VULNERABILITY ANALYSIS OF INDUSTRIAL RC PRECAST BUILDINGS DESIGNED ACCORDING TO MODERN SEISMIC CODES

Marianna Ercolino[a], Davide Bellotti[b], Gennaro Magliulo[c,d*], Roberto Nascimbene[b]

[a] *Department of Engineering Science, University of Greenwich, Central Avenue, Chatham, ME4 4TB, United Kingdom*

[b] *EUCENTRE, European Centre for Training and Research in Earthquake Engineering, Via Ferrata 1, 27100 Pavia, Italy*

[c] *Department of Structures for Engineering and Architecture, University of Naples Federico II, Via Claudio 21, 80125, Naples, Italy*

[d] *Construction Technologies Institute ITC-CNR, Via Claudio 21, 80125, Naples, Italy*

Abstract

Seismic performance-based design approach is currently implemented in modern building codes. Design requirements and provisions ensure an adequate structural performance under different intensity levels of seismic action. However, the probability of attainment of a performance level is implicitly considered in the code design approach (provisions and requirements); for instance, the minimum requirements in concrete structures cannot be simply correlated to the probability of collapse of the building as well as to its overall structural response.

The aim of this work is to assess the vulnerability with respect to the collapse limit state of industrial single-story RC precast buildings designed according to the current Italian seismic code. The comparison between the Italian code and the Eurocodes is provided throughout the paper. A parametric study is performed by investigating the safety against the collapse of 40 RC single-story precast structures. Multi-stripe analyses are performed by non-linear dynamic analyses at 10 intensity levels. The fragility of the structures is defined by means of the incremental N2 method, which has been demonstrated to be a suitable method for evaluating the collapse capacity of single-story precast buildings. The results demonstrate that the buildings are safe against the collapse mainly because of the structural overstrength with respect to seismic actions. The modelling assumptions are also validated in order to demonstrate the negligible influence of the cracking on the collapse as well as the importance of the geometrical nonlinearities for precast buildings.

Keywords: precast structures; seismic safety; fragility curves; incremental N2 method; multi-stripe analysis; building collapse.

* Corresponding author

1 Introduction

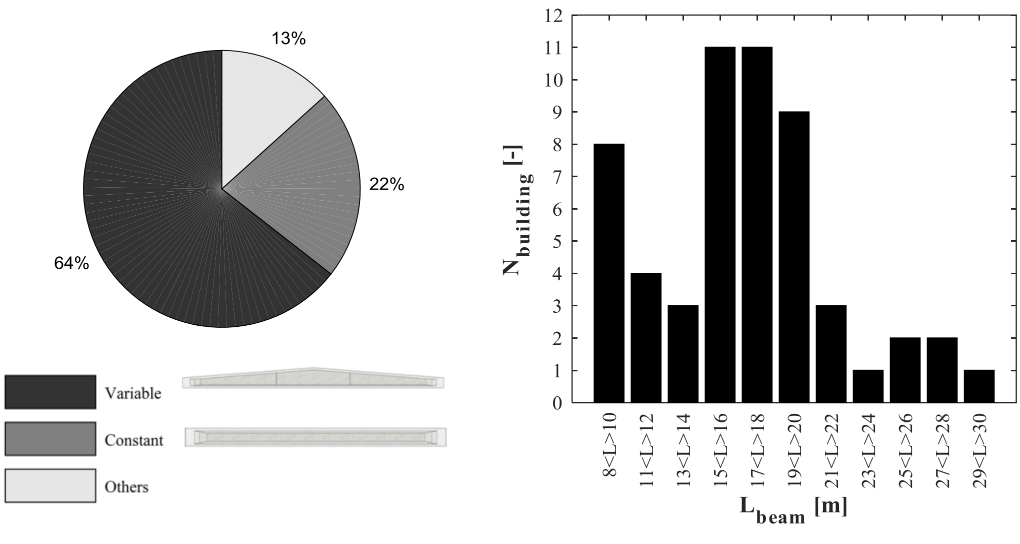
The current approach of modern building codes, such as European Eurocodes [1] and Italian building code [2], is based on the performance-based design approach. In the case of the seismic design, the structures have to achieve/show a defined performance under a given intensity level of the seismic action. A semi-probabilistic approach is adopted for the definition of both actions and strength (materials) by means of safety factors and combination coefficients. Moreover, code requirements and provisions (e.g., capacity design rules and seismic details) implicitly allow obtaining an adequate safety against the occurrence of limit states and brittle collapse mechanisms. In the last decades, many authors investigated the efficiency of modern codes for different structural typologies. For instance, some studies have been performed in order to define the effects of the design approach on the structural vulnerability. Most of them investigated existing [3, 4] and new RC frame structures [5]. The current codes significantly improved the design provisions with respect to past laws and regulations for RC precast structures. However, the code improvements have not been fully investigated as well as their influence on the global seismic vulnerability. Moreover, some recent seismic events in Europe have brought into sharp relief the vulnerability of precast buildings [6, 7], opening a scientific debate on the safety of this structural typology. Several experimental tests were performed on new buildings and connection systems [8-10]. On the contrary, few studies have been performed in terms of seismic assessment [11, 12] and risk study of precast buildings [13, 14]. Ercolino et al. [11] assessed the seismic performance of an existing precast structure in Emilia-Romagna (Italy), which was not designed for seismic actions. The modelling approach was validated by comparing the results of nonlinear dynamic analysis to the actual damage of the structure after the earthquakes on May 2012. The seismic assessment demonstrated the high vulnerability of existing buildings because of the poor detailing as well as the absence of adequate connection systems. Casotto et al. [13] developed a set of fragility curves for existing precast RC industrial buildings at different limit states. The buildings were designed essentially for static horizontal loads and with simply supported beam–to-column connections. Kramar et al. [14] performed a systematic seismic risk study on a set of precast structures. The nonlinear model consisted of one equivalent column and the parametric study was defined by varying the seismic masses as well as the column size. The structures were designed for seismic actions according to Eurocode 8 for one value of the peak ground acceleration (0.25 g). The results of the nonlinear analyses demonstrated that that the seismic risk is low if all EC8 requirements (including minimum longitudinal reinforcement requirement) are considered. An extensive parametric study was performed by Palanci et al. [12].The authors defined the fragility curves of 98 existing buildings in Turkey, designed and built from 1990 and 1998. The study results demonstrated that the older buildings can be defined as "low quality structures" whereas the newer buildings are the "good quality structures".

Most of these past research studies investigated the seismic performance of either existing buildings or structures designed by considering only the seismic actions. The combination of non-seismic actions is usually not considered in the design of the structural elements, even if these actions can influence the safety of the buildings. Moreover, nonlinear analyses were performed on single degree of freedom system by neglecting the presence of the eccentricity at the connection levels between the structural elements. Such an approach can influence the feasibility of the results in case of large vertical actions on the buildings.

This paper aims to define the seismic safety in terms of collapse of single-story RC precast buildings, designed according to the Italian building code. An extensive parametric study is performed on several configurations of single-story precast buildings, defined by varying some geometrical features. A rigorous design procedure is adopted in order to simulate real industrial RC precast structures and its results are discussed. A comparison between Eurocode and Italian code requirements is also performed, to extend the obtained outcome. The safety of the analysed buildings with respect to the collapse is investigated through both multi-stripe analyses and the incremental N2 method. In the final part of the paper, some modelling assumptions are discussed, in order to validate the study results as well as to define reliable models for future studies.

