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COMPARISON AMONG DESIGN SOLUTIONS FOR PRECAST BUILDINGS IN MEDIUM-TO-MODERATE SEISMICITY ZONES

J.I. Eskubi[(1)], F. López-Almansa[(2)]

[(1)] *Assistant Professor, International University of Catalonia, Architecture Department, Barcelona, jieskubi@uic.es*
[(2)] *Professor, Technical University of Catalonia, Architecture Technology Department, Barcelona, francesc.lopez-almansa@upc.edu; currently Associate Researcher Natural and Anthropogenic Risks Research Center, Univ. Austral de Chile, Valdivia*

Abstract

This paper compares, mainly in terms of construction cost and rapidity, two seismic-resisting systems for low-to-midrise precast buildings in moderate-to-medium seismicity zones (in Spain): (i) ordinary buildings accounting for the cooperation of infill and cladding walls, and (ii) use of additional dampers. Precast buildings are popular in Spain (even in seismic zones) mainly because of their construction rapidity despite they are costlier than cast-in-situ buildings. Conversely, they are not suitable for seismic resistance (high self-weight, low lateral strength and stiffness, low ductility, not monolithic, and low inherent damping). Nowadays, the current Spanish code apparently endorses the lack of seismic consideration in ordinary importance precast buildings not exceeding seven stories and located in low seismicity zones. However, neither the European regulation nor the forthcoming new Spanish code contain any such prescription; therefore, the new situation might generate misunderstandings and conflicts. This underlines the opportunity of this study; it consists of 4 rather consecutive stages:

- Selection of 3 prototype buildings. Building # 1 is a 3-story school building, # 2 is a 2-story fire-station building, and # 3 is a single-story industrial building.
- Analysis of energy dissipators for precast buildings. Hysteretic devices are preferred because of their reliability, simplicity, robustness, and low maintenance requirements.
- Formulation of energy-based design criteria. Precast buildings with masonry walls or concentric steel bracing can be code-type designed; conversely, more tailored formulations are needed for buildings with additional dampers. Design energy spectra derived after Spanish records will be mainly used.
- Parametric study. This study consists essentially in designing buildings equipped with the aforementioned protection systems, under varying situations. The parameters of the study are the PGA (0.04 g, 0.12 g, 0.24 g), the soil type (B and D, according to the European classification), and (ii) the seismic system (ordinary buildings, additional dampers). The performance indices are the construction cost and rapidity.

Keywords: precast buildings; energy-based seismic design criteria; energy dissipators; parametric study.

1. Introduction

Precast buildings are widely used in Spain, even in seismic zones. They are popular mainly because of their high uniform quality and construction rapidity; these major advantages commonly compensate their higher cost (compared to cast-in-situ buildings, mainly). However, other of their circumstances make them not very suitable for earthquake conditions: high self-weight, low lateral strength and stiffness (mainly because of the difficulty of obtaining rigid connections), low ductility, rather poor monolithism, and low inherent damping. Nowadays, some prescriptions of the current Spanish seismic design code are routinely erroneously pleaded to justify the lack of seismic consideration in ordinary importance precast buildings not exceeding seven stories and located in zones with design seismic acceleration less than 0.08 g. Conversely, neither the European regulation nor the forthcoming new Spanish code contain any such prescription; therefore, the new situation might generate relevant confusion and controversy. This highlights the opportunity of carrying out this study.

Given the aforementioned seismic limitations of precast buildings, their seismic performance is of big concern [1]. Regarding this issue, in Europe, a number of Research Projects have been undertaken: ASSOBETON [2], ECOLEADER [3], PRECAST [4], SAFECAST [5], SAFECLADDING [6] and LESSLOSS [7]. All these projects were sponsored by companies and associations of precast producers in the most seismic countries in Europe; this proves their interest in this subject. In other countries, similar research was also performed; the USA-Japan coordinated Precast Seismic Structural Systems (PRESSS) program [8] is of particular interest.

The goal of this paper is to compare, mainly in terms of construction cost and rapidity, different solutions for low-to-midrise precast buildings in moderate-to-medium seismicity zones; as discussed previously, this study focuses in Spain. Two situations are contemplated: (i) the most common condition in actual precast buildings in Spain (in other words, ordinary buildings), and (ii) incorporation of energy dissipators. Other possible solutions are disregarded; among them, base (seismic) isolation and cast-in-situ strengthened beam-column connections. Base isolation is difficult to apply to the standard Spanish precast buildings, and the cast-in-situ connections are not common in Spain, as being rather expensive and impairing the construction rapidity.

