{0.0018}{\mu_{\phi}\varepsilon_{\phi sy,d}}\frac{f_{ck}}{F_y}$ $\varepsilon_{\phi sy,d}$ is 0.2% (EHE08) $\rho_{min} = 0.5\frac{f_{cm}}{f_{yk}}$ ≥A/4 in all length in both top and bottom (Being A the maximum top reinforcement at the supports)	Maximum and minimum reinforcement ratio in the tension zone of beams: $\rho_{max} = \rho' + \frac{0.0018}{\mu_{\phi}\varepsilon_{\phi sy,d}}\frac{f_{ck}}{F_y}$ $\rho_{min} = 0.5\frac{f_{cm}}{f_{yk}}$
Intermediate rebar		2ϕ10 each 250 mm	-	-
Stirrups	ϕ≥6 mm	ϕ≥6 mm	ϕ_{stirr}≥6 mm	ϕ≥6 mm

	Critical region: extend 2h from column surface $s_{max} \leq \{h_b/4; 8\phi_{minlongrebar}; 150\}$ Middle region: $s_{max} \leq h_b/2$	Critical region: extend 2h from column surface $s_{max} \leq \{h_b/4; 6\phi_{minlongrebar}; 150\}$ Middle region: $s_{max} \leq h_b/2$	Critical region: first stirrup separated from beam end min 50 mm All length: $s_{max} \leq \{h_b/4; 24\phi_{stirr}; 225; 8\phi_{minlongrebar}\}$	Critical region: first stirrup separated from beam end min 50 All length: $s_{max} \leq \{h_b/4; 24\phi_{stirr}; 175; 6\phi_{minlongrebar}\}$

Table 2. Specific provisions for columns according to each seismic code and ductility classes.

	NCSE02	**EC8**	
	Mandatory if $a_c \geq 0.12g$	**DCM**	**DCH**
Geometrical properties	$b_c \geq 250$ mm		$b_c \geq 250$ mm
Reinforcement ratio		$0.01 \leq \rho \leq 0.04$ In symmetrical cross-sections: $\rho = \rho'$ Rebar maximum separation: 200mm	$0.01 \leq \rho \leq 0.04$ In symmetrical cross-sections: $\rho = \rho'$ Rebar maximum separation: 150mm
Intermediate rebar	At least, 3 rebar in each surface. $s_{max} = 200$ mm	One intermediate rebar between corner rebar	One intermediate rebar between corner rebar
Stirrups	$\phi \geq 6$ mm Middle region: $s_{max} \leq 15\phi_{stirrups}$ Critical region: extend $2b_c$ from beam surface. $s_{max} \leq \{b_c/3;\ 100$ if $\phi_{longrebar} \leq 14;\ 150$ if $\phi_{longrebar} \geq 16\}$ Being in this case b_c, the minor dimension of the column.	$\omega_{\omega d} = 0.08$ at the base of columns $\phi \geq 6$ mm $s_{max} \leq \{b_0/2;\ 8\phi_{stirrup};\ 175\}$	$\omega_{\omega d} = 0.08$ at the base and $\omega_{\omega d} = 0.12$ above the base of columns $\phi \geq 0.4\phi_{minlongrebar}\sqrt{F_y/f_{ck}}$ $s_{max} \leq \{b_0/3;\ 6\phi_{stirrup};\ 125\}$

Regarding the beam depth, both codes establish that only deep beams can be used in DCM and DCH structures. Both codes focus on the same parameters for the determination of the ductility class with different limits. In the case of the EC8, those limitations are more restrictive. Concerning the stirrups, both codes indicate similar restrictions for the spacing (using different formulae) and the diameter (≥6 mm) for each ductility class. The main differences can be found in the column design. In fact, the NCSE02 only indicates specific provisions for columns when the Spanish basic ground acceleration (a_c) is higher than 0.12g.

It should be noted that the minimum requirements from other structural codes may provide greater amount of reinforcement than that based on the seismic design (Žižmond y Dolšek 2016). In the case of Spain, the EHE08 (Spanish Ministry of Public Works [Ministerio de Fomento de España] 2008) concerning RC must be considered.

3.2. Nonlinear static analysis

The aforementioned factors, which describe the dissipative capacity of buildings, can be obtained by means of nonlinear static or dynamic analyses. The state of the art revealed that no significant differences were obtained from carrying out each type of analysis for in plan and in elevation regular structures. Therefore, in this work, the seismic performance of the models has been assessed by means of nonlinear static analyses (pushover). Torsional effects can play a key role in existing asymmetric buildings. In this case, the building is symmetric, presenting a homogenous distribution of frames, bays, infills, masses and openings (Fig. 3). Therefore, torsional effects can be considered negligible as in (Vielma et al. 2010) and the structures have been modelled in 2D. In addition, FEM has been used to model the structures. This method is preferred in earthquake engineering as it provides detailed information of the damage evolution in the concrete and steel (Reza Azadi Kakavand et al. 2018).

