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SEISMIC ASSESSMENT OF EXISTING ITALIAN RC PRECAST STRUCTURES

Chiara Di Salvatore[1], **Gennaro Magliulo**[2,3]

[1] University of Naples Federico II, Department of Structures for Engineering and Architecture,
Via Claudio 21, 80125 Naples, Italy
e-mail: chiara.disalvatore@unina.it

[2] University of Naples Federico II, Department of Structures for Engineering and Architecture,
Via Claudio 21, 80125 Naples, Italy
e-mail: gmagliul@unina.it

[3] Construction Technologies Institute, National Research Council,
Via Claudio 21, 80125 Naples, Italy

Abstract

The paper presents a comprehensive seismic assessment of an existing single-story reinforced concrete (RC) precast structure, resembling the typical characteristics of Italian industrial facilities. For a more realistic evaluation, a simulated design is performed, accounting for old codes and seismic zonation. Then, state-of-the-art nonlinear models are developed in OpenSees, considering or not the contribution of lateral cladding, made by masonry infills. Nonlinear dynamic analyses are performed, achieving a wide and accurate seismic assessment. Obtained outcomes highlights the main vulnerabilities of existing precast buildings in Italy, providing useful insights on their seismic behavior. Lastly, failure rates are evaluated and presented.

Keywords: reinforced concrete precast structures, seismic assessment, friction connections, masonry infills, nonlinear analyses, fragility functions.

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1 INTRODUCTION

The paper reports the outcomes of a wide research project (RINTC-E 2019-2021) [1], carried out by the ReLUIS network and funded by the Italian Department of the Civil Protection, aiming at the determination of the seismic risk of existing Italian buildings. Several structural types were investigated: ordinary reinforced concrete buildings for residential use, steel industrial buildings, masonry buildings and industrial RC precast structures.

The here presented paper focuses on existing RC precast buildings, whose seismic vulnerability has been widely recognized after past seismic events [2-5]. Lack of proper connection systems between structural elements, and/or between structural and nonstructural components, and poor reinforcement details for columns were found to severely affect the response of such buildings against horizontal actions. In particular, before 1987 in Italy, in seismic zones, it was possible to design simply supported elements relying only on friction for the stress transfer. Classical examples of such practice are the beam-to-column connections, often realized through the interposition of a rubber (in general, neoprene) pad between structural members. In this case, seismic actions can cause the attainment of the friction threshold and the occurrence of relative displacement between the elements, with consequent dislocation of the beam up to the loss of support. Also, the role of nonstructural components, such as the lateral cladding, was always misjudges, ignoring that the collapse of these elements can cause injuries or heavy economic losses. Despite the potential catastrophic consequences of such construction solutions, only recently the scientific community has begun to deepen this issue [6-9]. The presented work is framed in this context, trying to provide a useful contribution to the literature on the topic.

The following sections describe the case-study building, the nonlinear modeling approach according to the latest state of the art in scientific literature, and the outcomes of vulnerability and fragility assessment, delivering insights and useful considerations on the seismic behavior of existing RC precast structures.

2 DESCRIPTION OF THE CASE-STUDY STRUCTURES

The case-study structure is selected in order to represent the typical configuration of existing Italian RC precast structures. It is a single-story building with a wide rectangular area (Figure 1), made of precast structural elements, i.e., columns and prestressed beams. The transversal direction (X) of the structure is covered by a single principal truss beam, even if three internal columns, serving as lateral frame for the tailgate, create a four 6 meters bays layout; in the longitudinal direction (Z), nine 5 meters bays are provided, characterized by secondary U-shaped beams, running along all the perimeter of the building. Precast prestressed double T elements are used as roof covering, finished with a poured in situ concrete slab on the external side. At the base, isolated socket foundation are provided for the columns. Regarding the beam-to-column connections, different solutions are detected for principal and secondary beams; in the former case, a 1 cm thick-neoprene pad is inserted at the beam-column contact surface, achieving a friction joint, whilst, in the latter case, a dowel connection is detected. Lateral cladding consists of 25 cm thick masonry walls, as commonly used in the past, before the advent of RC panels; they are symmetrically distributed around the perimeter of the building (blue bays in Figure 1). The structure is a real building design in 1973 in Naples (a_g = 0.168 g), on flat ground and a C soil type. However, other two Italian cities are considered as sites, in order to investigate the influence of the seismic hazard on the structural performance: L'Aquila (a_g = 0.261 g) and Milan (a_g = 0.049 g). It is worth specifying that Naples did not fall in any seismic zone in the Seventies, thus the building was designed without any seismic prescriptions. On the contrary, since L'Aquila was deemed a seismic city in the Seventies, a simulated design is performed for this case-study structure; as a result, reinforcement of both columns and principal beams increases whilst dimensions can be kept the same. Also, no changes are needed for beam-column joints. The structure in Milan is identical to the one in Naples, since Milan was not classified as seismic prone area at the time of construction.