The conducted research consists of four rather consecutive stages:

1. **Selection of a number of representative prototype buildings**. Aiming to obtain the maximum representativeness, three prototype buildings are selected. **Building # 1** is an actual 3-story school building. **Building # 2** is a 2-story fire-station building. **Building # 3** is a single-story industrial building devoted either to production or storage.

2. **Analysis of energy dissipators for seismic protection of precast buildings**. Since nowadays energy dissipators are not as common, their implementation in the prototype buildings will be studied. Among the available devices, the hysteretic ones (based on plastification of metals) will be preferred; are chosen because of their reliability, simplicity, robustness, low cost, and low maintenance requirements. Regarding their installation in the building, two major options are foreseen: (i) connection with the concentric steel braces (to take profit of the drift motion generated during the seismic shake), and (ii) connection between two adjacent concrete members (to take profit of the relative rotation angles that arise during the input duration).

3. **Formulation of energy-based design criteria**. The design of precast buildings with masonry walls or concentric steel bracing can be carried out following conventional code-type approaches; conversely, these strategies are not well suited for buildings equipped with energy dissipative devices. For that reason, more tailored formulations need to be investigated. Broadly speaking, design energy spectra derived after Spanish records will be mainly used to characterize the severity of the seismic action; they are considered more adequate than conventional acceleration or displacement spectra (given the well-known advantages of energy spectra). Depending on the obtained results, the derivation of new formulations that are better suited to the particular analyzed situations is going to be considered. The adequacy of the proposed solutions will be checked by following the current and new American

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regulations; depending on their availability, the forthcoming European regulations (2020) will be also contemplated.

4. **Parametric study**. This study constitutes the core of the research. It consists essentially in designing buildings equipped with the aforementioned protection systems, under varying situations. The major parameters of the study are: (i) Peak Ground Acceleration (PGA, corresponding to rock and 475 years return period) equal to 0.04 g, 0.12 g, 0.24 g, (ii) soil type (B and D, according to the European classification), and (iii) earthquake-resistant system (ordinary buildings, additional dampers). The performance indices that will be used to assess the quality of the solution are the construction cost and rapidity.

2. Prototype buildings

2.1. Buildings description

As discussed in the Introduction, three prototype buildings are selected to represent the vast majority of the existing precast buildings in moderate-to-medium seismicity zones of Spain; they are termed along this study as # 1, # 2 and # 3, respectively. The main features of the selected prototype buildings are described next.

- **Building # 1**. This is a regular 3-story school building. This type of building is intended to represent the following main characteristics: (i) uniform columns layout (regular orthogonal pattern), (ii) moderate span-length (rather alike in both directions), and (iii) moderate and rather uniform story height. This building, being rather long (near 70 m) is split into two main volumes that are separated by a bi-directional expansion joint at each story (Fig. 1.a). This link can behave as a seismic joint and guarantees a full structural independence between both volumes; thus, only the right volume (Fig. 1) is analyzed in this study. Noticeably, the slabs of this volume have no openings (e.g. stair cases). As usual in most of the actual buildings, the cladding is made of thin GRC (Glass-Fiber Reinforced Concrete) panels; these elements can either coplanar with the outer columns or external to them. In this last case, structural connection is provided; the ability of such connections to transmit the seismic forces is going to be studied. The façades have many openings (windows), being rather regularly distributed.
- **Building # 2**. This is a 2-story fire-station building. This type of building represents the following main characteristics: (i) orthogonal columns distribution but with different span-lengths in both directions, and (ii) atria (double-height spaces). The cladding consists of ordinary precast concrete walls placed outside the outer columns, although connected to them. There are less windows than in Building # 1, with rather uneven distribution.
- **Building # 3**. This is a single-story industrial building, either for storage or production. This type of building represents the following main characteristics: (i) there are no inner columns (just in the façades), (ii) large span-length (above 20 m) in the transverse direction and medium (about 10 m) in the longitudinal one, and (iii) rather high columns. Two types of cladding are considered: infill unreinforced masonry clay brick or concrete block walls, and precast concrete walls (analogously to Building # 2). There are few, small and regularly distributed windows.

Fig. 1, Fig. 2 and Fig. 3 contain sketches and images of Building # 1, # 2 and # 3, respectively. Fig. 1.a presents a plan view where the analyzed volume is highlighted in yellow and the joint is represented as a blue line, Fig. 1.b displays a picture taken during the construction stage. Fig. 2.a includes an elevation (left) and a plan view (right); as in Fig. 1.a, the building plan is highlighted in yellow. Fig. 2.b presents a picture of the building interior. Fig. 3.a contains a plan view where the analyzed volume is highlighted in yellow and the joints rare represented as a blue line. Finally, Fig. 3.b presents an interior picture.

