Assessment of Existing RC Precast Industrial Buildings According with Eurocode 8 - Part 3

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Abstract

Precast reinforced concrete (PRC) buildings are common in the Portuguese industrial park, as well as throughout Europe. In past earthquakes, namely in Italy and Turkey, this typology of buildings showed a poor performance, namely at structural level. One of the major concerns at structural level regards the connection between the beams and columns. In recent surveys at the Portuguese industrial buildings, it was observed that the buildings built before 1980 present, most of the times, only friction connections between beam and columns. This type of connection is pointed as one of the weakest structural system in this typology of structures and consequently led to serious local and global damage when subjected to seismic loads. Regarding this issue, it was considered appropriate and necessary the study of existing Portuguese buildings with this type of beam-to-column connection and compare it with the use of mechanical connections (dowels), which is a more common solution in the recent buildings. The seismic behaviour of two PRC buildings built with these two solutions was analysed considering nonlinear static and dynamic analyses addressing both global and local (connections) response parameters. When analysed in view of the seismic regulation for existing buildings (Eurocode 8 –Part 3), the results obtained show that, overall, this typology of buildings present an acceptable structural performance. Nonetheless, unsatisfactory performance was observed at the beam-to-column connection in one of the buildings built without steel dowels. The results presented in this study highlight the need to consider adequate models to simulate these connections in order to accurately represent the seismic behaviour on the structure and identify possible limitations in the use of nonlinear static procedures to assess the seismic performance of this typology of buildings.

In general, the buildings in study, located in a region of moderate seismicity, exhibited a satisfactory behaviour. However particular attention should be paid to beam-to-column connections, especially in the case of friction connections. In this regard it is important to accurately represent in the model the mechanisms involved in the connections, particularly in buildings located in moderate to high seismic zones.

Keywords: Precast, Reinforced concrete, Industrial buildings, Seismic assessment.

Introduction

The damage caused by recent earthquakes in structural and non-structural elements of precast reinforced concrete (PRC) industrial buildings, exposed the vulnerability of this type of structures, in particular the ones design without seismic provisions (Batalha, Rodrigues and Varum 2019; Belleri et al. 2015; Liberatore et al. 2013; Magliulo et al. 2014). According to Bournas *et al.* (2013), after the Emilia Romagna earthquake in 2012, more than half of the existing precast structures exhibited significant damages. The seismic performance of precast structures is largely governed by the behaviour of the elements that guarantee the connection between structural elements and between these and non-structural elements (Batalha, Rodrigues and Varum 2019; Bournas, Negro and Taucer 2013; Magliulo et al. 2014). The performance of the beam-to-column connections, which are typically ensured by pure friction or friction and mechanical connections (dowels), represents one of the most critical aspects especially in the absence of dowels and deficient seismic detail due to the lack of the code design requirements (Belleri et al. 2015; Magliulo et al. 2014; Sousa et al. 2020).

An extensive research was previously done related with the characteristics of the PRC buildings of the Portuguese industrial park, where the constitution of the buildings was examined in detail in order to obtained the most accurate data to characterize the industrial Portuguese park (Rodrigues, Sousa and Vitorino 2020). Based on the data collected it was possible draw some conclusions. One of the most relevant to develop the present work regards the identification of the main characteristics and its evolution through the years that allowed to divide the buildings in three different classes: 'pre', 'moderate' and 'post code' buildings. In the present work two existing buildings are studied and compared: one belonging to the 'moderate code' and the other one belonging to 'pre-code'. The buildings were chosen in order to evaluate the relation between the main characteristics and their seismic behaviour, namely at the beam-to-column connection level. The seismic assessment is carried out by means of nonlinear static (pushover) and dynamic analyses, following the recommendations presented on Eurocode 8 – Part 3 (CEN 2005).

1. Precast building characterization

This section starts with a brief overview of this typology of buildings inserted in the Portuguese industrial park, together with a more detailed geometrical and mechanical characterization of the case – study buildings analysis in this study.