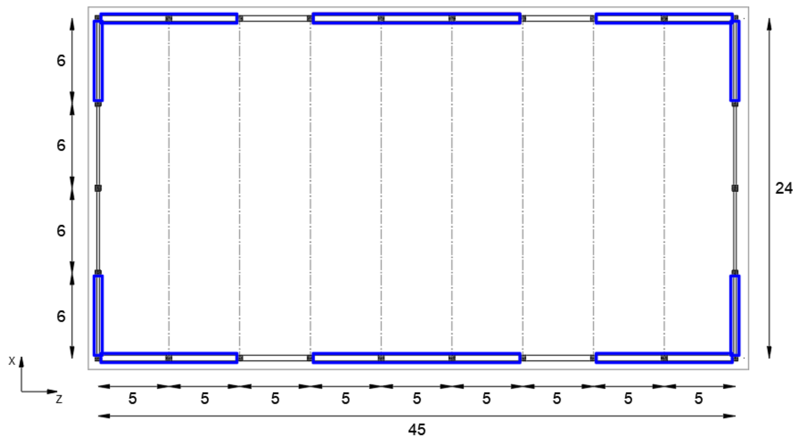


Figure 1. Plan view of the case-study building.

3 NONLINEAR MODELING

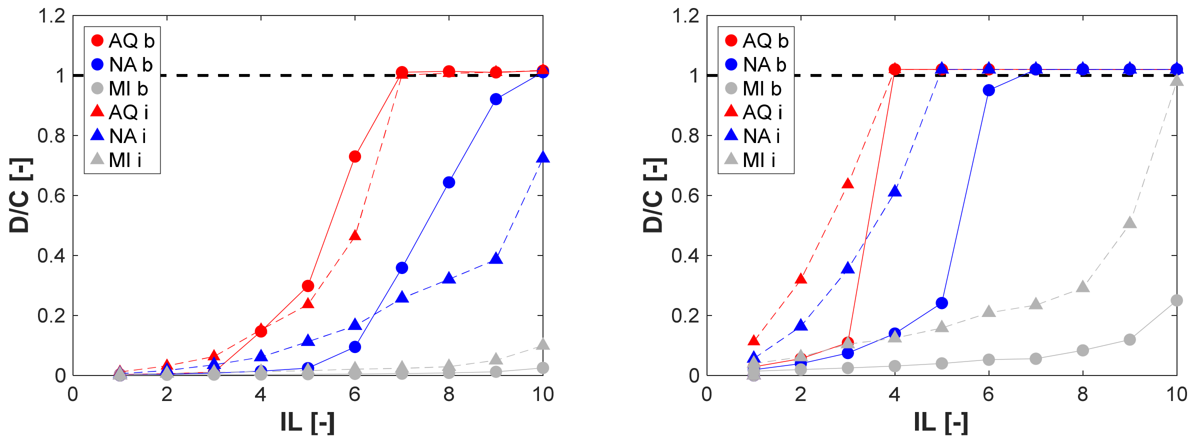
Modeling is performed according to the literature state-of-the-art in OpenSees [10]. In order to investigate the influence of the lateral cladding, both the bare and the infilled configurations are assessed. Beams are represented as elastic elements, whilst a lumped plasticity approach is chosen for the columns. Plastic hinges are introduced at the columns base, according to the cantilever scheme, characterized by a moment-chord rotation backbone working in series with the elastic column element. Formulations by Fardis et al. (2003) [11] are implemented to describe the monotonic behavior of hinges, while hysteretic properties are reproduced applying the model by Ibarra et al. (2005) [12]. A rigid diaphragm is implemented for principal beams, since the presence of the 5 cm thick concrete slab on the roof covering. The mass of the deck, including the beams, is lumped in the center of the deck, whilst the mass of columns and infills is distributed along the vertical elements. Columns are fully restrained at the base. Beam-to-column connections are reproduced depending on the typology of the beam: for secondary beams an internal hinge is applied; for principal beams, friction connections are explicitly modeled through zero-dimension elements inserted between the connected members, reproducing the shear stiffness of the neoprene pad and the sliding of the beam on column head, once the friction strength is attained. Mean mechanical properties of structural materials are obtained from relevant database in literature [13, 14]. Data suggests a mean compressive strength for the concrete equal to 21.03 MPa, and a mean yielding strength for the reinforcement equal to 535 MPa.