Fig. 1, Fig. 2 and Fig. 3 show that the structures of the buildings are composed of continuous precast columns (not prestressed) being clamped to the foundation (precast spread footings). The slabs are made of hollow core prestressed slabs resting on precast prestressed beams, and are topped with a cast-in-place reinforced concrete layer. The roof is made of light sandwich panels. Details of the cladding and the infill walls can be also seen.

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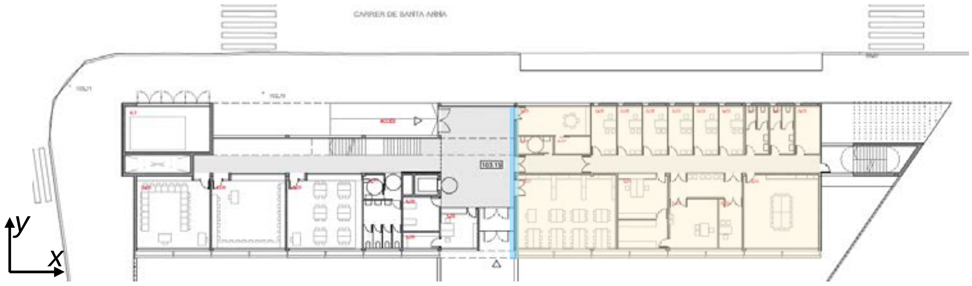
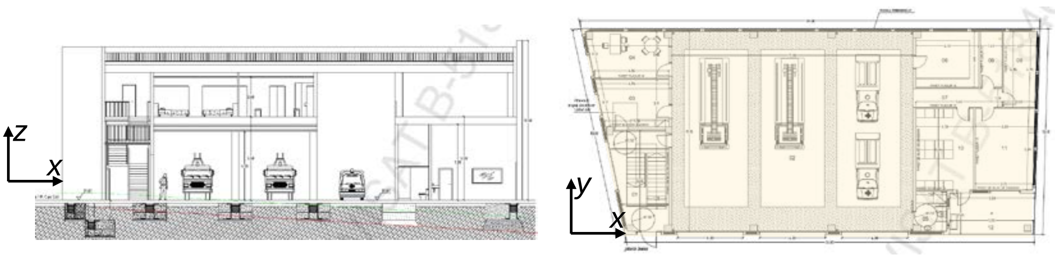


(a) Plan view

(b) Global view during the construction stage

Fig. 1. Building # 1 (3-story school building)



(a) Elevation (left) and plan view (right)

(b) Interior view during the construction stage

Fig. 2. Building # 2 (2-story fire-station building)

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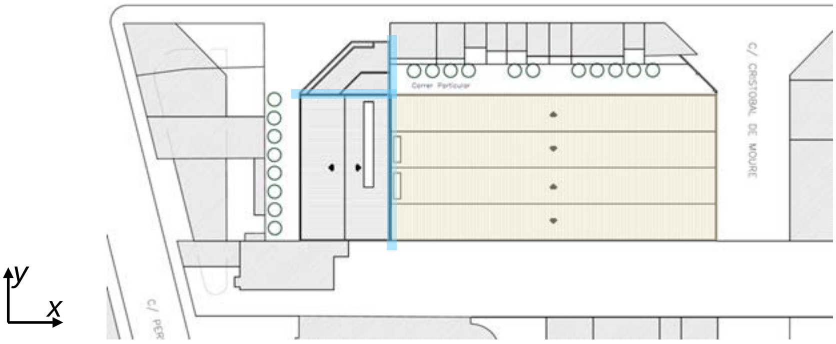

(a) Plan view

(b) Building # 3

Fig. 3. Building # 3 (single-story industrial building)

Noticeably, in Fig. 1, Fig. 2 and Fig. 3, the directions indicated by the axes (*x*, *y* and *z*) are maintained along the full paper.

In Spain, the precast sector begun by industrial (or farm) buildings, such as # 3; then, other types of buildings were also considered, being multistory and less regular, as # 2. More recently, this sector has expanded to conventional multistory buildings (# 1); the use can be teaching, housing, commercial, administrative, hospital, hotel, among others. These considerations highlight the representativeness and interest of the selected prototype buildings.