1.1. General overview of the Portuguese industrial park

The buildings presented in this study belong to a database that served as a starting point for the development of the work of Rodrigues *et al.* (2020), which characterizes the Portuguese PRC industrial building stock. The database consists of a set of 73 design projects of existing buildings. The parameters collected in each design project includes global geometry, column dimensions, reinforcement ratios, connections details, mechanical properties of the materials and other information that was considered important for the characterization of buildings, such as year of construction. Without going into detail, some data is important to mention to provide a general overview of the characteristics of the buildings of the Portuguese industrial park. The collected projects date from 1960 to 2020, with a clear concentration from 1990. The main activities performed in the buildings are related with light industrial activities and warehousing, and the majority (91%) of the buildings have 1 or 2 bays and 1 storey (78%). The span length in the principal direction (longitudinal direction of the precast principal beams) range between 20 and 25 m (50%), while in 25% of the buildings have spans between 25 and 30 m. The typical columns height varies between 6 and 8 m (80%). Regarding the material properties, concrete strengths between 20 MPa and 30 MPa was the most frequent, and the most recurrent reinforcement class was the S400 (49%) and S500 (47%). Concerning the beam-to-column connections, only in 60% of the design projects was possible to access the details of the dowel connections, being expectable that in most of the remaining 40% of the buildings the connection could be ensured only by friction. In cases where the connection was detailed, 71% of the buildings had 2 dowels and the most used dowels were the 16 mm of diameter (30%), 25 mm (27%) and the 20 mm (18%).

Finally, it is important to mention that it was verified that the dowels properties do not seem to be related with the seismic actions expected for the associated locations of the buildings.

1.2. Characterization of the case-study buildings

The existing PRC buildings considered to perform the seismic assessment, following the prescriptions of the Eurocode 8 – part 3 were collected from the database, whose results was presented in Rodrigues *et al.* (2020), and briefly presented in the previous subchapter. The buildings were chosen with the objective of reflecting the typical properties on each period. Given the lack of specific codes addressing the design of PRC buildings in Portugal, it was decided to define three sub-classes based on the year of construction, as an important fraction of the mechanical and geometric properties depend on the year of construction. The sub-classes were defined as 'Pre code', 'Moderate code' and 'Post code'. In the Table *1* it is presented the structures ID, the corresponding seismic zone and the code sub-class based on the year of construction. The 'Pre code' buildings were defined as those built from 1960 to 1980, the 'Moderate code' from 1980 to 2000 and the 'Post code' from 2000 to 2020. One building from 'Pre code' and other from 'Moderate code' was analysed. In Figure 1 the buildings location is presented in terms of the seismic zonation for the Portuguese territory, corresponding to seismic zone 1.3 (type 1).

Table 1. Structures information.

	Structure ID	Year	Seismic Zone
Mid code	B3_ModC	1997	1.3
Pre code	B5_PreC	1979	

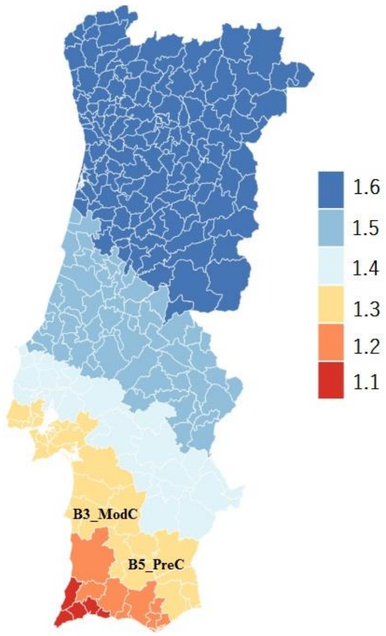
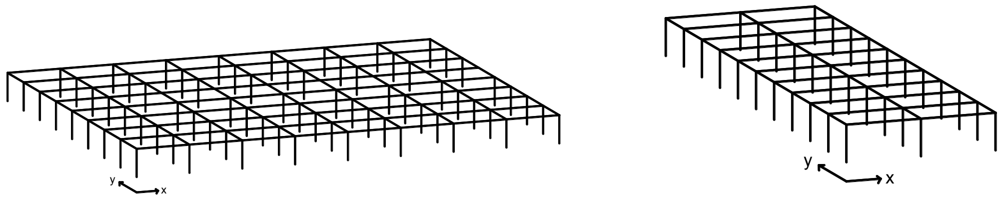


Figure 1. Buildings distribution vs Type 1 seismic zonation.