2 Typical structural layout

The investigated structures are single-story RC precast buildings, typically hosting industrial and commercial activities in Europe. Their geometrical features are selected in order to represent the most common buildings in Italy. A wide-ranging database was obtained by collecting data of Italian reports about the typology classifications and post-earthquakes assessment surveys [6, 15-17]. Figure 1 shows common beam typologies and typical bay spans in Emilia-Romagna region (Northern Italy). The adopted geometrical features are also typical of other seismic-prone countries in Europe, such as Slovenia and Turkey. In Turkey the precast RC members are used commonly for the construction of industrial facilities. Typical spans in these facilities varies between 15 m and 25 m and the typical height of these precast structures ranges between 6 m and 8 m [18]. Babic and Dolsek [19] define the typical geometrical features of one-storey precast buildings in Slovenia: median value of the height is 6.5m with variation of 0.25, the length of the beam is 14.9m (with the dispersion equal to 0.3) and the distance between portals is typically equal to 6.8 (with the dispersion equal to 0.28).



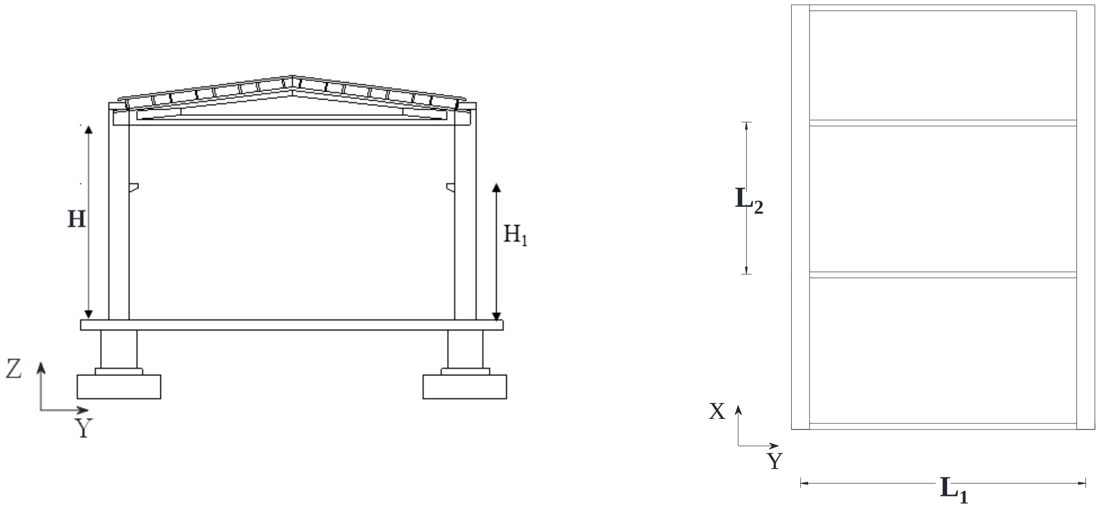
(a) (b)

Figure 1 Emilia Romagna database: (a) typology of beams; (b) distribution of bay spans

The investigated structural typology is defined as “isostatic columns” (Figure 2), i.e., single-story structures with cantilever columns. These RC precast buildings usually consist of the following elements: isostatic columns, prestressed principal and secondary beams pinned to the columns, roof elements pinned to the beams and vertical/horizontal cladding panels connected to the beams/columns.

Figure 2 shows the frontal view of the case studies in Y direction (a) and the plan view (b). The structure consists of 4 bays in X direction and 1 bay in Y direction. In this study the structural elements have the following features: monolithic precast columns, rectangular-shaped beams along X direction; prestressed principal beams with variable cross-section along Y direction; prestressed TT roof elements along X direction and vertical cladding panels. The columns have a fork (or corbel) at the top and a socket foundation at the base. The fork of the columns is adopted in order to restraint the torsional and lateral rotation of the beams. The principal/secondary beams are connected to the columns by steel dowels. The dowel connection consists of steel threaded bars inserted in both the beams and the columns (Figure 3). Such connection systems can be assumed as a pinned connection because of the limited rotational strength under horizontal loads [20, 21]. Panel-to-structure connections consist of steel elements and they are designed in order to support the weight of the panels and the seismic actions in the out of plane direction. This connection system should allow the relative displacement between the panel and the structure. Recent seismic events demonstrate that the panel can interact with the main structural elements under seismic actions because of either construction errors or limited space in the connection elements. However, the strength of the panel-to-structure connection under seismic loads is very small and it fails at very low intensity without significantly influencing the global seismic performance [9].

Four geometrical configurations are assumed in this study by varying the height of the columns (H in Figure 2(a)) and the width of the bays (L_1 and L_2 in Figure 2(b)). Table 1 shows the geometrical features of the investigated case-studies. The presence of a crane is assumed in the case studies since single-story precast structures usually host industrial activities. The crane is supported by a bracket, placed on the column at 4.5m for the geometries 1 and 2 and at 7.5m for the geometries 3 and 4 (H_1 in Table 1).



(a) (b)

Figure 2 Layout of the case studies: (a) frontal view (Y direction) and (b) plan view

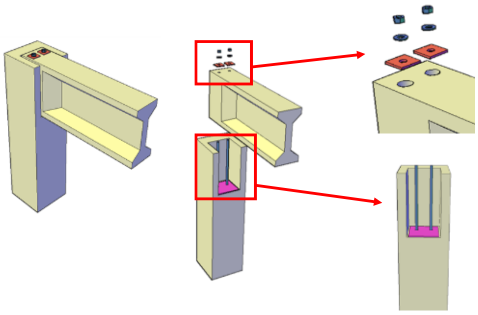


Figure 3 Typical beam-to-column dowel connection

Table 1 Geometrical configurations of the case studies

Geometrical configuration (Geo)	L_1	L_2	H	H_1
	[m]	[m]	[m]	[m]
1	15.00	6.00	6.00	4.50
2	20.00	8.00	6.00	4.50
3	15.00	6.00	9.00	7.50
4	20.00	8.00	9.00	7.50

3 Prototype structures: design and modelling

The study investigates several single-story precast structures. This section describes the adopted design approach as well as the nonlinear numerical model. The main features of all the buildings will be presented in terms of dimensions of structural elements and reinforcement details.

The described buildings are designed according to the Italian building code [2] in five sites in Italy (Table 2). Two typologies of soil are considered: type A (the average velocity of S waves in the upper 30 m, $V_{s,30}$, is larger than 800m/s) and type C ($V_{s,30}$ is in the range: 180m/s - 360m/s). Therefore, the parametric study consists of 40 case studies. In the following sections, each case study is identified by an ID, defining the geometrical configuration, the site and the soil type: for instance ID "Geo1-CA-soilA" refers to the case study with the geometrical configuration 1 ("Geo 1") and located in Caltanissetta (CA) on a rigid soil (A).

Table 2 Sites of the case studies

Site (S)	Latitude	Longitude	a_g [g]-T_R=50years	a_g [g]-
L'Aquila (AQ)	13.399	42.349	0.104	0.261
Napoli (NA)	14.268	40.854	0.060	0.168
Roma (RM)	12.479	41.872	0.055	0.123
Caltanissetta (CA)	14.060	37.480	0.034	0.073
Milano (MI)	9.186	45.465	0.024	0.050

3.1 Design approach

The case studies are designed according to the Italian building code [2]. The design of precast buildings takes into account several actions: permanent actions (self-weight and permanent non-structural weights), non-seismic variable actions loads (e.g., imposed load, wind and snow) and seismic actions (both vertical and horizontal components). Two limit states are considered under seismic loads: the Damage Limitation (DL) Limit State (LS) and the Ultimate (U) Limit State (LS).