Table 1 – Main parameters of the selected prototype buildings

Building	**Stories / height (m)**	**Plan area (m)**	**Span-length (m)**	**Columns (cm)**	**Floor slab**		**Roof slab**	
					Beams (cm)	**Slabs (cm)**	**Beams (cm)**	**Slabs (cm)**
# 1	3 / 9.60	13.80 × 28	7 (*x*) × 6.40–7.40 (*y*)	40 × 40	40 × 60 (prestressed)	Hollow core 20 + 5 (prestressed)	40 × 60 (prestressed)	Hollow core 20 + 5 (prestressed)
# 2	2 / 9	29 × 14	5 (*x*) × 13 (*y*)	40 × 40 40 × 50	40 × 70 (prestressed)	Hollow core 40 + 8 (prestressed)	50 × 50-120 (prestressed)	15 × 25 (prestressed) + sandwich panel
# 3	1 / 10	90 × 40	10 (*x*) × 20 (*y*)	50 × 50	-	-	25-35 × 60-160 (prestressed)	14 × 26 (prestressed) + sandwich panel

Table 1 displays the most important geometrical, constructive and architectonic properties of the three selected prototype buildings. Noticeably, the parameters in Table 1 correspond to buildings designed without any seismic consideration, i.e. accounting only for gravity and wind forces.

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2.2. Buildings modelling

The static behavior of the buildings is described with structural models that are implemented in the ETABS v. 18.1 software code [9]. The structural behavior is assumed to be linear, and the nonlinearities are concentrated in the energy dissipators; therefore, the material constitutive laws are linear elastic, and only first-order analyses are performed.

The columns, hollow core slabs and beams are described by frame elements; their stiffness correspond to sectional gross properties (i.e. it is not reduced due to tensioned concrete cracking); this consideration is based on the rather low seismicity in Spain and the fact that many elements are prestressed (in future stages of this research, the stiffness reduction due to cracking will be accounted for). The steel braces are modelled as pin-ended, and the beam-column connections are described as fully hinged (i.e. the columns behave as vertical cantilevers clamped to the foundation); in other words, the stiffening effects due to the dowel effect are not considered. The soil is assumed to be infinitely rigid; in other words, the SSI (Soil-Structure Interaction) is not accounted for (in future stages of this research, it will). Regarding the material mechanical parameters, for all the buildings, in the columns, the characteristic value of the concrete compressive strength is f_{ck} = 40 MPa, in the beams it is f_{ck} = 50 MPa, and in the hollow core slab, is f_{ck} = 45 MPa; in all the elements, the reinforcement steel yield point is f_{yk} = 500 MPa.

Fig. 4, Fig. 5 and Fig. 6 display general views of the structural models of Building # 1, # 2 and # 3, respectively. Each Figure presents also the configuration of meaningful eigenmodes; the information on such modes is presented in Table 2 and, mainly, in Table 3.

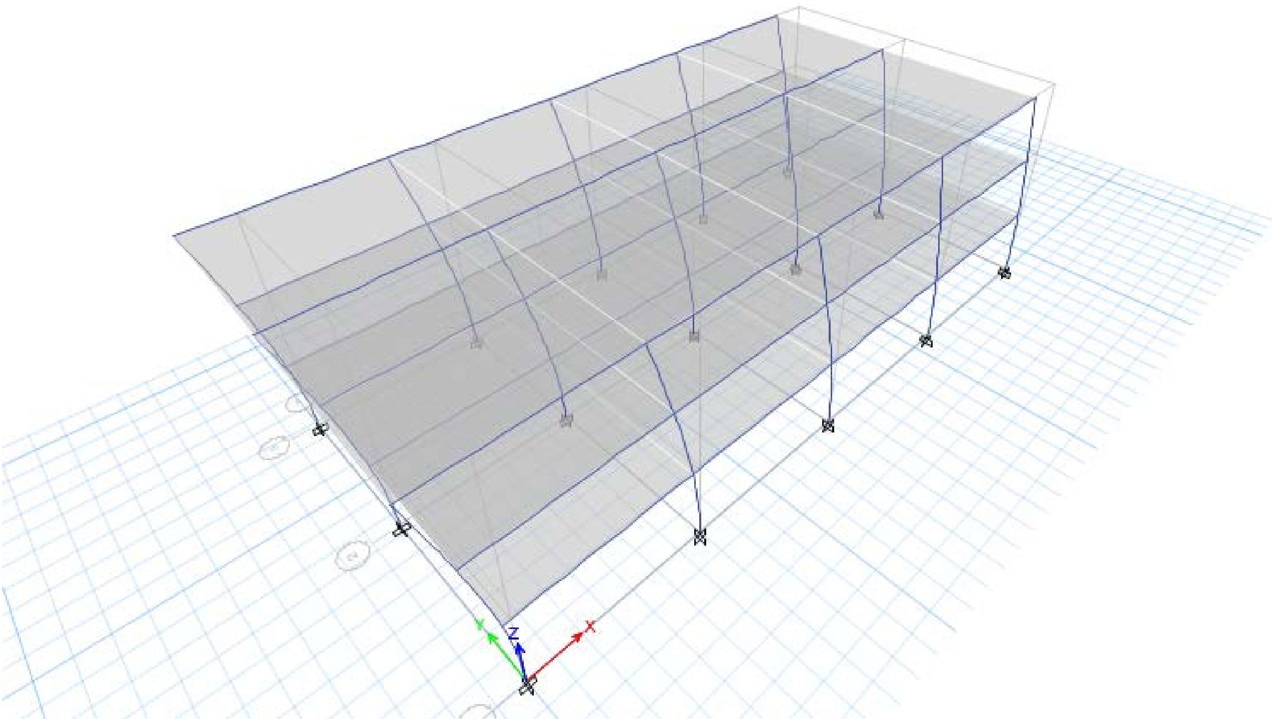

Fig. 4. Structural model of the Building # 1 (first mode, Table 2)