1.3. Geometric and mechanic characterization of the buildings

The global configuration of the buildings analysed is presented in Figure 2 while the main geometric and material characteristics are summarized in the Table 2 and Table 3, respectively.



a) B3_ModC b) B5_PreC

Figure 2. Model of the structures in study.

Regarding the geometric characteristics (Table 2), the column slenderness ratio was calculated according to the expression described below and recommended in EC2 (CEN 2010):

$$\lambda = \frac{l_0}{i} \tag{1}$$

where l_0 is the effective length and i is the radius of gyration of the uncracked concrete section. The slenderness was calculated for both directions and the values are shown in the Table 2.

Table 2. Geometric characteristics of the buildings in studying

Structure ID	**Number of spans**		**Span length**		**Height [m]**	**Columns Slenderness**	
	x	y	x [m]	y [m]		x	y
B3_ModC	8	8	17.0	6.0	9.0	69	89
B5_PreC	2	10	15.0	4.2	7.5	65	65

The properties of the buildings selected reflect the evolution in terms of geometry and material observed with the year of the design project (Table 3). For instance, in the 'Pre Code' building the beam-to column connection is ensured by the friction between these elements, while in the 'Moderate Code' building the connection is made by means of a mechanical element – the dowel – combined with the friction component.

Table 3. Material and reinforcement detailing characterization.

Building ID	**Concrete f_{cm} [MPa]**	**Steel fym [MPa]**	**Column b*x*h [m]**	**% Steel**		**Dowel Ø [mm]**
				Longitudinal	Transversal	
B3_ModC	33	440	0.45x0.35	1.60	0.17	2Ø16
B5_PreC	24	440	0.40x0.40	0.79	0.16	-

2. Structural Analysis

2.1. Numerical Modelling

The structural behaviour of the PRC buildings was simulated along the two main directions with a 3D model using the structural analysis software OpenSees (McKenna 2011). In these models, the columns were simulated using force-based nonlinearBeamColumn elements with distributed inelasticity with 5 integration points in each element, whilst the beams, which are expected to remain undamaged, were modelled with linear elastic elements. In terms of materials, for the concrete it was used the Concrete02 model, whereas the columns longitudinal reinforcement was simulated considered the Steel02 model, based on the Giuffre-Menegotto-Pinto (Menegotto and Pinto 1973) material model. Regarding the beam-to-column connections, its behaviour was simulated through a macro-element proposed by Sousa *et al.* (2020) which is capable of precisely describe the main mechanisms identified in conventional beam-to-column PRC connections, namely friction between the different elements, steel dowels and the neoprene pad.

2.2. Nonlinear Static Analysis

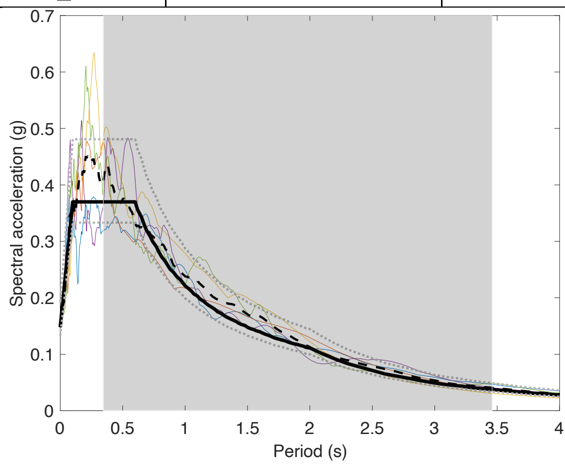
The assessment of the buildings was firstly carried out through nonlinear static (pushover) analyses. These analyses were carried out along the two main directions of the buildings adopting a distribution of incremental horizontal forces proportional to the shape of the fundamental modes and a uniform distribution proportional to the mass, according with the Eurocode 8 recommendations. For both cases it was also considered the inclusion of the effects of the accidental eccentricity was also considered, through the movement of the centre of mass by 5% of the building's length perpendicularly to the direction of the acting seismic action, in order to account for possible variations in the distribution of masses in the structures. Additionally, the normative resistance for the flexural and shear mechanisms was calculated, along with the appliance of the N2 method, as defined in the Eurocode 8 (CEN 2005). The determination of the target displacement associated with the seismic hazard at the building location was based on the procedure presented on the Annex B of Eurocode 8 – Part 1 (CEN 2005), adopting an iterative procedure for improved accuracy. This approach follows the N2 method proposed by Fajfar (2000) and enables to determine the buildings seismic demand based on the elastic (5% damped) response spectrum. In Section 4, the responses of the elements at the global target displacement are compared against the elements capacity to assess their expected seismic performance.