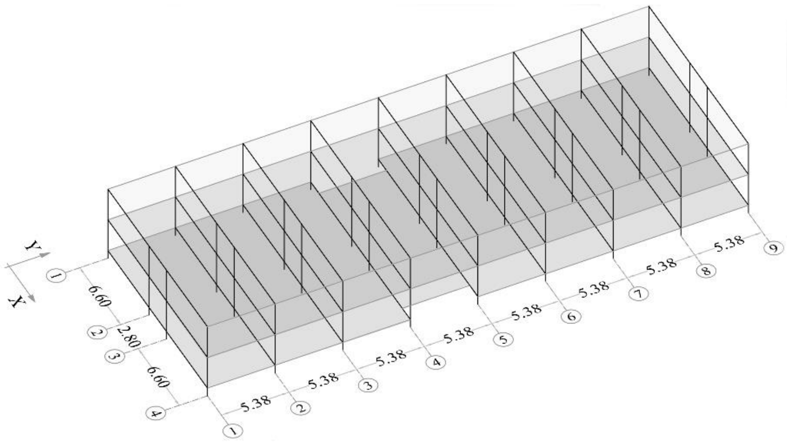


Fig. 3. 3D view of the building.

An evaluation of the *q*-factor of existing buildings has been undertaken by assessing the two components contributing to it (Mwafy et al. 2002): ductility and overstrength factors. These factors are based on the nonlinear behaviour of the building and the bilinearization of its capacity curve (Fig. 4). Several approaches can be found to estimate the bilinear curve as analysed by (Park 1998). In this study, the bilinear curve has been obtained according to the N2-method (Fajfar 2000). This method is based on the equivalent elasto-plastic energy absorption. Moreover, this method is settled in Annex B of the EC8-1 and it is later used to determine the performance point of the building.

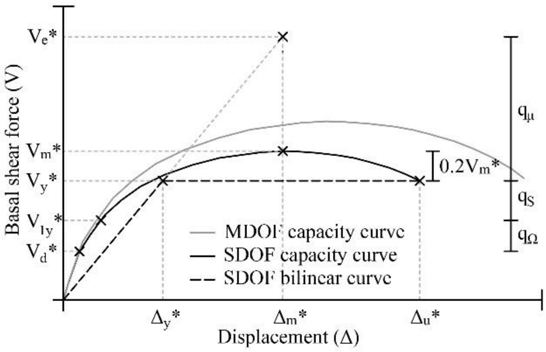


Fig. 4. Determination of the idealized elasto-perfectly plastic force-displacement relationship (EC8-1).

Where SDOF refers to the single degree of freedom system and MDOF refers to the multi degree of freedom system. Variables followed by * concern the SDOF system.

The first approach proposed for the assessment of the ductility factor was (Miranda y Bertero 1994). Since then, several approaches have been developed. However, (Miranda y Bertero 1994) remains the most used and has been widely accepted. The authors proposed the expressions Eq. (3) and (4) depending on the transition period (T_0) value of the building (Fajfar 1999). Eq. (4) is assumed for structures that exhibit a period higher than 0.5 seconds (Mwafy et al. 2002). In this work, Eq. (3) was used due to the value of the period of the models analysed.

$$q_\mu = \sqrt{2\mu - 1} \qquad T \leq T_0 \tag{3}$$

$$q_\mu = \mu \qquad T \geq T_0 \tag{4}$$

The displacement ductility factor was first defined in (Park 1998). It represents the ratio between the ultimate displacement and the yield displacement as expressed in Eq. (5). The overstrength factor represents the ratio between the yielding base shear and the design base shear as shown in Eq. (6) (Park 1998). This ratio is expressed as α_u/α_1 in the EC8. α_u represents the ultimate strength of the structure before collapse while α_1 is the force for the first plastic hinge is formed in any member (European Union 2004a).

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{5}$$

$$q_{\Omega} = \frac{V_y}{V_d} = \frac{\alpha_u}{\alpha_1} \tag{6}$$

3.3. Finite element model technique

In this study, the FE numerical simulations have been performed through the ABAQUS software (Dassault Simulia 2014a), which is a general-purpose FE program (Demir et al. 2016b). Models have been composed of 3D-solid concrete elements combined with 1D-truss steel rebar elements. Tie constraints have been determined to bond the concrete parts: columns to columns and beams to columns. The "embedded region" constraint has been determined to fully bond the steel rebars and the concrete parts (Alfarah et al. 2017). "Encastre" type boundary conditions have been determined in each column base. Meshing has been performed for each element part. It should be pointed out that the element type must be properly selected for each element during meshing.

Two steps have been generated to obtain the pushover curve in ABAQUS. First, a "Static, General" step in which the gravitational loads have been added. Then, a second "Static, Riks" step, in which the horizontal loads have been applied. The "Riks" method has been generally used to predict the unstable geometrically nonlinear collapse of a structure (Dassault Simulia 2014b). This method assumes that all load magnitudes vary with a single scalar parameter and no bifurcations occur during the analysis (Fu 2009). All applied loads have been designed as "concentrated" at beams nodes. These are the only load type permitted in the "Riks" step as well as "Body" type forces. EC8 establishes that two load patterns must be taken into account in nonlinear static analyses: one proportional to the product of the masses and the height of the storeys and one proportional to the first vibration mode displacements. In this case, the first one has been considered since similar capacity curves were obtained for each pattern as in (Requena-García-Cruz et al. 2019). Finally, the reaction forces (RF in Abaqus) have been obtained from the sum of each node force at the column bases and for the direction of interest. Then, the displacements (U in Abaqus) of the control node have been determined.