Fig. 4 shows that the first mode corresponds mainly to motion in the y direction, as confirmed by Table 2 and Table 3. Fig. 4 also shows that there is no relevant torsion and that the rigid diaphragm effect is clearly fulfilled in all the stories.

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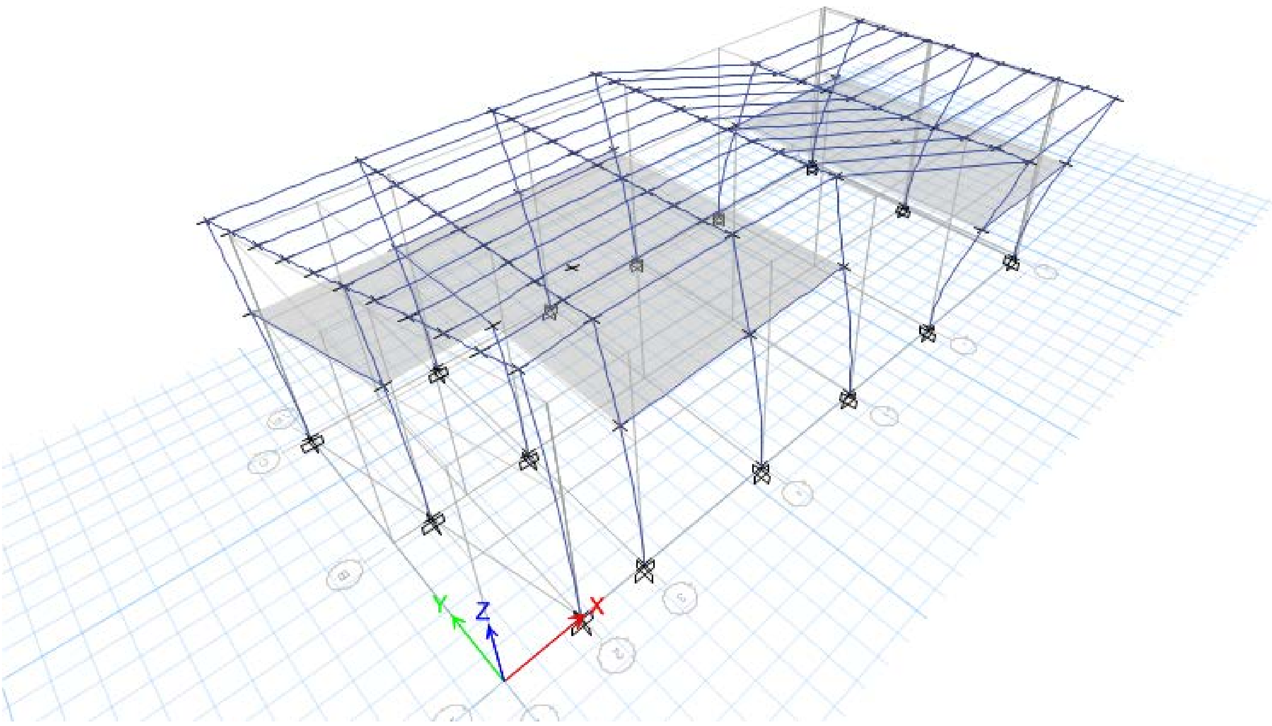

Fig. 5. Structural model of the Building # 2 (second mode, Table 2)