2.3. Nonlinear Dynamic Analysis

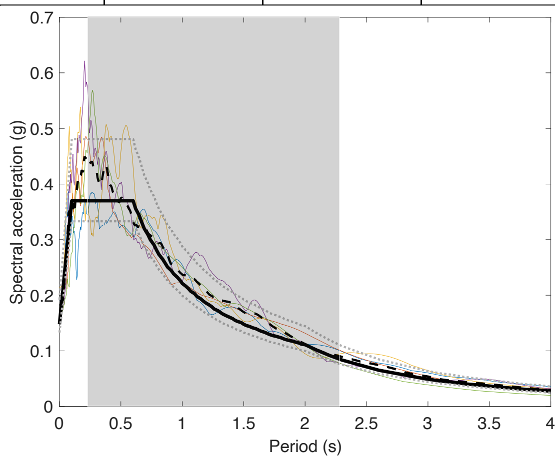
The seismic assessment of the building's performance was also carried out through nonlinear dynamic analyses. According to Eurocode 8 – Part 1, a suit of at least 7 analyses should be carried out in order to define the seismic demand as the average of the analysis set. In the present study, 10 analyses were considered for each building, corresponding to 5 different events, with each seismic component acting along the two main horizontal directions of the buildings. Each analysis considers the ground acceleration acting simultaneously along the two horizontal directions and the vertical one, corresponding to the accelerations recorded at the stations for each event.
The records were selected from suit of nearly 3500 records included in a database of ground motions recorded in the Mediterranean region. The selection and scaling of each suit of accelerograms follow generically the strategy prescribed in Eurocode 8, i.e., the average spectrum of the selected ground motion should be higher than the code peak ground acceleration and higher than 90% of the code spectra along the period interval between 0.2 and 2 times the fundamental period of vibration. Given that, the accelerograms are applied simultaneously along the two horizontal directions with accelerograms recorded also along two directions, the average of the fundamental periods in the two directions (T_m) was adopted as reference period of vibration of the building (see Table 4), whilst the event spectra was defined as the geometric mean of the two horizontal directions of the recorded motion.
Additional constrains were also imposed to limit the scaling to a factor of 2.5 and to minimize the error in terms of maximum spectral accelerations. Figure 3 shows the comparison between the code acceleration spectrum for the Seismic Zone 1.3 in Portugal and the acceleration spectra associated with the selected records. The graphs include also the average of the selected spectra (thick dashed line), the reference limits corresponding to 90% and 130% of the code spectrum (thin dashed line), and the period interval of interest (shaded area).

Table 4. Periods of vibration determined for the buildings in study.

Building Id	Fundamental period of vibration – T (s)		T_m (s)	$S_e(T_m)$	$S_{De}(T_m)$
	x	y			
B3_ModC	1.50	1.96	1.73	1.68	0.096
B5_PreC	1.13	1.14	1.14	2.55	0.062



a) B3_ModC



b) B5_PreC

Figure 3. Acceleration response spectra of the records selected to perform the nonlinear dynamic analyses for: a) building B3_ModC and b) B5_PreC.