Table 3 shows the considered actions for the main structural elements. The design of both the secondary beams and the cladding panels is not performed in this study. Constant geometrical features are assumed for these elements in all the case studies: the secondary beams have rectangular-shaped cross-section (30cm x 60cm); the vertical panels have a self-weight of 4kN/m^2 and their height is assumed equal to the sum of the column height, the maximum height of the principal beam at the mid span and the height of the roof elements. This simplified approach does not jeopardize the reliability of the study because in real structures the geometry of these elements does not significantly change in different geometrical configurations.

The connection systems are not designed in this study and their seismic behavior is not modelled in the numerical analyses; they are assumed strong and stiff enough to avoid their brittle/premature failures. Both Italian code and Eurocodes give specific design provisions for the connections in precast structures. They should be designed according to the capacity design principle by adopting large safety factors. Moreover, other design guidelines as well as past research studies recommend detailing of the connected elements in order to avoid any brittle failure of the connection. The above cited provisions/guidelines ensure that the connection strength is large enough with respect to the yielding capacity at the column base. Such statement was also justified by past experimental tests [21].

The foundation system is not designed; in the numerical analyses a fixed constraint is assumed at the base of the columns. The column fork is constant in all the buildings: the height is equal to 60cm and the thickness is 15cm. The structural materials are constant for all the case studies: the concrete has a cubic cylinder compressive strength of 45N/mm^2 and the reinforcement steel has a characteristic yielding strength of 450N/mm^2.

Table 3 Loads for the designed structural elements

<table>
<tr><th>Structural element</th><th>Permanent actions</th><th>No-seismic variable actions</th><th colspan="2">Seismic horizontal action</th><th>Seismic vertical action</th></tr>
<tr><td rowspan="2">Roof elements</td><td rowspan="2">Self-weight</td><td>Live load</td><td colspan="2" rowspan="2">None</td><td rowspan="2">Negligible</td></tr>
<tr><td>Snow</td></tr>
<tr><td rowspan="2">Principal beams</td><td rowspan="2">Self-weight</td><td>Live load</td><td colspan="2" rowspan="2">None</td><td rowspan="2">ULS</td></tr>
<tr><td>Snow</td></tr>
<tr><td rowspan="5">Columns</td><td rowspan="5">Self-weight</td><td>Live load</td><td rowspan="5">DLLS</td><td rowspan="5">ULS</td><td rowspan="5">None</td></tr>
<tr><td>Snow</td></tr>
<tr><td>Wind</td></tr>
<tr><td>Crane</td></tr>
<tr><td>Imperfections</td></tr>
</table>

3.1.1 Roof elements

The roof of the structures consists of TT prestressed elements (Figure 4). These elements are connected to the principal beams by means of dowel connections. At the top they are connected to each other by means of metallic systems as well as by a cast-in-situ concrete slab (with a thickness of 5cm). According to the Italian code [2], this connection system (i.e., connected roof elements and a concrete slab thicker than 4cm) ensures that the roof behaves as a rigid body in its own plane. In Eurocode 8 [1], the rigid diaphragm hypothesis is not related to either geometrical limits or connection systems; it should be verified in terms of in-plane flexibility by means of structural analyses. However, the assumed hypothesis is valid, as broadly recognized in past studies [22].

The roof is designed for the following actions: the live load is equal to $0.5kN/m^2$ (roof of an industrial building) and the snow is evaluated as a distributed vertical load, according to the site. The Italian code requests to consider the vertical component of the seismic action in the design of a prestressed horizontal element if: 1) the building is located in a medium-high seismic zone (e.g., the peak ground acceleration, a_g, of the site is larger than 0.150g for a return period of 475years); 2) the span of the roof element is larger than 8m. Therefore, the vertical component of the seismic action is neglected for the design of the roof elements in all the investigated case studies. In Eurocode 8, the vertical component of the seismic action is considered if the peak ground acceleration is larger than 0.25g (e.g., L'Aquila) and the horizontal or nearly horizontal elements are pre-stressed (there are no limits on the span width). However, this difference does not significantly influence the design results for the two codes; the demand due to the non-seismic actions is usually larger than the seismic one for these horizontal elements. Table 4 shows the dimensions of the TT roof elements in all the case studies: B is the width of the roof element and H_{roof} is the height. The details about the reinforcement bars are not reported in this paper since they are not required to perform the nonlinear analyses (see Section 4).

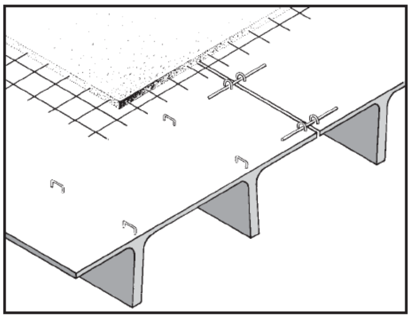


Figure 4 Layout of the roof: TT elements and cast in-situ slab

Table 4 Dimensions of the TT roof elements

B	H_{roof}	Site	Soil	Configuration
[cm]	[cm]	[-]	[-]	[-]
160	40	All	A, C	1, 3
240	40	All	A, C	2, 4

3.1.2 Principal beams

The principal beams are prestressed RC elements with variable cross-section: a T-shaped section at the connection to the column and an I-shaped section close to the mid-span. The principal beams are designed under two combinations of actions: 1) the combination of vertical actions (self-weight, live/imposed loads and snow) and 2) the combination of the seismic vertical component. For these elements the vertical component is considered for the structures located in AQ and NA (when a_g is larger than 0.15g at the ULS) with all the geometrical configurations (the span of the beams is always larger than 8m).

Table 5 shows the dimensions of a rectangular-shaped section, equivalent to the principal beams in terms of area and mass: B_{mean} is the mean value of the beam base and H_{mean} is the mean value of the beam height. The principal beams are pinned to the columns and they do not contribute to the lateral response under seismic actions. The details about the reinforcement are not reported in this paper since they are not used in the numerical analyses (see Section 4).

Table 5 Mean dimensions of the principal beams for all the case studies

B_{mean} [cm]	H_{mean} [cm]	Site [-]	Soil [-]	Geometry [-]
19	114	All	A, C	1
19	150			2
19	114			3
33	140			4

3.1.3 Columns

The columns are precast square-shaped elements; both the cross-sections and the reinforcement ratios are constant in each structure. As anticipated in Table 3, the design of the columns should take into account several actions and, therefore, several combinations. A tri-dimensional elastic model (Figure 5) is implemented in OpenSees program [23] in order to evaluate the demand on the structures under both vertical and seismic actions. The accidental eccentricity is defined for the centre of mass according to the provisions in both the Italian code and Eurocode 8 in order to take account for uncertainties in the location of masses. The dimensions of the structures are given in Table 1. The model consists of elastic one-dimensional elements: 1) elastic columns with a 50% reduced stiffness, 2) elastic principal beams with an equivalent rectangular-shaped section (Table 5) and 3) elastic secondary beams. The use of an equivalent section for the principal beams does not influence the final results of the paper since the main goal is the assessment of the seismic safety of the structure by considering the failure at the column base only. If some failure modes of the beams should be taken into account, the modelling approach of the beam should consider the actual torsional and lateral stiffness of the elements.