3.4. Determination of material properties.

There are several approaches to represent the nonlinear response of concrete i.e. plasticity and damage theories (Alfarah et al. 2017). The vast majority of works on concrete implemented these theories to simulate its nonlinear behaviour (Tao y Chen 2015). It should be noted that the concrete response in compression acts primarily according to the plasticity theory and in tension, it is attributed to the damage (Lubliner et al. 1989). Therefore, a new model named concrete plastic damage model (CPDM) was presented to consider both plasticity and damage theories (Dassault Simulia 2014b) and the inelastic behaviour of RC (Demir et al. 2016b). In this paper, the concrete was modelled according to the CPDM proposed by (Alfarah et al. 2017). This model was based on the formulation developed by (Lubliner et al. 1989) and (Lee y Fenves 1998), which is implemented in the ABAQUS CPDM input. Moreover, CPDM does not require calibration nor validation with experimental results and uses laws based on European recommendations.

The concrete behaviour depends on five constitutive parameters that are given in Table 3 and are commonly used in CPDM analyses as in (Demir et al. 2016b). The material parameters for the simulation of the concrete are tabulated in Table 4. The RC is designated as HA-175 and the steel rebar as AEH-400, in accordance with the existing documentation of the school analysed. These terms refer to the designation of the structural materials of old Spanish RC codes. The Poisson coefficient may be taken as 0.2 according to Eurocode-2 (EC2) (European Union 2004b). The material properties of the reinforcement steel are E_s 200,000 MPa, F_y 420 MPa and ν 0.3.

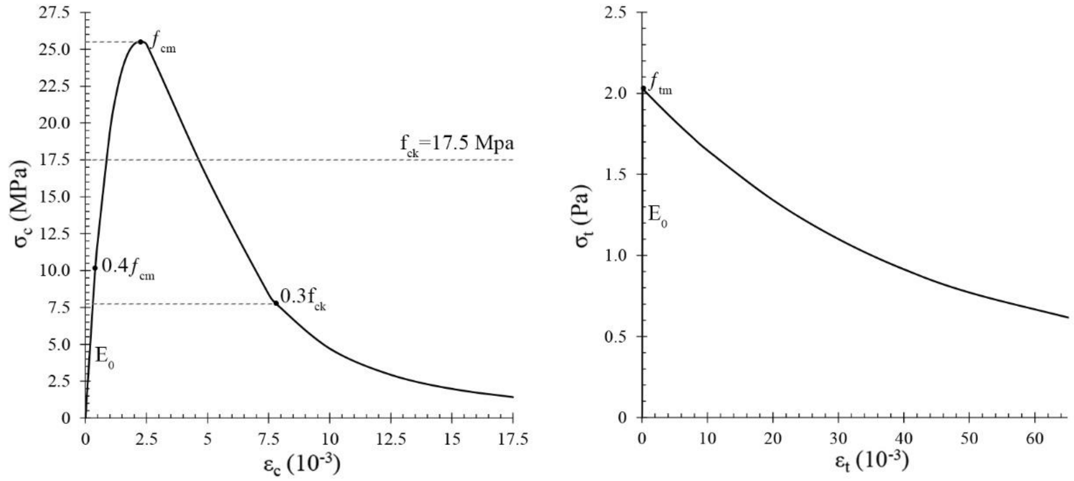
Table 3. Parameters of CPDM.

K_c	Ψ (º)	f_{b0}/f_{c0}	ϵ	ν
0.7	13	1.16	0.1	0.0001

Table 4. Parameters for the simulation of the concrete.

f_{ck} (MPa)	f_{cm} (MPa)	f_{ct} (MPa)	E_0 (MPa)	E_{ci} (MPa)	G_{ch} (N/mm)	G_F (N/mm)	b	a_c	a_t	l_{eq} (mm)	b_c	b_t
17.5	25.5	2.03	25,252.88	29,9433.82	20.57	0.13	0.8	7.87	1	125	305.22	2918.77

The stress/strain relation of uniaxial compressive and tensile behaviour of concrete are depicted in Fig. 5(a) and Fig. 5(b), respectively. These plots and the compressive/tensile damage laws (damage evolution) constitute the required input of any model to describe the global structural behaviour of RC (Alfarah et al. 2017). It should be noted that only the inelastic strain must be implemented in ABAQUS. It is calculated as the strain minus the elastic strain. The failure of the concrete is established as $0.3f_{ck}$ (Grassl et al. 2013).