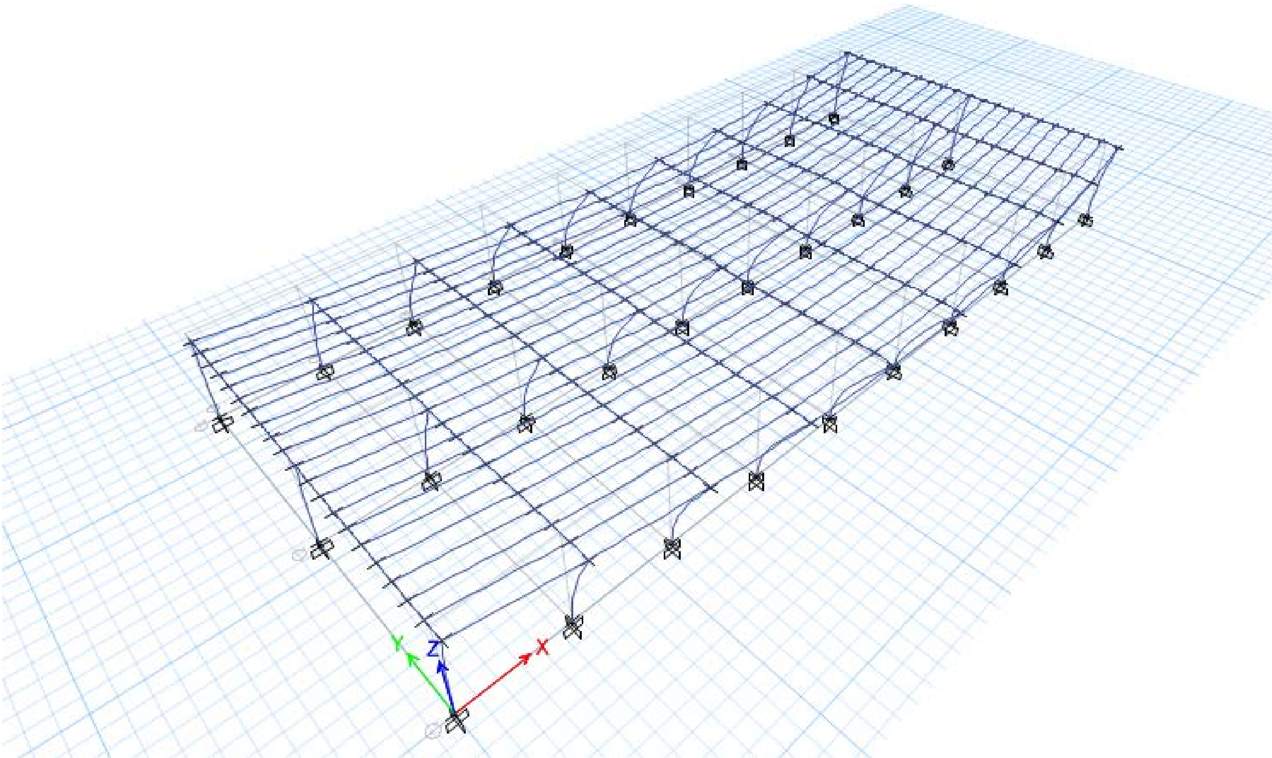

Fig. 6. Structural model of the Building # 3 (third mode, Table 2)

Fig. 5 shows that the second mode corresponds mainly to motion in the *y* direction. This is in apparent contradiction with Table 3, as this mode corresponds roughly to torsion; however, a closer view to Fig. 5 points out that this mode involves *y* displacements of the two slabs in opposite directions (thus cancelling themselves

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and generating little global *y* displacement). On the other hand, the roof deformation highlights that the rigid diaphragm effect is not fulfilled at all; in the slab, neither it is, given that their two parts are not connected (Fig. 2). Fig. 6 illustrates that the third mode includes mainly displacement in the *y* direction; this fits Table 3. The roof deformation highlights that the rigid diaphragm effect is not satisfied (Fig. 3).

Table 2 displays the most meaningful mechanical parameters of the three prototype buildings. Mass eccentricity refers to the distance between the centers of mass (gravity) and rotation (stiffness); for the multistory buildings (# 1 and 2), it corresponds to the bottom level (first story). The seismic weight correspond to the permanent (dead) load and a percentage of the variable (live) load; such percentage is stated as 50% for all the buildings.

Table 2 – Main structural parameters of the selected prototype buildings

Building	Mass eccentricity (m)		Seismic weight (kN)
	x direction	*y* direction	
# 1	-	0.15	10750
# 2	1.54	0.25	5410
# 3	-	-	5980

Table 3 displays the modal parameters (natural periods and modal masses) of the three prototype buildings; the first twelve modes are included. The natural periods and the modal masses refer to the "naked" buildings, i.e. without accounting for the contribution of the infill walls or the cladding elements; moreover, the beam-column connections are assumed to be completely hinged. In Table 3, *x* and *y* correspond to the horizontal directions (Fig. 4, Fig. 5 and Fig. 6) and φ refers to twist (torsion).