For the infilled configuration, bare models are enriched by the explicit representation of the masonry infill walls. The single strut approach is implemented, considering, for each infilled bay, two diagonal concentric struts, working in compression only. Each strut is characterized by a trilinear force-displacement response according to the model by Decanini and Fantin [15]. The backbone is characterized by an elastic branch up to the first macro-cracking, a hardening branch up to the maximum strength and a softening branch up to the residual strength, set equal to 0 in this application. The maximum strength can be obtained evaluating the minimum resistance given by the four possible failure modes: diagonal tension, sliding shear, corner crushing and diagonal compression. Values for the fundamental displacements are derived from the literature [16-18]. Mechanical properties of the masonry are obtained from literature and codes [19-21].

4 NONLINEAR ANALYSES

Two conventional limit conditions are investigated for the case-study buildings: the global collapse condition (GC) and the usability-preventing condition (UPD). The former identifies the complete disruption of the structure, whilst the latter represents the damage level responsible for the interruption of use of the building.

In view of the seismic assessment, it is necessary to link proper damage occurrences to the specified limit conditions. In turn, the definition of the significant occurrences enables to set relevant structural capacities, to be compared with the seismic demand coming from nonlinear dynamic analyses. The condition corresponding to the GC state is assumed as the first beam loss of support phenomenon, in both the directions and for both the configurations. Thus, the relevant engineering demand parameter (EDP) is selected as the relative principal beam-column displacement; the capacity is set equal to the friction connections support length, that is 28 cm in the transversal direction and 18 cm in the longitudinal direction. It is worth specifying that the nonlinear dynamic analyses algorithm is modified in order to stop the analysis at the first loss of support occurrence, since the successive response simulation would not be meaningful. For the UPD condition, a distinction between the bare and the infilled configuration is needed. For the bare structures, the UPD condition was identified as the beam-column relative displacement reaches the 10% of the support length. For the infilled structures, three conditions are assessed: i) the attainment of a beam-column relative displacement equal to 10% of the support length, ii) the attainment of the ultimate deformation for a single infill wall, iii) the attainment of the peak strength for half of the infill walls of the building. For all cases, the most unfavorable condition is the one related to the friction connections. Thus, the EDP always consists in the relative displacement between principal beams and columns. Nonlinear dynamic analyses are performed on all the structural models in the form of multi-stripe analyses (MSA). Ten intensity levels are considered, each one characterized by twenty bi-directional seismic inputs, for a total of two hundred analyses for each model. Ground motions are selected so as to match the target spectrum in T_1 (fundamental period of the buildings, equal to 2 s for the bare structures and to 0.5 s for the infilled ones), according to the Conditional Spectrum approach [22]. Figure 2 reports, for each stripe, the maximum value of the mean demand-capacity ratios (D/C) of the two directions, for (a) the GC condition and (b) the UPD condition. Sites differ in colors (L'Aquila in red, Naples in blue and Milan in grey), whilst the structural configurations for markers and lines style (bare models represented with continuous lines and circles, infilled models with dashed lines and triangles).



a) b)

Figure 2. Mean D/C ratios from dynamic analyses for a) GC condition and b) UPD condition.