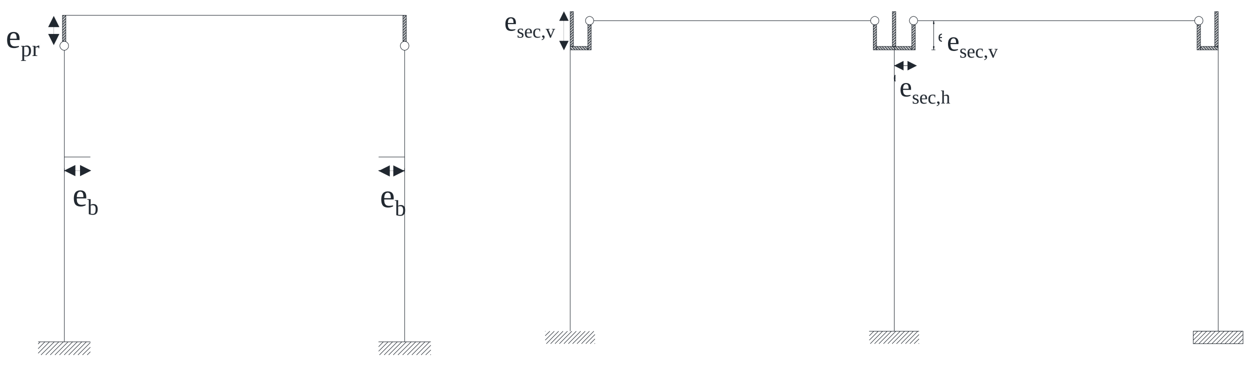
Concerning the connection systems, the following modelling assumptions are considered.

- The columns are fixed at the base and the height is equal to the values in Table 1 (H).
- The principal beams are connected to the columns (Figure 5(a)) by hinges, simulating the dowel connections. The eccentricity between the column and the beam axis (e_{pr} in Figure 5(b)) is assumed equal to half of the mean beam height (Table 5).
- The secondary beams are connected to the column forks by hinges. Horizontal and vertical eccenticities are considered (Figure 5(b)):

  $e_{sec,h} = B/2 - t/2$ (1)

  $e_{sec,v} = h_f + H_{sec}/2$ (2)

  In Eq. (1) $e_{sec,h}$ is the horizontal eccentricity, B is the column dimension and t is the fork/corbel thickness (15cm). In Eq. (2) $e_{sec,v}$ is the vertical eccentricity, h_f is the fork/ corbel height (60cm) and H_{sec} is the height of the secondary beams.
- The crane is applied to the column by means of a bracket. This bracket is assumed as a rigid element with a breadth of 60cm (e_b in Figure 5(a)).



(a) (b)

Figure 5 Elastic model of the precast buildings

Figure 6 shows the considered non-seismic actions; all the symbols are described in Table 6. Both the Italian code and the Eurocode requires to evaluate the demand on the structure under non-seismic actions according to a specfic combination at the ultimate limit states. In this combination one variable action is defined as either the main variable action or a secondary one. For instance, if the snow and the wind are the two variable actions, two combinations should be defined: 1) the snow is the main variable action and the wind is the secondary one; 2) the wind is the main variable action and the snow is the secondary one. During the design phase all the possibile combinations of the action in Table 6 are considered. The values of the actions depend on both the geometry of the case-studies and the site. Table 7 shows typical values (minimum and maximum) of the loads due to the wind, snow and crane in all the case-studies. The number of the considered combinations increases if the following conditions are taken into account: the imperfections and the wind actions are not considered acting simultaneously in the two horizontal directions; different locations of both the crane and the crane hook should be assumed and the temperature distorsion (15˚C) can assume both positive and negative values. It is worth to highlight that the demand due to non-seismic actions is amplified in order to take into account the effect of the geometrical nonlinearities, if necessary.

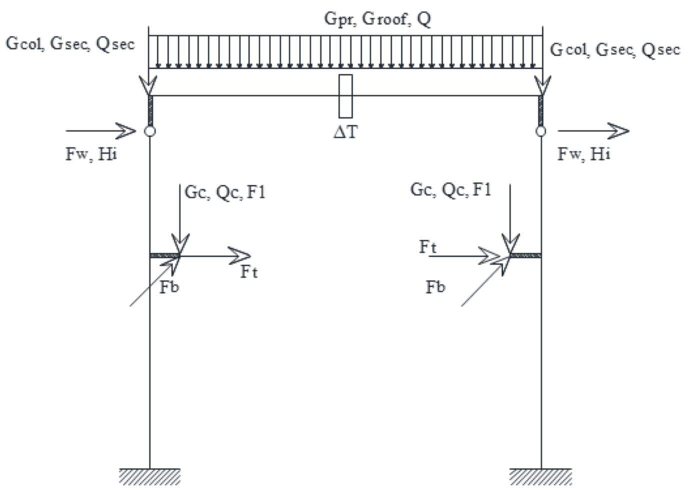


Figure 6 Load distribution on the elastic model

Table 6 Loads on the columns

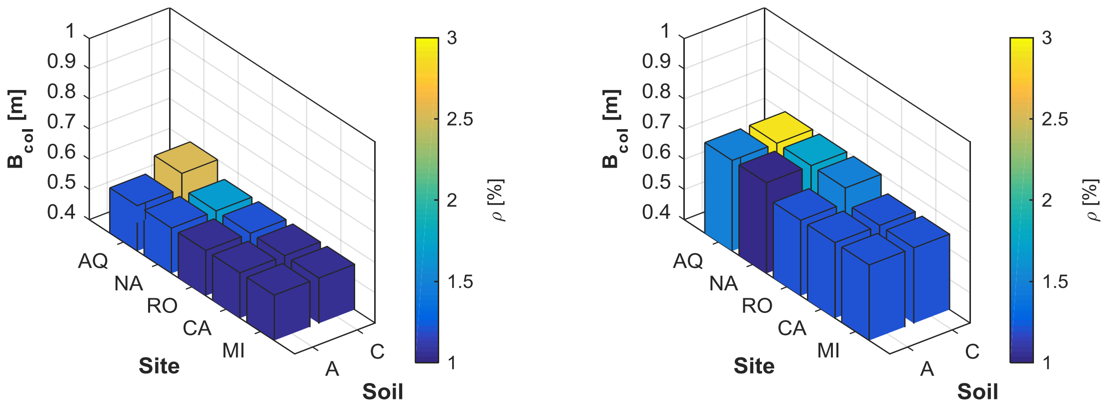
Actions	Symbol
Crane - hook	F_1
Crane – Self weight	G_c
Crane – supporting beam	Q_c
Crane – skewing force	F_t
Crane – acceleration force	F_b
Wind	F_w
Imperfections	H_i
Self-weight	G_{col}
Secondary beams – self-weight	G_{sec}
Secondary beams – live load	Q_{sec}
Principal beams - self-weight	G_{pr}
Roof elements - self-weight	G_{roof}
Roof elements - live load	Q
Temperature	ΔT

Table 7 Typical values of actions for the case-studies

Actions	Typical values (minimum and maximum) [kN]
Crane - hook	100
Crane – Self weight	25
Crane – supporting beam	[4.65-12.40]
Crane – skewing force	15
Crane – acceleration force	21.43
Wind	[12.83-89.15]

Dynamic linear analyses are perfomed by means of the above described elastic model in OpenSees in order to evaluate the seismic demand on the column. The analyses are performed in both the horizontal directions and the seismic effects are then combined. According to the Italian code, the structure should be verified for two limit states: DLLS and ULS. In this study the behavior factor is assumed equal to 2.5 for all the case studies, according to the Italian code provisions for isostatic columns in ductility class “B” (medium ductility in EC8). In EC8 the investigated structures are defined as frame structures; therefore, the behaviour factor is larger (q=4.5) than the factor used by the Italian code. However, this q value can lead to an overestimation of the dissipative properties of the precast industrial buildings under seismic actions [24]. In the combinations of seismic actions, several cases should be defined to take into account the different locations of the crane. P-Δ effects are also taken into account during the design phase according to the Italian code (and Eurocode 8) provisions: 1) amplification of the effets according to the value of the stability factor, θ [25]; 2) mimimum dimensions of the column section, i.e. larger than 1/10 of the shear length, if P-Δ effects are not negligible (θ larger than 0.1).