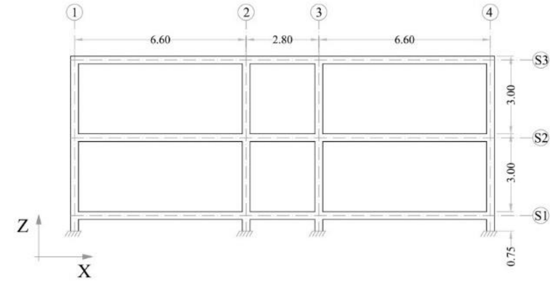
(a) (b)

Fig. 5. Stress-strain relation of uniaxial compressive (σ_c-ε_c) (a) and tensile (σ_t-ε_t) (b) behaviour of concrete.

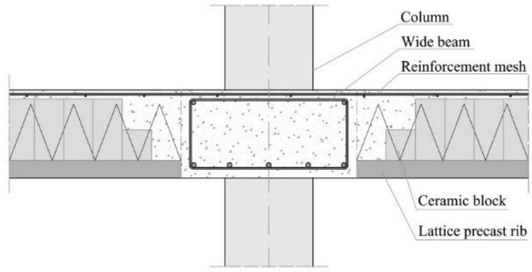
In addition, a sensitivity analysis on the variability of the material properties has been performed, considering a +/-10% variation of f_{ck}, E_0, E_s and F_y

3.5. Building configuration

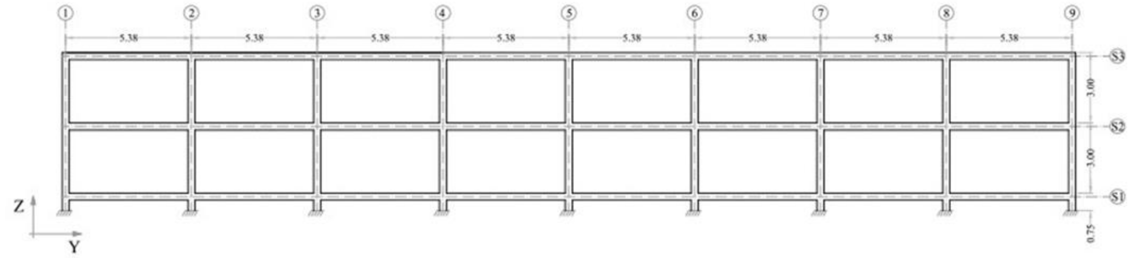
In Huelva, a total amount of 138 primary school buildings has been identified, 60% of which are two-storey RC frame buildings. Most of them were constructed during the 1970's leading to a lack of seismic considerations in their design, such as the use of RC wide-beams frames or short columns. These elements are known to be some of the main causes of building damage during earthquakes (Rodgers 2012). It should be noted that the NCSE94 (first Spanish seismic code) (Spanish Ministry of Public Works [Ministerio de Fomento de España] 1994) was introduced in 1994. In addition, these buildings share similar constructive characteristics: structural element sizes, height and bay dimensions. For these reasons, this type of building has been selected as the most relevant for this study. One of these buildings has been designated as the index building for the typology due to the amount and quality of available blueprints and documentation. It is a two-storey building composed of RC frames and of 30 cm thickness ribbed slabs, spanning in the Y direction. The views of the building, which are representative of the rest of the buildings of this typology, are shown in Fig. 6.



(a)



(b)



(c)

Fig. 6. School configuration: load bearing (a) and tie (c) frames geometry; and wide beam cross-section and ribbed slab configuration (b).

The building is located in Almonte, (Huelva), where a PGA value of 0.1g is determined according to the Spanish updated seismic action values (Spanish Ministry of Public Works [Ministerio de Fomento de España] 2012). The elastic response spectrum considered has been taken from the EC8 provisions. The PGA designated must be multiplied by 0.8 to obtain the EC8 reference ground acceleration (a_{gR}) as indicated by the Spanish EC8 National Annex (Spanish Ministry of Public Works [Ministerio de Fomento de España] 1998). This value is then multiplied by the importance factor (γ_I) to determine the design ground acceleration (a_g) according to EC8. School important class is III, resulting in a γ_I-factor of 1.30 (Spanish Ministry of Public Works [Ministerio de Fomento de España] 1998). According to a nearby geotechnical study, the soil is characterized by the presence of medium-low compactness silt-sand corresponding to a type C soil of EC8.

Additional gravitational loads (GL) to the self-weight (W) of the structural elements have been calculated using Eq. (7), according to the NCSE02 -dead loads (DL) and live load (Q)-.

$$GL = W + DL + 0.3Q \tag{7}$$

where:

DL are the sum of the weight of ribbed slabs, internal partitions, the ceiling and the ceramic flooring which is 5.5 kPa

Q is the correspondent value for public spaces according to the Spanish CTE-DB-SE-AE (Spanish Ministry of Public Works [Ministerio de Fomento de España] 2009), which is 3 kPa.

3.6. Versions of the case study building compared

Several configurations of the load-bearing frame (X-direction) have been analysed in order to determine the variability of the ductility value. The length of the bays has remained unchanged, while several combinations of the structural element dimensions and reinforcement ratios have been selected, reproducing typical RC school configurations. Models have been determined according to the EC8 and the NCSE02 specific provisions to comply with each ductility class. According to the codes, both the cross-section and the longitudinal reinforcement ratios for the DCM and DCH classes are similar. The NCSE02 does not provide specific provisions for columns since the a_c in Almonte is less than 0.12g. Therefore, only models designed according to EC8 have been considered in the analyses.