Table 3 – Modal parameters of the Buildings # 1, # 2 and # 3

Mode	Building # 1				Building # 2				Building # 3			
	Period (s)	Modal mass (%)			Period (s)	Modal mass (%)			Period (s)	Modal mass (%)		
		x	*y*	φ		*x*	*y*	φ		*x*	*y*	φ
1	2.37	73	-	-	1.25	-	85	3	1.61	59	-	-
2	2.37	-	73	-	1.12	-	2	31	1.46	-	-	7
3	1.86	-	-	73	0.98	89	-	0	1.14	-	89	-
4	0.33	21	-	-	0.75	-	1	56	1.11	-	-	59
5	0.33	-	21	-	0.34	-	1	1	1.06	-	7	-
6	0.29	-	-	21	0.26	-	2	1	1.02	-	-	10
7	0.13	6	-	-	0.26	-	-	-	1.00	1	-	-
8	0.13	-	6	-	0.24	-	6	-	0.97	-	2	-
9	0.11	-	-	6	0.23	-	1	1	0.93	-	-	3
10	-	-	-	-	0.23	1	1	1	0.90	-	1	-
11	-	-	-	-	0.19	1	-	2	0.88	-	-	1
12	-	-	-	-	0.19	7	-	-	0.86	-	-	15

Table 3 shows that the periods are extremely long; this is due to the low lateral stiffness, given that the columns behave basically as cantilevers. Moreover, Table 3 provides the following specific remarks for the three analyzed buildings.

- **Building # 1**. The displayed results reveal a largely regular behavior, with equal periods in both directions. 100% of the building mass is mobilized by the first nine modes.
- **Building # 2**. The behavior is less regular. However, the first four modes are quite regular, gathering most of the translational and rotational masses.
- **Building # 3**. The behavior is highly irregular. The first twelve modes gather only about 60% of the mass in *x* direction; this circumstance might be due to the high in-plane roof flexibility (Fig. 6).

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3. Energy dissipators for seismic protection of precast buildings

As discussed in Section 1, the hysteretic dissipative devices are going to be considered; such elements are characterized by their energy dissipation capacity being based on plastification of metals (commonly, ordinary construction steel). These dampers are chosen as being cheap, robust, simple, dependable, and easy to manipulate and maintain; other types of devices (viscous, visco-elastic, friction, shape memory alloys, magneto-rheological, among others) might provide some advantages, but at a significantly higher cost and other requirements.

The dissipators can be connected to the building to be protected mainly in two different ways: to concentric steel braces (diagonal or chevron) [10], or to beam-column joints [7,11-15]. In the first case, the energy dissipation will be generated through the drift displacements, and in the second case it will be produced by the relative rotation between both connected members. Obviously, this last option allows higher architectural flexibility, although with less energy dissipation capacity. On the other hand, some studies [16]. Finally, the work [7] discusses the applications of additional dampers (energy dissipators) for seismic protection of precast buildings, particularly the INERD Pin Connection [17].

4. Seismic design criteria

4.1. Spanish design codes

At the time of writing (early 2020), the old Spanish regulation [18] is still enforced, despite having major limitations, and being clearly outdated. Regarding the flaws of such code that are relevant to this research, clause 1.2.3 states that any construction with frames in two directions (well connected to each other) does not need any seismic provision in areas whose basic seismic acceleration (a_b) does not exceed 0.08 g; this parameter represents the expected acceleration in stiff soil for 500 years return period. This statement is routinely (and incorrectly) pleaded to waive the need for seismic design of precast buildings in moderate seismicity regions of Spain. Concerning the lack of updating of [18], the recent Lorca earthquake (11-05-2011) triggered the review of the Spanish seismicity map, leading to relevant general and local increases. Moreover, this augmentation was accompanied with a change in the reference parameter: a_b (stiff soil, 500 years return period) was replaced with a_g (rock, 475 years); noticeably, this last parameter is fitted to the international standards, particularly the European ones [19]. Concerning the relation between a_b and a_g, the NAD (National Application Document) for Spain [20] states that such conversion should be made as 0.8 $a_b = a_g$. Nowadays, the legal status of the seismic design regulations in Spain is unclear, as the old Spanish code [18] coexists with the more recent (and significantly better) European ones [19-20]. To sum up, the Spanish code is less demanding (both in terms of seismicity and of requirements for precast buildings) than the European documents; the new expected Spanish regulation will presumably fit them. The main output of this context is that the enforcement of the European codes will generate relevant controversy regarding the seismic design of precast buildings; this is the origin of this research. Aiming to bring some clarification, the seismic designs will be conducted according to the aforementioned current European regulations; given that such documents do not discuss deeply the seismic design of buildings incorporating supplemental dissipative devices, the American documents [21] and the forthcoming European codes will be used instead. Subsection 4.2 discusses this issue.