The column dimensions are reported in Figure 7 for all the case studies: the height of the bars is the cross-section dimension and the color represents the ratio of the longitudinal reinforcement.



Geo 1 **Geo 2**

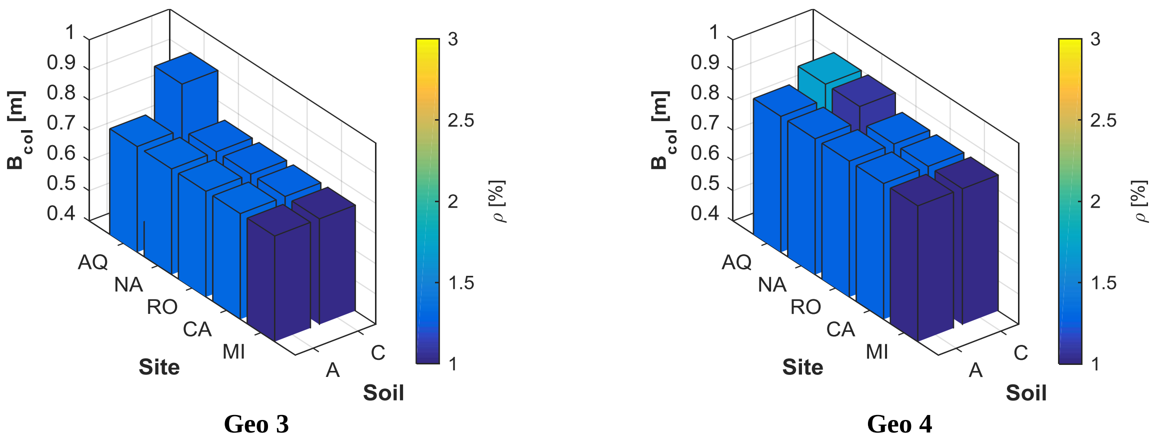


Figure 7 Column dimensions and ratio of the longitudinal reinforcement

According to these results, some conclusions can be drawn.

- Most of the case studies provide a low reiforcement ratio, about 1% (i.e., the minimum ratio required by the code). This minimum requirement can lead to a significant overstrength of the structures.
- The reinforcement ratio is larger than 2% if the structure is located in a high seismic zone (e.g., AQ) on flexible soil ("C") and the column height is 6m (Geo1 and Geo2). In this case, the column dimensions are smaller than the corresponding case studies with Geo3 and Geo4 because the DLLS verification is less restrictive; hence, at the ULS a larger quantity of reinforcement is requested.
- Only in few structures the reinforcement design is influenced by the seismic action rather than by other actions. Figure 8 shows the limit interaction surfaces of two case studies (Geo3-AQ-SoilA and Geo3-AQ-SoilC) along with the points corresponding to the demand for all the combinations of actions. These combinations are identified in the legend by the main variable action. The maximum effects are due to the seismic actions only for flexible soil; in the other cases, wind action gives the maximum demand on columns.
- In few cases the column dimensions are larger than 1/10 the shear length (i.e., the column height in the investigate structures); in the most of the structures the geometrical nonlinearities are negligible (i.e., stability factors are smaller than 0.1).

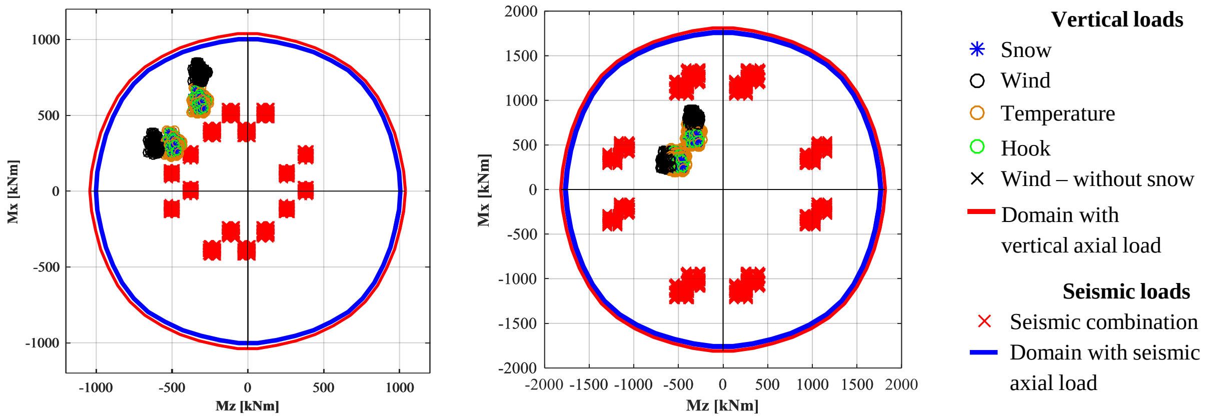


Figure 8 Limit interaction surfaces for a perimetral column: (a) Geo3-AQ-A and (b) Geo3-AQ-C

3.2 Proposed modelling approach

Nonlinear dynamic analyses are performed for all the case studies in order to evalaute their seismic performance. The nonlinear model is implemented in OpenSees; it consists of columns, secondary beams and principal beams, as shown in Figure 2 and Figure 5. The horizontal elements (beams) are elastic single-dimensional elements. The nonlinear behaviour of the structure is concentrated at the columns base and it is simulated by means of a lumped plasticity approach. The columns are fixed at the base in the nonlinear model. The beam-to-column connections are assumed strong pinned system. The plastic hinge is defined by means of a tri-linear moment-rotation curve [26], consisting of: the yielding point (green point in Figure 9), the capping point (yellow point in Figure 9) and the post-capping point (red point in Figure 9). The yielding moment (M_y) is defined by performing a fiber analysis on the RC cross-section and it is evaluated as the minimum value between the yielding of the steel reinforcement and the attainment of the maximum strenght in the concrete cover. The yielding rotation (θ_y) is evaluated according to Fardis [27] and both the capping point (M_c, θ_c) and post-capping rotation (θ_{pc}) are evaluated according to Haselton [28]. The hysteretic behaviour is modelled according to Ibarra [29]. The seismic mass of the structure is concentrated in the geometrical barycenter at the roof level.

For each case study two different plastic hinges are defined for perimetral columns and corner columns, according to the axial loads on the columns. The mean value of the material (concrete and steel) strength is used in the nonlinear analyses (Table 8). The mean strenghts are evaluated by adopting typical exeperimental values of the coefficients of variation (COV) [30]. The model of the structures is provided as supplementary material with this paper.