3. Buildings Assessment

The results determined from both static and dynamic analysis are discussed in this section in view of the elements compliance with respect to Eurocode 8 – Part 3 capacity prescriptions, in terms of elements chord rotation and shear force.

3.1. Structural capacity

The capacity of the buildings elements was carried out in terms of chord rotation (deformation) and shear strength, whose values are shown in the Table 5, following the expressions proposed in the Eurocode 8 –Part 3 (CEN 2005). The assessment was carried out for the significant damage (SD) limit state, for which, the chord-rotation capacity θ_{um} is defined using the expression (2) presented below for convenience. In Table 5, the chord-rotation capacity values correspond to a front façade column, as an example.

$$\theta_{um}=\frac{3}{4}\frac{1}{\gamma_{el}}0,016\cdot(0,3^{\nu})\left[\frac{\max(0,01;\omega')}{\max(0,01;\omega)}f_{c}\right]^{0,225}\left(\min\left(9;\frac{L_V}{h}\right)\right)^{0,35}25^{\left(\alpha\rho_{SX}\frac{f_{yw}}{f_c}\right)}\left(1,25^{100\rho d}\right) \quad (2)$$

The symbols of the expression above are described in the Eurocode 8 –Part 3 (CEN 2005). In terms of shear strength of the RC elements, the capacity is given by expression (3).

$$V_R=\frac{1}{\gamma_{el}}\left[\frac{h-x}{2L_V}\min\left(N;0,55A_cf_c\right)+\left(1-0,05\min\left(5;\mu_{\Delta}^{pl}\right)\right)\cdot\left[0,16\max\left(0,5;100\rho_{tot}\right)\left(1-0,16\min\left(5;\frac{L_v}{h}\right)\right)\sqrt{f_c}A_c+V_w\right]\right] \quad (3)$$

The geometry and material properties to include in the previous expressions was defined based on the data collected from the original project and assuming a limited knowledge level (KL1) that, according with the code prescriptions, should result in a reduction of the material properties by a factor of 1.35.
The previous equations were applied only to the column given that for this typology of buildings, the beams should remain essentially undamaged. However, despite the code does not provide any specific consideration for PRC buildings, particular attention is given to the behaviour of the beam- to- column connection, as it is one of the main focuses of damage in recent earthquakes. In this regard, it was considered a relative displacement of 6 cm as suggested by Cornali *et al.*(2017).

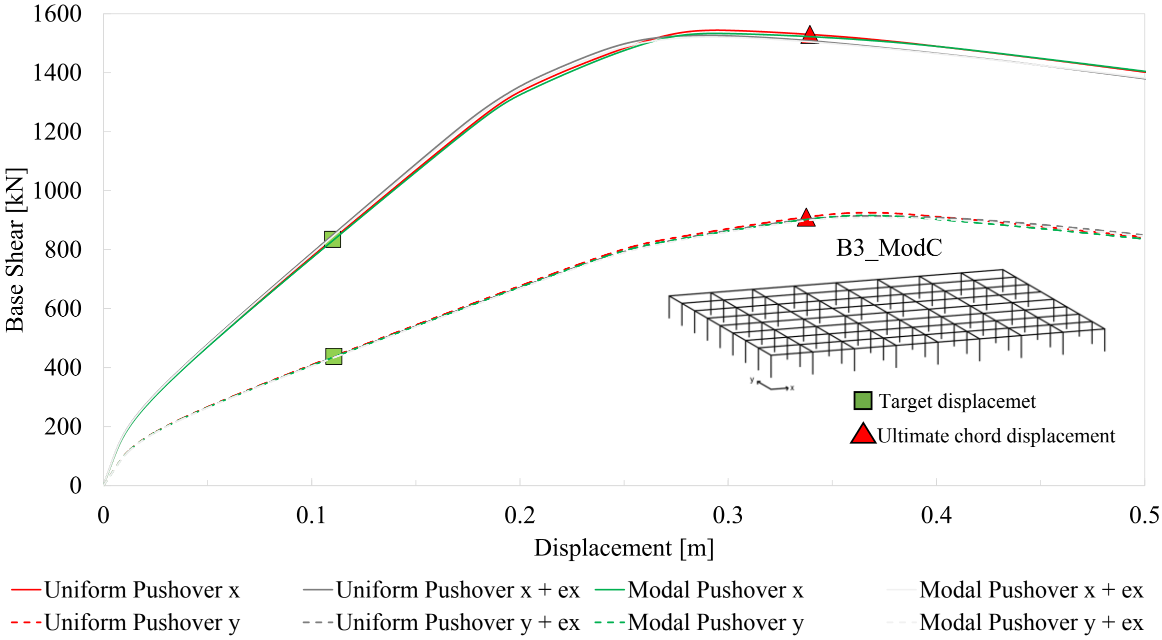
Table 5. Capacity values calculated according Eurocode 8 – Part 3 (CEN 2005).