All models have been first analysed to comply with the stresses derived from *GL*. In fact, the EHE08 requirements for the minimum reinforcement ratio have been considered. Table 5 shows the geometrical properties of the structural elements and their reinforcement detailing of each of the models to be assessed.

Table 5. Geometric properties of each model to be assessed.

Models	Beams													Columns											
	b	h	n^o_{top}	ϕ_{top}	n^o_{bot}	ϕ_{bot}	n^o_{int}	ϕ_{int}	ϕ_{stir}	s_c	s_m	ρ_{cs}	ρ_{long}	h	b	n^o_{cor}	ϕ_{cor}	n^o_{int}	ϕ_{int}	ϕ_{stir}	s_c	s_m	ρ_{cs}	ρ_{long}
Real	600	300	3	10	6	20			6	255	255	1.18	2.07	300	300	4	12			6	263	263	0.50	2.07
RealMo	300	600	3	10	6	20			6	255	255	1.18	2.07	300	300	4	12			6	263	263	0.50	2.07
M1	300	400	3	16	6	20	2	10	6	100	250	2.07	2.62	300	300	4	12	4	10	6	95	195	0.85	3.45
H1	600	400	2	14	6	16	2	14	6	95	202	0.63	3.22	300	300	4	12	4	12	6	95	213	1.00	3.63
M2	600	300	3	10	6	20	2	10	6	75	250	1.18	2.74	300	300	4	12	4	10	6	95	195	0.85	3.46
H2	600	300	3	14	6	20	2	14	6	75	202	1.30	3.19	300	300	4	12	4	12	6	95	159	1.00	3.77
S8	600	300	3	10	6	20			8	150	255	1.18	2.81	300	300	4	12			8	150	263	0.50	2.81
S6_4LE	600	300	4	10	6	20			6	150	255	1.22	4.14	300	300	4	12			6	150	263	0.50	4.14
Supduct	600	300	3	10	6	20			6	50	150	1.18	4.30	300	300	4	12	4	12	6	50	150	1.00	5.88

To summarize, three different constructive features have been varied: the beam dimensions, the longitudinal reinforcement, and the transversal reinforcement, resulting in nine models:

- In the “Real” model, no changes have been applied: it has been designed with the same element dimensions and reinforcement ratios as the existing building.
- In the “RealMo” model, only the orientation of the beams has been changed, transforming the wide beams into deep beams.
- In the “M1” and “H1” models, the three aspects have been changed according to the DCM and DCH EC8 ductility classes, respectively.
- For the models “H2” and “M2”, the same applies, but only the reinforcement ratios have been modified while the beam sizes have not been changed.
- In the “S” models, only the transversal reinforcement ratio varies since it is mainly used to resist the shear stress produced by earthquakes. In “S8”, the diameter of the stirrups has been increased to 8mm. In “S6_4LE”, four legged stirrups of 6mm diameter have been used.
- The “Supduct” model has been designed with minimum separation of the stirrups and the longitudinal rebar, with the beam dimensions unchanged.

First, a pushover analysis has been carried out on one load-bearing frame (Fig. 6 (a)) of each of the nine models. The goal of this analysis is to obtain the relative influence of each aspect in the ductility value.

Considering the results of this phase, the most relevant models have been analysed by means of a similar pushover analysis, this time considering all the bays of the structure; i.e. the complete building. These models were the “Real” and “RealMo” and have been analysed in order to determine the global behaviour of the structures.

3.7. Construction costs and damage level determination

In this work, the construction costs (*C*) of each model have been determined. The costs have been estimated by means of a bill of quantities. Unlike the rest of studies of this type, a specific detail of the prices and the dimensions of each model has been performed. To do so, an updated Spanish construction cost database has been used (CYPE Ingenieros S.A.). This database contains the costs of the materials and the work units, taking into account the labour and indirect costs, the industrial benefit and the construction times.

The specific detail of the prices considering the characteristics of each of the models analysed are listed in Table 6. The volume of concrete and the weight of steel of each of the models has been measured and multiplied by the cubic meter price calculated considering the prices of the database (Column Cost €/m^3). The total cost column refers to the sum of the construction costs of the complete frame, considering the total number of beams and columns. It should be pointed out that the price obtained has been estimated assuming that the model is constructed from zero. This is due to the fact that the aim of this study is to analyse the influence of the different constructive features in the ductility value, not its retrofitting. By considering the construction costs, it is possible to check whether the configuration that best improves the ductility and the performance of the building can be also competitive in terms of costs. The goal is to determine the most profitable solution taking into account other aspects rather than just the performance. The results obtained from this study can be applied in the design of new buildings and in the analysis of the ductility.

Table 6. Specific detail of the prices considering the characteristics of each of the models.