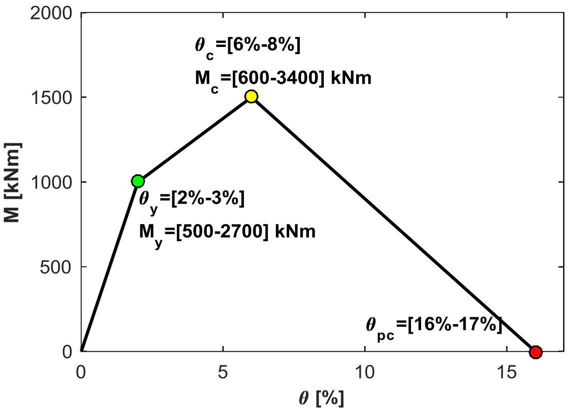


Figure 9 Moment-rotation backbone curve of the columns and range of the characteristic points

Table 8 Mean value and COV for concrete and steel

Concrete		Steel	
fcm	**COV**	**fym**	**COV**
[N/mm^2]	[%]	[N/mm^2]	[%]
59.75	15	490.3	5

4 Vulnerability assessment

The seismic response of precast buildings is investigated by means of the following steps: 1) nonlinear static analyses are performed in order to define the capacity of the structures at the collapse limit state; 2) multi-stripe analyses are performed in order to define the seismic safety of the structures and 3) incremental N2 (IN2) method is also applied in order to define their seismic safety. The multi-stripe analyses [31] are performed at 10 intensity levels (i.e. 10 return periods in Table 9); a set of 20 real records is selected at each intensity level for five sites and both the soil types. A detailed probabilistic hazard study has been performed in the framework of the same national project of this research. The details of the study are presented in [32]. The records were selected by means of the Conditional Spectrum (CS) approach [33-35] at a reference period of 2.0 seconds, given the structural typology. Figure 10 shows the response spectra of the selected records for Caltanissetta on Soil A. The colours show the intensity levels.

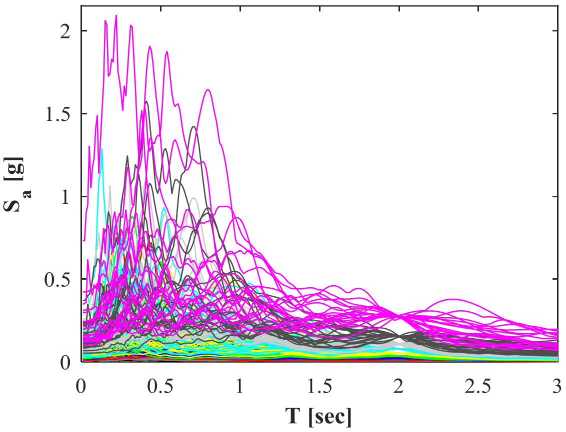


Figure 10 Spectra of the records for Caltanissetta on soil A

Table 9 Return periods for 10 intensity levels

IM [-]	**1**	**2**	**3**	**4**	**5**
T_R [years]	10	50	100	250	500
IM [-]	**6**	**7**	**8**	**9**	**10**
T_R [years]	1000	2500	5000	10000	100000

4.1 Numerical results

The capacity of the structures is evaluated at the collapse limit state by means of pushover analyses. The capacity point (collapse force and displacement) is evaluated on the pushover curve (red point in Figure 11) at 50% reduction of the peak strength (green point in Figure 11). Table 10 shows the capacity drift of the structures. The values of the collapse drift are almost constant with the site and the geometry. The collapse of the structures occurs at an average reduction value of the maximum moment equal to 36%.

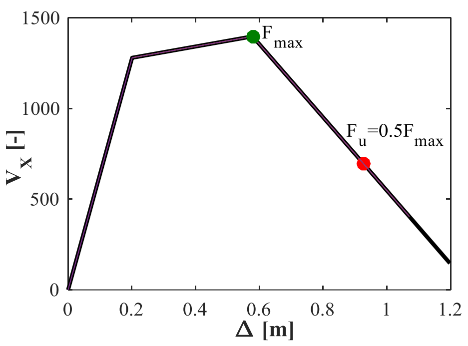


Figure 11 Pushover curve for the case study Geo3-AQ-A: capacity point (red marker)

Table 10 Capacity in terms of drift [%]

Site (S)	**Soil Type**	**Geo 1 Drift**	**Geo 2 Drift**	**Geo 3 Drift**	**Geo 4 Drift**
L'Aquila	A	10.9	12.8	10.0	9.8
	C	12.6	13.9	10.6	10.6
Napoli	A	11.0	12.5	10.0	9.9
	C	11.5	13.0	10.0	9.9
Roma	A	10.8	12.4	10.0	9.9
	C	11.0	12.8	10.0	9.9
Caltanissetta	A	10.8	12.4	10.0	9.9
	C	10.8	12.4	10.0	9.9
Milano	A	11.3	12.4	9.5	9.6
	C	11.3	12.4	9.5	9.6

The multi-stripe analyses are performed on all the case studies and the results are reported in terms of demand/capacity (D/C) ratios along with the reference (T=2.0sec) spectral acceleration at each intensity level (Figure 12). The ratio is evaluated as the maximum of the ratios in the two horizontal directions. The square markers are the cases of dynamic instability; for these points, a fictitious displacement is plotted since they correspond to very large demand in terms of drift in the structures. According to the results of the nonlinear dynamic analyses, the following conclusions can be found.

- By changing the seismic hazard of the site, the safety of the structure significantly changes. This can be justified by the adopted design approach. The column cross-sections and the reinforcement ratios do not change in most of the structures (Figure 7) because of the code minimum requirements in seismic prone areas.
- The geometry does not significantly influence the seismic behaviour of the structure. The higher structures (H=9m) are safer in L'Aquila (high seismic hazard): the design code provides severe requirements for high flexible structures (minimum section of columns because of P-Δ effects).
- The soil type influences the structural response in terms of number of collapse cases (e.g. Geo2-AQ). This means that the increment of design reinforcement and section dimensions due to the soil C with respect to the soil A is not able to guarantee the same safety against the collapse.

4.2 Vulnerability

According to the results of the multi-stripes analysis, very few collapses (D/C>1) are recorded for the investigated case studies. The multi-stripe analysis can be adopted for defining the fragility curves if there is a wide range of spectral accelerations at the collapse and a larger number of collapses. Therefore, this analysis method has been used only for four case studies (Geo3-AQ-SoilA, Geo3-AQ-SoilC, Geo4-AQ-SoilA and Geo4-AQ-SoilC) by amplifying the last IM level (10) up to achieve the collapse for 50% of the records (due to either the attainment of the collapse displacement or the dynamic instability). The maximum adopted record amplification factor is equal to 1.6 for all the case studies. The fragility curves for the four case-studies are presented in Figure 13.

A different method is needed to assess the vulnerability with respect to the collapse limit state of the other case studies, such as the incremental N2 (IN2) method [36]. In order to verify the reliability of such method for precast single-story structures, the IN2 curves can be compared to the fragility curves obtained by the multi-stripe analyses in terms of mean collapse acceleration. These curves are obtained by multi-stripe analyses, according to the maximum likelihood method [37].