Building ID	Chord rotation [rad]		Shear strength [kN]	
	x	y	x	y
B3_ModC	0.038	0.039	5316	5733
B5_PreC	0.037	0.037	1691	1691

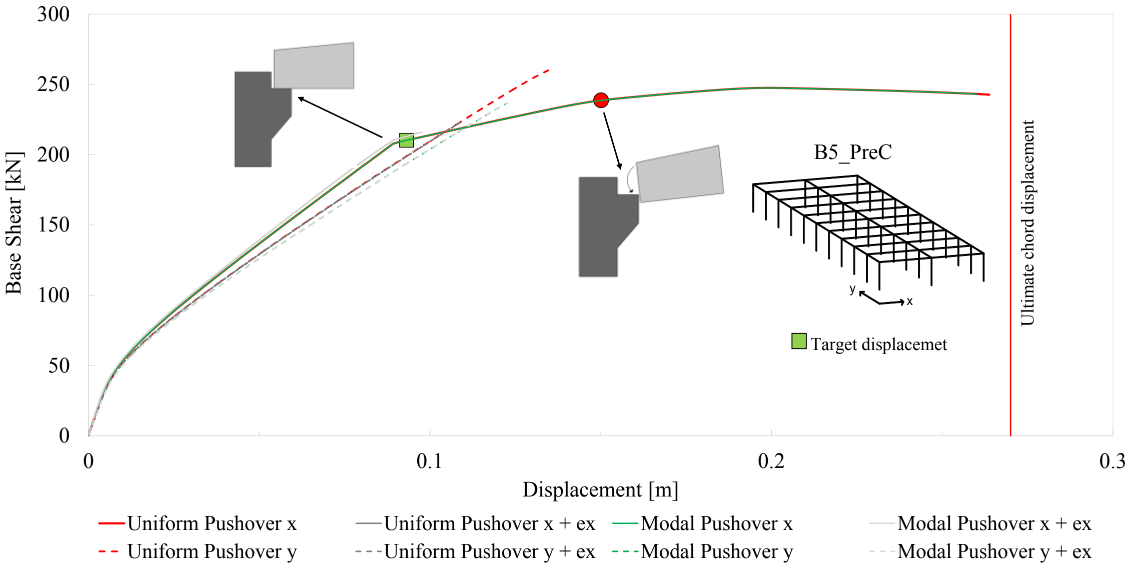
3.2. Static Analysis

The results of the pushover analysis are presented in Figure 4, showing the capacity curves in X and Y directions (solid and dashed lines, respectively) associated with the uniform and modal load distributions (red and green lines, respectively), as well as these distributions affected by the accidental eccentricity (dark and light grey lines for uniform and modal analysis, respectively).
The first clearest conclusion, valid for both case studies, regards the almost perfect overlapping of the pushover curves associated with the uniform and modal distributions, which is associated with the fact that the structures are single-story buildings and regular in plan.
Regarding the seismic safety assessment, both building appear to fulfil the code requirements, given that the target displacement associated with the seismic zone 1.3 is lower than the displacement associated with the exceedance of the elements chord rotation and shear capacity (see Figure 4). The latter mechanism, contrarily to what is commonly observed in conventional RC buildings, is actually very unlikely to occur given the large slenderness of the columns. Hence, based on the code requirements, the building could be classified as seismically safe.
Similar results were attained considering the behaviour at the beam-to-column connection, in particular when in presence of steel dowels that prevent the connection to fail under seismic loads. Yet, when the connection is ensured by friction only, potential failures might be expected under moderate to high seismic loads. Based on a limit differential displacement at the beam-to-column connections of 6 cm, as proposed by Cornali *et al.* (2017) for severely damaged connections, it was possible to verify that for the model B5_PreC (pre code design) the connection fail for a global displacement lower than the one associated with the ultimate chord rotation. Although the connection limit state occurs for a global