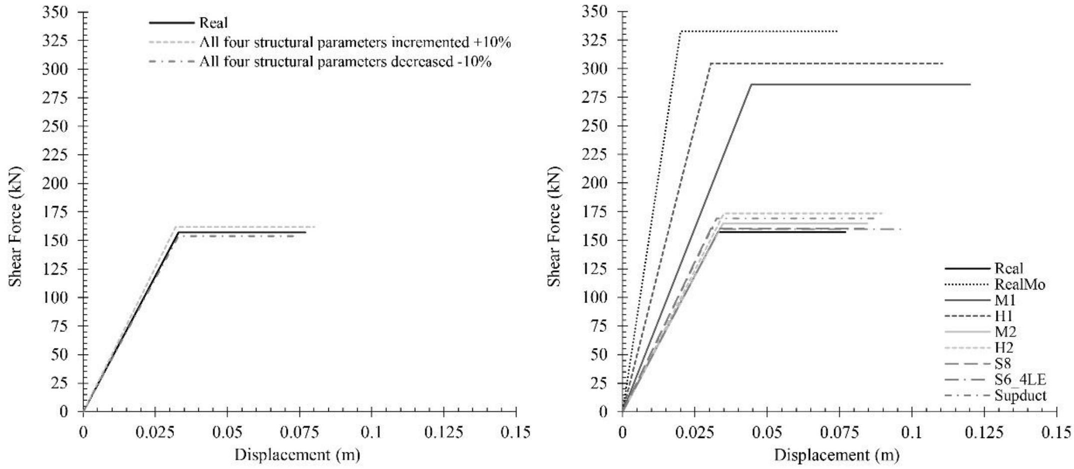
		Beams						**Columns**					**Total**
Models	Bay	Wstirrups (kg)	Wrebar (kg)	Vconcrete (m^3)	Cost €/m^3	Nº elements	Cost €	Wstirrups (kg)	Wrebar (kg)	Vconcrete (m^3)	Cost €/m^3	Cost €	Cost €
Real	Exterior	12.78	104.15	1.13	282.29	6	1920.70	3.60	9.52	0.243	382.2	603.67	2908.5
	Interior	5.96	41.33	0.45	284.55	3	384.14						
RealMo	Exterior	12.78	104.15	1.13	340.18	6	2314.60	3.60	9.52	0.243	382.2	603.67	3380.50
	Interior	5.96	41.33	0.45	342.44	3	462.29						
M1	Exterior	10.66	129.92	0.75	376.03	6	1705.67	5.66	16.13	0.243	419.20	662.14	2708.90
	Interior	5.01	51.55	0.30	379.06	3	341.08						
H1	Exterior	19.28	89.49	1.51	267.86	6	2430.02	6.17	19.04	0.243	433.90	685.26	3600.95
	Interior	8.67	35.51	0.60	269.81	3	485.65						
M2	Exterior	16.18	111.87	1.13	293.37	6	1996.08	5.66	16.15	0.243	419.20	662.14	3058.56
	Interior	7.66	44.39	0.45	296.54	3	400.32						
H2	Exterior	19.16	130.38	1.13	314.85	6	2142.24	6.17	19.04	0.243	354.90	560.56	3132.43
	Interior	8.94	51.74	0.45	318.24	3	429.62						
S8	Exterior	22.65	104.15	1.13	292.12	6	1978.58	6.37	9.52	0.243	394.00	622.38	3009.69
	Interior	10.57	41.33	0.45	296.09	3	399.72						
S6_4LE	Exterior	17.13	108.01	1.13	290.54	6	1976.83	3.60	9.52	0.243	382.20	603.66	2976.69
	Interior	7.99	42.86	0.45	293.47	3	396.18						
Supduct	Exterior	25.55	104.15	1.13	295.05	6	2007.52	9.26	19.04	0.243	447.1	706.11	3116.56
	Interior	11.5	41.33	0.45	298.46	3	402.92						

Regarding the damage level, a fragility curves approach has been followed according to (Estêvão 2019). Fragility curves provide the probability of reaching or exceeding a certain damage limit state (ds) given a determined spectral acceleration ($a_{g,k}$), or the correspondent spectral displacement (S_d). The probability of occurrence of each damage limit state has been determined according to the fragility curves (obtained considering the building behaviour and (Estêvão 2019)) and the performance point (obtained according to the EC8-1 approach). Once the probability of reaching each damage limit has been determined, the mean damage index (DI) has been obtained according to the procedure established in (Vargas et al. 2013). This approach considers the three limit states defined in the EC8-3 (near collapse, significant damage and damage limitation) and the new one to be included in the future EC8 generation. This limit is called operationality and it is included in the Italian Code NTC 2018 (Ministero delle infrastrutture e dei trasporti: Roma 2018).

There are numerous studies that proposed different DI approaches as in (Barbat et al. 2008). Unlike the rest of studies, the DI presented in (Vargas et al. 2013) is a very simple index which represents the total damage expected. It ranges between 0 and 4: DI=0 means that the probability of no-damage is equal to 1 while DI=4 indicates that a probability of complete damage state or collapse equals to 1. By calculating this DI, it is possible to compare the seismic performance of several models in terms of damage probability.