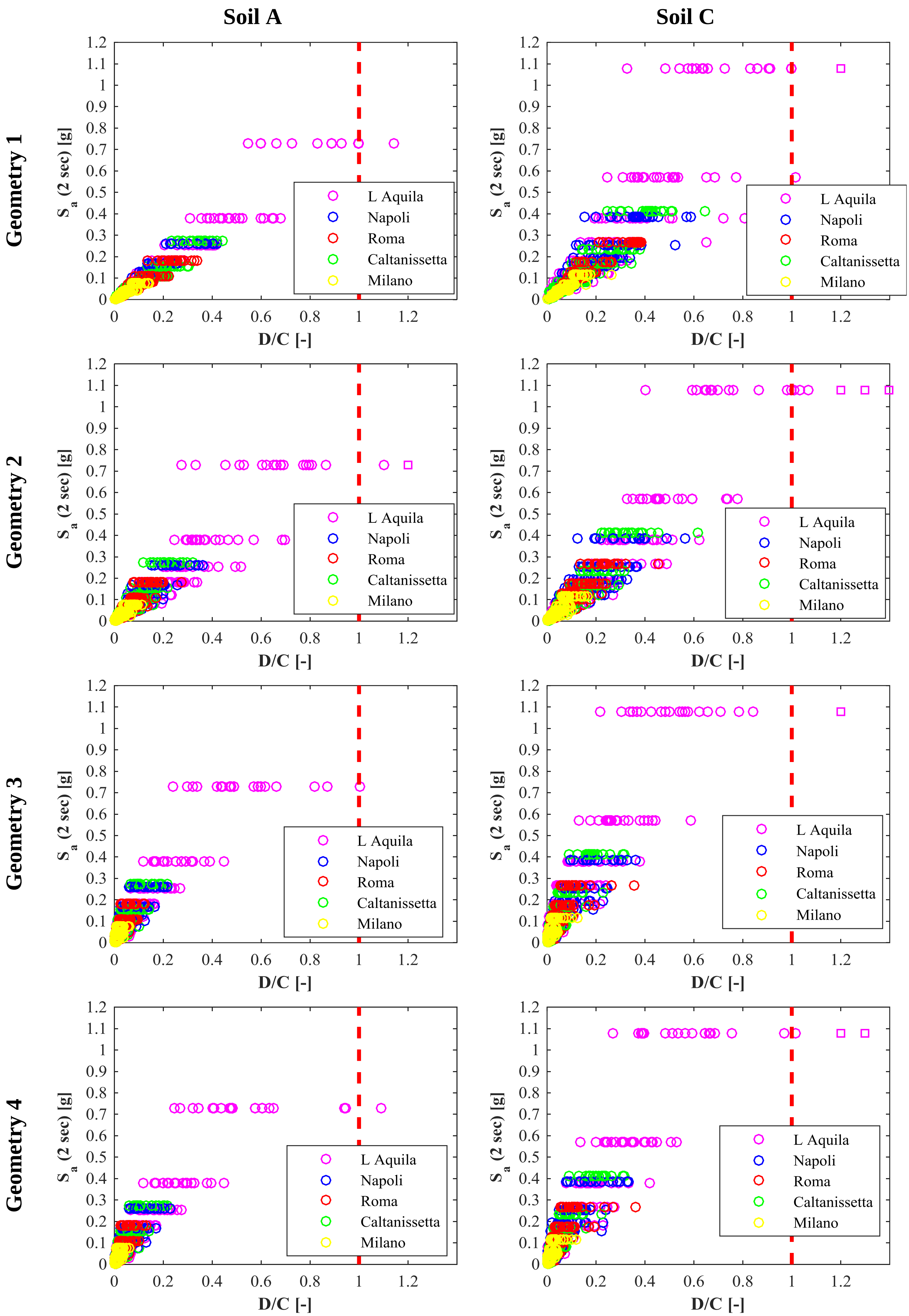


Figure 12 Results of the multi-stripe analysis: D/C ratios along with the spectral accelerations (T=2sec)

Figure 14 shows the IN2 curves for the case studies by adopting the relationships R-μ-T for bare structures [36]. The final point of the IN2 curves corresponds to the attainment of the collapse displacement. In the case of fragility curves, the median value of Sa is assumed as the collapse spectral acceleration. For the two case studies on soil A the values of the collapse S_a in the two methods are very similar (the maximum difference between the two methods is equal to 10%), while in the case of the structures on soil C the differences are about 29%. The results of the IN2 method are consistent with the fragility study; moreover, the method outcomes are consistent with the overall behaviour of the structures in terms of influence of both the geometry and the soil type. Therefore, the IN2 method is adopted in the following in order to define the fragility of the case studies at the collapse limit state, in terms of PGA (Table 11) and S_a.

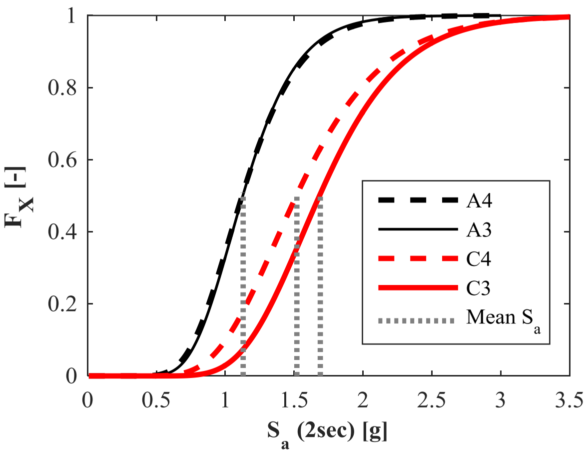


Figure 13 Fragility curves for Geo 3 (red line) and Geo 4 (black line) on soil A (solid line) and soil C (dashed line)

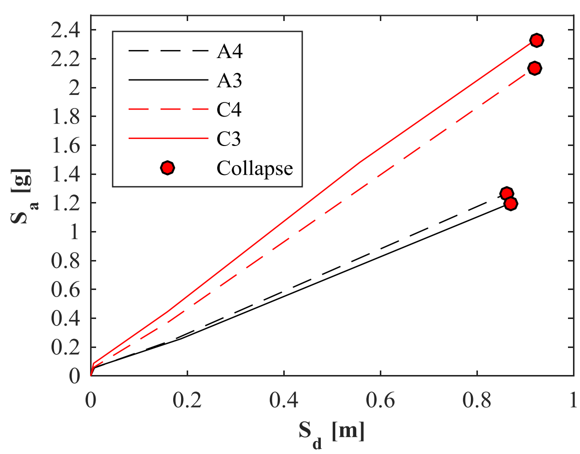


Figure 14 IN2 curve for Geo 3 (red line) and Geo 4 (black line) on soil A (solid line) and soil C (dashed line)

Since the reliability of the IN2 method has been proved in the previous comparison (Figures 12 and 13), this method is performed to assess the safety of all the case-studies with respect to the collapse. Table 10 shows the following parameters:

- PGA_c is the capacity of the structures evaluated by the IN2 method at the collapse displacement.
- PGA_d is the seismic demand, defined as the peak ground acceleration providing the collapse displacement.

By applying the IN2 method, the safety of all the case studies is demonstrated with respect to the collapse. The values of capacity in terms of PGA at the collapse are significantly influenced by the seismicity of the site; the value at L'Aquila is always 40% larger than the PGA at Milano. This difference is larger for the highest structures (H=9m) and for soil C.

The large capacity over demand ratios are consistent with some experimental evidences as well as recorded damage during recent seismic events. The good seismic performance of precast buildings was demonstrated in Kramar et al. [14] by the stable response under seismic actions of the buildings up to large value of drift (8%). Moreover, recent seismic events showed the good performance of the most recent structures under the earthquake because of the use of modern provisions both in Turkey [38] and in Italy [39].