displacement larger than the target displacement (see Figure 4b), the results points for o a potentially vulnerable seismic behaviour.



a) Pushover curves for B3_ModC



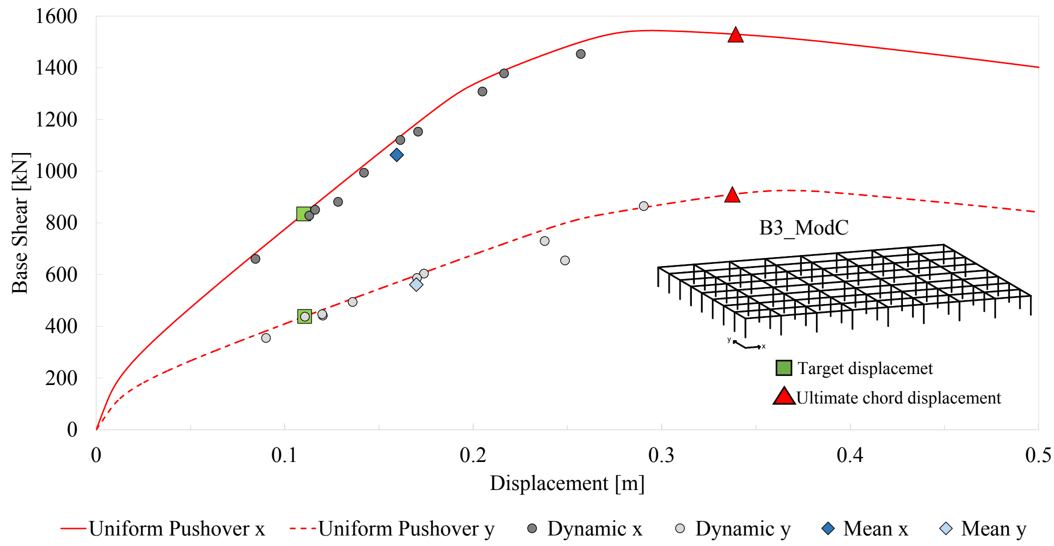
b) Pushover curves for B5_PreC

Figure 4. Pushover curves for x and y direction for both buildings in study.