4. Results

The most relevant results obtained from the models analysed are shown in this section. In Fig. 7 (a), the idealized bilinear curves for the models considering the variability of the structural parameters values are shown, while Fig. 7 (b) shows the idealized bilinear curves for each model considering the real values of the structural parameters.



(a) (b)

Fig. 7. Idealized bilinear curves for: (a) the models considering the variability of the structural parameters values and (b) the models in the X direction.

Table 7 summarizes the μ-factor and the q-factor of all the models studied, highlighting the best q-factor results.

Table 7. Nonlinear parameters of the models.

Models	**Nonlinear parameters**							
	$\boldsymbol{\Delta_u^*}$ **(m)**	$\boldsymbol{\Delta_y^*}$ **(m)**	$\boldsymbol{\mu}$	$\boldsymbol{q_\mu}$	$\boldsymbol{V_y^*}$ **(kN)**	$\boldsymbol{V_d^*}$ **(kN)**	$\boldsymbol{q_\Omega}$	$\boldsymbol{q}$
Real	0.077	0.033	2.33	1.91	157.0	95.5	1.64	3.14
RealMo	0.075	0.020	3.75	2.54	332.5	178.5	1.86	4.74
M1	0.120	0.044	2.69	2.09	286.0	158.5	1.80	3.78
H1	0.110	0.030	3.62	2.49	304.5	155.0	1.96	4.90
M2	0.084	0.034	2.44	1.97	164.5	95.5	1.72	3.39
H2	0.089	0.035	2.55	2.02	173.5	95.0	1.82	3.7
S8	0.083	0.030	2.73	2.11	160.1	92.5	1.73	3.65
S6_4LE	0.096	0.033	2.86	2.17	159.8	91.5	1.74	3.79
Supduct	0.089	0.032	2.73	2.11	169.0	96.0	1.76	3.72

In Table 8, the construction costs and the enhancement percentage of each solution are listed, respectively. Three percentages have been determined. The ratios represent the construction costs, μ-factor and q-factor of the assessed solution in relation to the existing buildings. The costs are presented in €/frame.

Table 8. Construction costs and variation of costs, μ-factor and the q-factor with respect to the existing building and the ratio between the costs and the average μ-q.

Models	**Cost** (€)	$\boldsymbol{C - C_{real} / C_{real}}$ (%)	$\boldsymbol{\mu - \mu_{real} / \mu_{real}}$ (%)	$\boldsymbol{q - q_{real} / q_{real}}$ (%)	***Cost/Av (μ-q)*** €/%
Real	2908.51	-	-	-	-
RealMo	3380.55	16.23%	60.71%	50.86%	60.6
M1	2708.90	-6.86%	15.57%	20.14%	151.7
H1	3600.95	23.81%	55.27%	55.96%	64.7
M2	3058.56	5.16%	4.63%	7.82%	491.3
H2	3132.43	7.70%	9.59%	17.68%	229.7

S8	3009.69	3.48%	17.09%	16.17%	180.9
S6	2976.69	2.34%	22.81%	20.67%	136.9
Supduct	3116.56	7.15%	17.36%	18.32%	174.7

Based on the results of this phase, as discussed in the next section, the most relevant models ("Real" and "RealMo") have been further analysed in order to obtain the global behaviour of the structures. In Table 9, the idealized bilinear curves parameters and the variation ratios of the two models are shown.

Table 9. Results for complete buildings.

Models	Δ_u^* (m)	Δ_y^* (m)	μ	V_y^* (kN)	V_d (kN)	q	C/C_{real} (%)	μ/μ_{real} (%)	q/q_{real} (%)
Real	0.105	0.041	2.56	1645.5	795.0	4.20	-	-	-
RealMo	0.108	0.028	3.88	1842.5	855.0	5.26	16.23%	35.74%	25.11%

Table 10 enlists the damage probability and the DI for each building model.

Table 10. Damage probability for each building.

Damage / Models	**Non damage D1** (%)	**Slight D2** (%)	**Moderate D3** (%)	**Severe D4** (%)	**Collapse D5** (%)	**DI**
Real	0	0.43	32.3	23.07	44.2	3.10
RealMo	0.21	3.25	62.63	20.8	13.23	2.43

5. Analysis of the results

The sensitivity analysis performed on the variability of the material properties suggests that the behaviour of the models studied barely depends on them. A maximum variation of a 3% in the idealized bilinear curves has been observed, for variations of up to +/-10% in the material properties. Therefore, this variability has been considered negligible and only the mean values have been used in the analyses.