Table 11 Results in terms of collapse capacity (PGA_c) and collapse demand (PGA_d) by means of IN2 method

	Site	PGA_c	PGA_d		Site	PGA_c	PGA_d
Geo 1 – Soil A	L'Aquila	1.09	0.26	**Geo 1 – Soil C**	L'Aquila	1.49	0.36
	Napoli	1.10	0.19		Napoli	1.09	0.27
	Roma	1.27	0.12		Roma	0.94	0.17
	Caltanissetta	0.42	0.07		Caltanissetta	0.89	0.10
	Milano	0.64	0.05		Milano	0.85	0.070
	Site	**PGA_c**	**PGA_d**		**Site**	**PGA_c**	**PGA_d**
Geo 2 – Soil A	L'Aquila	1.57	0.26	**Geo 2 – Soil C**	L'Aquila	1.34	0.36
	Napoli	1.51	0.19		Napoli	1.21	0.27
	Roma	1.91	0.12		Roma	1.54	0.17
	Caltanissetta	0.57	0.07		Caltanissetta	0.53	0.10
	Milano	0.93	0.05		Milano	0.79	0.070
	Site	**PGA_c**	**PGA_d**		**Site**	**PGA_c**	**PGA_d**
Geo 3 – Soil A	L'Aquila	0.81	0.26	**Geo 3 – Soil C**	L'Aquila	0.90	0.36
	Napoli	0.79	0.19		Napoli	0.57	0.27
	Roma	0.94	0.12		Roma	0.67	0.17
	Caltanissetta	0.34	0.07		Caltanissetta	0.29	0.10
	Milano	0.44	0.05		Milano	0.35	0.070
	Site	**PGA_c**	**PGA_d**		**Site**	**PGA_c**	**PGA_d**
Geo 4 – Soil A	L'Aquila	0.82	0.26	**Geo 4 – Soil C**	L'Aquila	0.88	0.36
	Napoli	0.81	0.19		Napoli	0.59	0.27
	Roma	0.96	0.12		Roma	0.68	0.17
	Caltanissetta	0.35	0.07		Caltanissetta	0.29	0.10
	Milano	0.46	0.05		Milano	0.36	0.070

4.3 Some considerations about modelling assumptions

In order to validate the performed study in terms of modelling approaches/assumptions, the following results are presented for the case study Geo3-AQ-SoilA.

Figure 15 shows the fragility curves obtained for this case study with (blue curve) and without (red curve) modelling the cracking point in the plastic hinge. In this case, the influence of the modelling assumption is negligible and the median value at the collapse limit state does not change. This happens for all the analyzed cases, because in ductile structures the collapse behavior is not much influenced by the cracking behavior. Figure 16 shows the influence of P-Δ effects on the seismic response of the same case-study. In this case, the geometrical nonlinearities influence the behavior: the median value of the spectral acceleration at the collapse limit state with P-Δ effects is 11% smaller than the value without these effects. This conclusion can be generally extended to all the other analyzed cases.

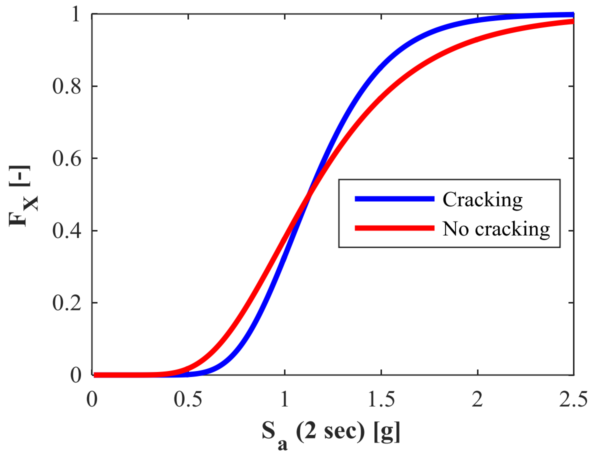


Figure 15 Comparison between fragility curves with and without modelling cracking (AQ-SoilA-Geo3)

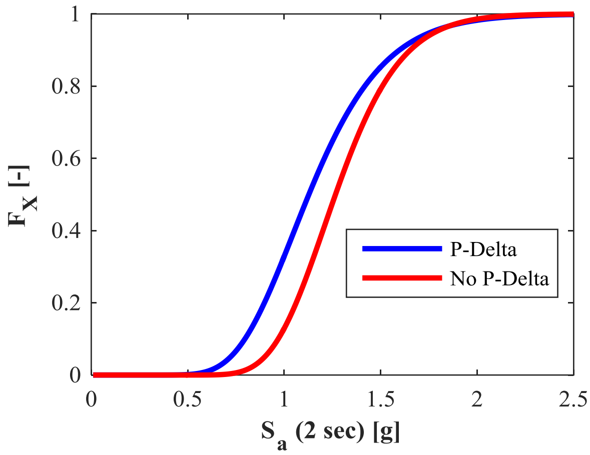


Figure 16 Comparison between fragility curves with and without P-Δ effects (AQ-SoilA-Geo3)

5 Conclusions

The seismic performance of industrial single-story RC precast buildings is evaluated by means of nonlinear dynamic and static analyses on a set of buildings, designed according to modern building codes. According to the results of the design, it is found that most of the structures are influenced by the seismic details required by the code; in many cases the seismic action does not give the largest demand on the structural elements.

By analysing the seismic performance of the structures, the following conclusions can be drawn.

- The capacity of the structures is quite constant with the geometry and it is lightly influenced by the seismicity of the site because of the seismic details and the design overstrength.
- The results of the multi-stripe analyses show a large overstrength of the structures, which decreases as the site seismicity increases; in the case of low seismicity, the seismic demand on structures can be very low even for an intensity measure with a return period of 100000 years. Only for the case studies in L'Aquila on soil C at the maximum intensity level some collapses occurred.

The overstrength has a fundamental role in the seismic response of the structures. The minimum provisions and the wind and static loads significantly influence the final strength. The capacity of the structures is quite constant with the geometry and it is lightly influenced by the seismicity of the site because of the wind and static loads, the seismic details and the design overstrength. The results of the multi-stripe analyses show a large overstrength of the structures, which decreases as the site seismicity increases; in the case of low seismicity, the seismic demand on structures can be very low even for an intensity measure with a return period of 100000 years. Only for the case studies in L'Aquila on soil C at the maximum intensity level some collapses occurred.

The IN2 method is defined as the most feasible analysis method for assessing the capacity of precast one-story buildings at ultimate limit states. The large overstrength of the structures require very large intensities in order to achieve the collapse of the columns. Such intensities cannot be simulated with real records and the amplification factors become unrealistic.

The modelling approach is also validated: 1) the cracking of the section can be neglected in the assessment of the seismic safety at ultimate limit states and 2) the geometrical nonlinearities significantly influence the seismic response of precast structures, given the flexibility of the slender columns as well as the pinned connections to the beams.

The study results are valid for structures with strong connection between the structural elements. Such hypothesis is guaranteed by the capacity design approach provided by the modern codes. The influence of the cladding panels is not considered in the structural model and, moreover, the failure of these non-structural elements is not considered in the definition of the attainment of the limit state, i.e. by limiting the maximum drift capacity of the columns.

Acknowledgments

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