Outcomes show that, for the same configuration, the seismic hazard plays the major role in determining the vulnerability of buildings. Indeed, the structures in L'Aquila are more vulnerable (i.e., experience more failures) with respect to the ones in Naples, which in turn are more vulnerable than the ones in Milan. This highlights the inefficacy of old Italian seismic codes and prescriptions. Also, as expected, more failures are detected for the UPD condition with respect to the GC condition, since the lower level of damage represented. Concerning the comparison between the bare and the infilled configuration, it can be noted that, for low seismic intensities, the infilled configuration is more vulnerable than the bare one, due to the higher stiffness and, thus, the higher seismic forces; on the contrary, for high seismic intensities, an opposite trend is observed, ascribable to an out-of-phase oscillations phenomenon occurring between the columns and the principal beam-deck system.

5 SESMIC RISK ASSESSMENT

In order to achieve a comprehensive seismic risk assessment, failure rates are calculated (Table 1), integrating the product of the fragility function and the derivative of the hazard curve [23].

Outcomes confirm that, for the UPD condition, the consideration of the infill walls leads to a lower structural reliability, whilst the opposite occurs for the GC condition, for the sites of L'Aquila and Naples. For the case of Milan, the coefficient 10^{-5}, added to the evaluated failure rates due to the truncation of the hazard curve at IL10 (T_R = 100000 years) [24], is significantly higher than the calculated value, making it negligible. Also, the hierarchy among sites persists, attributable to the predominant role of seismic hazard. As expected, for each configuration, structures in L'Aquila always shows the highest failure rates, whilst Milan the lowest ones. Lastly, given the slighter level of damage, the UPD conditions presents failure rates higher than the GC ones.

Case	Limit condition	
	GC	UPD
AQ b	1.13E-03	6.39E-03
AQ i	7.51E-04	8.15E-03
NA b	1.57E-04	1.42E-03
NA i	6.95E-05	2.48E-03
MI b	1.00E-05	2.35E-05
MI i	1.00E-05	5.50E-05

Table 1. Failure rates.

6 CONCLUSIONS

The paper investigates the influence of different parameters on the seismic risk of Italian existing RC precast structures. A case-study building is selected as representative of the Italian industrial building in the Seventies, consisting in a single-story structure with prestressed beams, friction connections and thick masonry infill walls. The building is located in three Italian sites, characterized by different level of seismic hazard. Numerical models are developed in OpenSees, accounting for the friction joints behavior and for the presence of the lateral cladding; in detail, two structural configurations are considered, the bare structure and the infilled structure. Nonlinear dynamic analyses with a multi-stripe approach are performed in order to evaluate the structural vulnerability for two limit conditions, the Global Collapse and the Usability Preventing Damage, corresponding to the overall structural disruption and to the damage level responsible for the use interruption, respectively. Lastly, failure rates are derived. The following conclusions can be drawn:

- the main source of seismic vulnerability for existing precast RC structures seems to lay in friction joints. Loss of support phenomena occur before the rise of plastic damage at the base of the columns, leading to the collapse of the structure (GC) in the elastic field. Also, for UPD condition, a significant dislocation of the beam from its initial position can be detected before the occurrence of severe damage to the infill panels;
- masonry infill walls considerably increase the stiffness of the buildings, lowering the structural period and involving higher seismic forces. Indeed, the infilled configurations achieve more failures, at least for low seismic intensities. On the contrary, the bare structures are more vulnerable than the infilled ones for high seismic levels, due to out-of-phase oscillations, biggering the relative beam-column horizontal displacements;
- the vulnerability of the structure increases increasing the site hazard, demonstrating the inefficiency of the past seismic code in Italy;
- failure rates provide a synthetic expression of the seismic risk of the buildings, reflecting the nonlinear dynamic analyses outcomes.

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