For each of the constructive feature analysed, different results have been obtained. Concerning the geometrical properties, as can be observed in Fig. 7 (b), the deep-beam models (M1, H1 and RealMo) have outperformed the rest of the models. They have also presented the highest values of ductility, as shown in the last column of Table 7. However, H1 has been the most expensive solution due to the increase in the beams' dimensions and reinforcement ratio. Models with wide beams (like the existing building) and enhanced reinforcement have merely shown a slight enhancement of the resistant capacity. The models designed with deep beams have presented higher variation ratios of μ-factor and q-factor as in (Gómez-Martínez et al. 2016b) and as shown in Table 7. Deep-beam models have reached up to 60% of ductility improvement compared to the existing building. Nevertheless, they have had the highest construction cost ratios (Table 8). This is due to the higher reinforcement ratios needed to comply with the code provisions and the beam's volume. Still, the cost increase has been moderate (16% and 24%, respectively), especially considering the enhancements achieved.

Conversely, models where only the reinforcement ratios have been varied have presented relatively small variation ratios compared to deep-beam models. These enhancements have ranged from 5% to 10% in the case of the μ-factor and 8% to 18% for the q-factor. The construction cost ratios have been below 7%. The models with higher ratios (H2 and Supduct) have presented higher values of resisted shear forces. The S8 and the Supduct models have also enhanced their initial stiffness. The results obtained from the nonlinear static analyses have been similar to those in (Vielma et al. 2010), despite the different approach followed by the authors to define the bilinear curves. Similar results were obtained by (Lu et al. 2001) when assessing three

EC8-designed simple frames subjected to earthquake simulations. The authors concluded that the amount of confining reinforcement at the critical regions of columns improved the local behaviour. However, the overall ductility of frames was improved only slightly.

Yet, modifying the transversal reinforcement by adding four legged stirrups has caused a considerable enhancement of the μ-factor and the q-factor up to 23% and 21%, respectively. This is due to the reduction of the distance between consecutive longitudinal rebar engaged by stirrups. This solution has presented almost no increase in costs compared to the existing building.

The ratio between the costs and the average μ-q have shown that the best models regarding the costs are the RealMo and the H1. Contrariwise, the worst results have been obtained with the H2 and the S8 models. Based on these results, the RealMo model has been selected as the best alternative to the existing building. This is due to the combination of the great improvement of the μ-factor and q-factor, the simplicity and feasibility of the solution and its relatively low increase in costs. The Real and RealMo models have been analysed considering all the bays of the structure. The aim of these analyses is to study accurately the influence of varying the beam orientations.

In the complete building analyses, the μ-factor and q-factor have been higher than the results obtained for the single frame. This is due to the shear resistant capacity of the tie beams. The damage level expected for the existing building has been severe (DI>3). Conversely, the RealMo model has caused a reduction of the damage level of up to 28% compared to the existing building. In this model, wide beams have been changed to deep beams, so this reduction can be further improved by also increasing the reinforcement ratio.

6. Conclusions

This paper aims to analyse the ductility of existing buildings to evaluate its influence in their seismic performance. This has been achieved through the retrospective analysis of an existing building in accordance with current seismic codes.

The study has concluded that the NCSE02 and the EC8 share similar considerations concerning the ductile capacity of new-designed buildings. However, each code establishes different procedures and factors to determine this capacity i.e. μ-factor in the NCSE02 and q-factor in the EC8, respectively. This study has revealed the lack of guidance in the NCSE02 and the EC8 regarding the assessment of the ductility of existing buildings. Although EC8-3 points out the importance of analysing their seismic behaviour, no ductility considerations are taken into account. This manuscript proposes a methodology to assess the ductility of Spanish existing buildings.

In this study, a pushover analysis has been carried out first in one load-bearing frame of each of nine different models. In these models, three different constructive features have been varied: the beam dimensions (wide and deep beams), the longitudinal reinforcement and the transversal reinforcement. The variability of the structural parameters has been analysed by modifying their values. The results have not considerably differed from those considering the real values. Therefore, this variability has been considered negligible and only the real structural values have been used in the analyses carried out.

It can be concluded that the best performance, regarding the ductility, has been obtained with the models designed with deep beams (RealMo and H1). It has also been demonstrated that these are also the best models when considering the costs. Conversely, models with wide beams, and where only the reinforcement ratios have been varied, have merely shown a slight enhancement of the resistant capacity. Still, the models with higher reinforcement ratios have presented higher values of resisted shear forces. Similarly, these models have shown relatively small improvements of μ-

factor and q-factor compared to deep-beam models. Yet, the addition of four legged stirrups has brought a considerable enhancement of these factors. This is due to the reduction of the distance between consecutive longitudinal rebar engaged by stirrups.

Based on the results of this first phase, the most relevant models have been analysed by means of a similar pushover analysis, this time considering all the bays of the structure. The models analysed have been the existing building and the model where only the orientation of the beams has been changed (wide vs. deep beams). In these analyses, the μ-factor and q-factor have been higher than the ones obtained for the single frame. This is due to the additional shear resistant capacity of the tie beams. The expected damage has been severe for the existing building and moderate for the deep beam model, respectively. Therefore, it can be concluded that, for a minimum increase in cost, buildings using deep beams achieve an important enhancement in their seismic behaviour.

This study has concluded that ductility affects the shear resistant capacity, and therefore, the seismic performance and the expected damage of RC buildings. Hence, the ductility assessment of these buildings must be performed thoroughly in order to propose appropriate seismic retrofitting solutions.

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