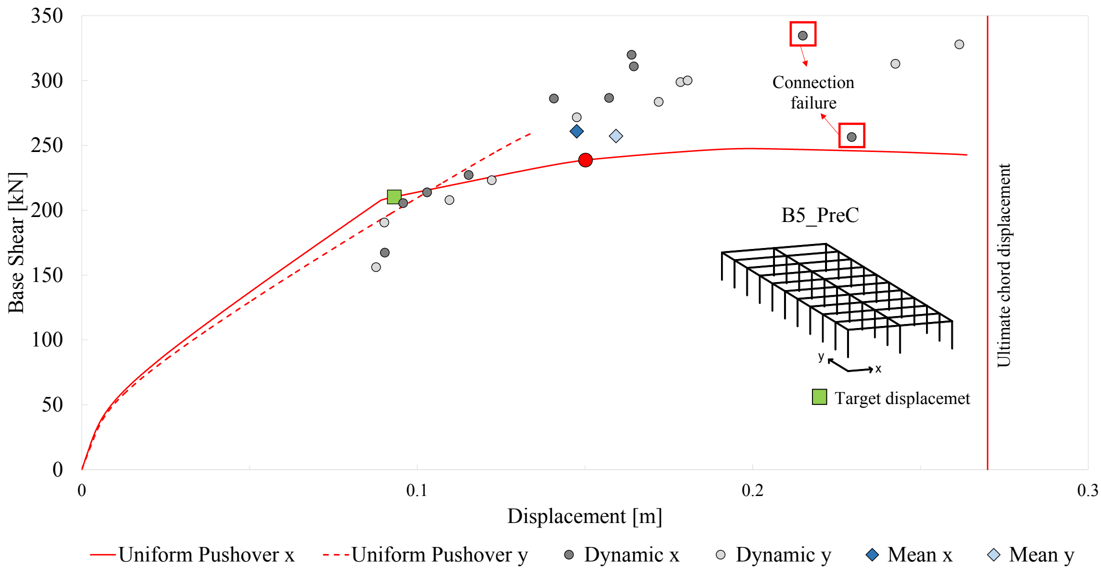
3.3. Dynamic Time History Analysis

This section discusses the results of the dynamic analyses comparing with the capacity curves obtained with the uniform load distribution, for both for X and Y directions (Figure 5). In this figure, the dark grey circles represent the response in the X direction and the light grey represents the response along Y direction. It is noted that these points represent the combination between the maximum base shear and the maximum top displacement that the structure experienced during the analyses, which may not necessarily be coincident in time nor be representative of a given structural state. Yet, for the sake of assessment and comparison with the pushover curves, these represent an admissible metric.

In general, for both buildings, the pushover curves present a good agreement with the set of records used to perform the dynamic analysis (better in the B3_ModC building). Yet, it is noted that most of the dynamic points present a larger displacement and base shear with respect to the target displacement obtained in the static procedure. Part of the differences is certainly justified by the conservative rules considered in the selection of records compatible with the code spectra. Nonetheless, the response measured during the dynamic analysis is, in some cases, significantly higher, indicating that the use of nonlinear static procedures appears to underestimate the seismic demand. For instance, the mean value of the dynamic analyses' response (blue diamond in Figure 5) is well beyond the target displacement obtained with static analyses.
Regarding the seismic safety, the results indicate that the building is safe with respect to the prescriptions defined in Eurocode 8 – Part 3, in terms of columns shear and chord rotation capacity, but fail in what respects the admissible beam-to-column connection relative deformation for two of the analysis, as illustrated in Figure 5b).



a) Dynamic analysis results for B3_ModC



b) Dynamic analysis results for B5_PreC
Figure 5. Dynamic results for x and y direction for both buildings in study.

4. Final Comments

The present work analysed the seismic performance of two existing PRC buildings of the Portuguese industrial building stock: one built in 1979 ('Pre code') and the other in 1997 ('Moderate code'). The assessment was carried out considering both static and dynamic analysis and evaluated the performance at the column and beam-to-column connection level.

From a numerical analysis point of view, the non-linear static analysis carried out considering a uniform and modal load distribution provide results in line with the ones obtained with the dynamic counterpart, especially in the essentially elastic regime. However, the nonlinear static procedures appear to underestimate the seismic demand, at least based on the limited number of analyses carried out for this typology.

The buildings, located in a region of moderate seismicity, exhibit a satisfactory behaviour when analysed through the expressions proposed by the Eurocode 8 – Part 3 to assess the columns performance. This conclusion was obtained considering whether static or dynamic assessment procedures. However, when assessing the performance of existing PRC buildings particular attention should be paid to the performance of the beam-to-column connection, which is not included in the prescription of the code. In fact, it was observed that, in the absence of steel dowels, the deformation overcome the limits reported in the literature. This observation points for the need to develop specific regulation to access existing buildings of this typology as well as to consider models that enables the accurate representation of the mechanisms involved in this type of connections, namely the contribution of the friction and steel dowel components.

Finally, results obtained point for a potential seismic risk associated with this typology of structures, in particular those built without adequate connection details and located in moderate to high seismic hazard. For those cases, detailed assessment and development of appropriate retrofitting technics is recommended.

Acknowledgments

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