4.2. Energy-based design criteria

Once the type of additional dampers has been chosen, the seismic analysis and design of buildings incorporating such devices consists of the following rather consecutive steps: (i) characterization of the site seismicity in terms of demanding energy spectra, (ii) initial selection of the stiffness and damping parameters of the dampers, and (iii) verification of the adequate dissipative capacity of the damping system. Although these steps are rather consecutive, the last two shall be performed iteratively until reaching a fully satisfactory performance.

The strategy in each of these steps is described next.

- **Seismicity**. The seismic demand is going to be characterized in terms of input hysteretic and energy spectra;

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such spectra had been previously obtained for Spain by some of the authors [22-23]. These spectra will be updated to incorporate the last recorded accelerograms, particularly those of the Lorca earthquake. After this information, the global design criterion is the fulfillment of the following inequality:

$$E_D \geq E_H \tag{1}$$

In equation (1), E_D is the energy dissipation capacity of the building equipped with the additional dampers and E_H is the demanding hysteretic energy (contributable to damage); both quantities are expressed in terms of equivalent velocity.

- **Initial design of the dampers**. This task consists basically in selecting the variation of the devices parameters along the building height. Initially, the simplified formulation in [24] will be employed to define the variation of the strength parameter (basically, the yield force); then, the more advanced approach in [25] will be utilized to better establish such variation and to define the one of the other stiffness and damping parameters. The objective of this process is to obtain a rather uniform distribution of the energy absorption along the building height. Obviously, this operation is not necessary in the Building # 3, given that it has only one story.
- **Seismic verification**. The suitability of the solution derived in the previous step is going to be checked according to the indications of Chapter 18 of [21]; the most simplified method (i.e. equivalent lateral analysis) will be employed. Given that this formulation is rather confusing (as the other ones in such document), the prescriptions of the forthcoming European regulations are to be considered.

5. Preliminary results

This section presents preliminary results of the code-type seismic design of the prototype Building # 1 (Section 2); the performed analysis is based on the Spanish NAD (National Application Document) of the European regulation [20]. The most demanding seismic situation is considered: soil D (Section 1) [19], and highest seismic zone in Spain (Granada, a_{gR} = 0.24 g [20]). The most simplified strategy (Lateral Force Method [19], corresponding to equivalent static analysis considering merely the contribution of a mode) is employed.

The ductility behavior factor (q, equivalent to R in the American regulations) is taken as 1.5, this being the lowest value according to the European regulation [19] for highly brittle constructions. The building fundamental period is hard to estimate, as the values in Table 3 have been derived after extremely conservative assumptions; 1 s is initially assumed. Finally, the importance factor is γ_I = 1.3, corresponding to importance class III [20]. Type 2 spectra for linear analysis (S_d) are considered [19]; the soil factor is S = 1.8, and the characteristic periods are T_B = 0.10 s (left corner period of the plateau), T_C = 0.30 s (right corner period of the plateau), and T_D = 1.20 s (long period transition). Regarding damping, its influence is not directly considered in the S_d spectra, but indirectly through the q coefficient. Then, the design base shear force in any horizontal direction is given by

$$F_b = S_d(T_1)\, m\, \lambda = 0.2808 \times 10750 \times 1 = 3019 \text{ kN} \tag{2}$$

In equation (2), T_1 is the building fundamental period, m is the building seismic mass, and λ is a dimensionless correction factor ranging between 0.85 and 1 according to the ratio between T_1 and T_C. Noticeably, the estimated period lies in the descending branch of the spectrum, thus showing a rather high sensitivity to the fundamental period estimation.

The force F_b in equation (2) is distributed among the building stories proportionally to their masses and heights; the resulting values are F_1 = 511 kN (1st floor), F_2 = 1023 kN (2nd floor), and F_3 = 1485 kN (3rd floor).

The building structural analysis shows that their columns lack totally the capacity to resist the demanding internal forces, particularly, the interaction between axial force and bending moment (the required reinforcement amount would largely exceed the maximum permitted [19]). Noticeably, the cooperation of the cladding elements and of the slight rigidity of the beam-column connections, might prove highly relevant.

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6. Conclusions

This paper presents the beginning steps of a numerical parametric study on precast buildings in medium-to-moderate seismicity zones of Spain. Three prototype buildings have been selected to represent the vast majority of the built environment: 3-story school, 2-story fire-station, and a single-storey warehouse. Preliminary results seem to indicate that the structural behaviour of the first building is regular, while the second and third ones exhibit several irregularities, possibly leading to seismic-resistant deficiencies. Apparently, the 3-story school building does not have the capacity to withstand the design seismic forces corresponding to the most demanding seismic conditions in Spain (soft soil in a medium seismicity area, near Granada). Therefore, not considering any seismic action in this kind of precast structures (as is nowadays customary in Spain) is apparently an extremely dangerous practice.

7. Acknowledgements

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