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THE ROLE OF MASONRY INFILLS ON THE SEISMIC PERFORMANCE OF EXISTING RC PRECAST STRUCTURES

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Abstract: *In recent years, several investigations have been performed on the seismic performance of existing RC precast structures, highlighting their great vulnerability, especially concentrated in structural and nonstructural joints. However, the influence of the external cladding is almost never considered. As a matter of fact, when the cladding interacts with the main structure, as in the case of typical masonry infills, the seismic response of the building under horizontal forces changes, mainly due to the increase in the global stiffness. The present work aims at the evaluation of the seismic response of infilled RC precast structures, in order to assess the influence of nonstructural components intended for cladding on the behavior of the whole structure. To this end, a case-study structure is selected, consisting in an existing one-story RC precast building located in a very seismic-prone zone in Italy. As common for old industrial precast buildings in the Country, the structure is provided with friction connections for the beam-column joints and with strong masonry infills. Two tri-dimensional nonlinear models are developed in OpenSees for the case-study structure, one without masonry infills (i.e., bare model) and one considering the masonry infills (i.e., infilled model). Nonlinear static and dynamic analyses, namely multi-stipe analyses, are performed on both the models, pointing out the stiffening effect of the cladding on the whole building and the consequent beneficial effect in reducing the displacement demand in seismic conditions. The paper provides useful information for the modeling of masonry infills in RC precast structures and demonstrates the need to consider nonstructural elements for a proper seismic performance evaluation of such building typology.*

Keywords: *existing RC precast structures, masonry infills, seismic assessment.*

Introduction

In the last decades, many scientific studies have been focused on the seismic performance of RC precast structures in Italy (Magliulo, Fabbrocino and Manfredi, 2008; Faggiano et al., 2009; Toniolo and Colombo, 2012, Magliulo et al., 2014; Liberatore et al., 2013; Casotto et al., 2015; Ercolino, Magliulo and Manfredi, 2016; Demartino, Monti and Vanzi, 2017; Savoia, Buratti and Vincenzi, 2017; Bosio et al. 2022). The interest on this topic is justified by the disastrous aftermaths of recent earthquakes that have hit the Country; indeed, existing RC precast structures showed high vulnerability against seismic loads, mainly due to inadequate seismic detailing and/or weak connection systems. This latter aspect has proved to be very crucial in the evaluation of the seismic response of RC precast buildings. In particular, structural joints between beams and columns, or roof members and beams, were usually achieved simply placing the beam/roof element upon the corresponding column/beam without any mechanical device; the stress transfer was then committed to the friction between the surfaces in contact. Often, in order to avoid heavy stress concentration and consequent localized concrete crushing, a rubber (usually neoprene) pad was placed between the elements in touch. This was allowed in Italy until the National code DM 1987 regardless the seismic hazard of the site. However, even after this code, friction connections were still used since a large part of the National territory was not deemed as a seismic prone zone. Also, when mechanical devices were implemented, e.g., dowel connections,

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the design was unreliable and lacking of proper construction details. This results in structural connections practically always under-designed, and consequently underperforming, due to inefficient or absent code regulations (Cimmino, Magliulo and Manfredi, 2020). This is the reason why, nowadays, many research studies tackle the issue related to this kind of dry joints in existing structure, allowing the achievement of robust and reliable formulations, useful for the design and/or the assessment phase. Attention has been paid also to non-structural joints, connecting cladding panels to the bearing structure (Biondini, Dal Lago and Toniolo, 2013; Belleri et al., 2017; Zoubek, Fischinger and Isakovic, 2016). Indeed, in past earthquake events, the weakness of such connections led to the detachment of heavy cladding elements, involving the necessary interruption of industrial activities, and seriously jeopardizing human life. It is worth noting that this kind of cladding is relatively modern; up to 1980, the most spread kind of cladding for industrial buildings was constituted by robust masonry infills, i.e., masonry walls with high thickness (25-35 cm). Nonetheless, very few studies focus on the behavior of RC precast structures with such kind of partitions (Bosio et al., 2022), representing a serious lack in the seismic assessment of the Italian precast building stock.

This work arises in the framework of a wider research activity regarding the seismic risk of several Italian structural typologies (Iervolino et al., 2022), i.e., ordinary RC buildings, precast RC buildings, steel industrial buildings, masonry buildings and RC bridges. This paper reports a part of the outcomes concerning typical Italian existing RC precast structures. In particular, the seismic vulnerability of a case-study structure is assessed in a high seismic hazard condition. The structure features cantilever columns and simply supported principal horizontal elements; the cladding function is played by robust masonry infills, i.e., masonry walls with high thickness. A tri-dimensional nonlinear model is developed in OpenSees (PEER, 2007), for the bare and the infilled building, and seismic assessment is performed for both the structural layouts through nonlinear static and dynamic analyses. Outcomes demonstrate that the highest source of vulnerability is represented by friction connections between columns and principal beams, since global collapse always occurs for loss of support of principal beams sliding on their support. Comparison between the bare and the infilled layout enables the detection of the cladding-structure interaction effect, mainly identifiable as a stiffening effect, with a consequence lowering in the vibrational periods and rising in the seismic forces.

Case-study structure

Description

The building under consideration is a real existing structure dating back to 1973 and located near Naples, in the South of Italy. According to the codes in force at the time of construction, the site did not fall in a seismic prone zone yet; thus, the structure was designed without any seismic provision. The building resembles the typical configuration of Italian industrial buildings: it is arranged in a single story, with a 24 m single bay in the transversal direction (X direction) and nine bays, spanning 5 m each, in the longitudinal direction (Z direction). A double-slope RC precast truss beam fills the entire length of the transversal bay (Figure 2), supported by 5.95 m cantilever columns at both the extremities through a single layer neoprene pad, achieving a purely friction link. Three internal columns are placed along the external transversal bays, with an infill-bearing function. Secondary beams, characterized by a U-shaped cross-section, run along the whole perimeter of the building, with the function of both collect rainwater and connect the transversal frames of the structure. These horizontal elements are constrained to the columns' head by means of a ϕ14 steel dowel for each end. Isolated socket foundations provide a fixed restrain at the base of columns. Roof covering is made up of double-T prestressed RC precast elements, solidarized by a cast in situ 5 cm thick concrete slab. Lateral cladding is accomplished by means of 25 cm thick masonry infills, referred to as DOPPIO-UNI masonry infills; in Figure 1a, bays highlighted in red are the ones left open, without any cladding system. Design material properties correspond to the ones associated to C45/55 class for the concrete and to FeB44K for the steel.

With the aim of assessing the effectiveness of the past seismic provisions, the building under consideration is relocated in another Italian city, which was assumed as a seismic prone area at the time of the building construction, L'Aquila. Thus, a simulated design following the construction codes of the Seventies' (Legge n. 64, 1974; CNR-UNI 10012, 1967) has been performed. The re-design process leads to the same columns' cross-section dimensions (40x40) cm^2 and the same shear reinforcement, corresponding to the minimum amount (2ϕ5/25 cm), but different longitudinal reinforcement; in particular, the geometric percentage of reinforcement, ρ_l, for main columns increases by more than double, passing from nearly 0.70% of the non-seismic case to nearly 1.5% in the seismic condition. Horizontal elements and structural connections are unchanged.

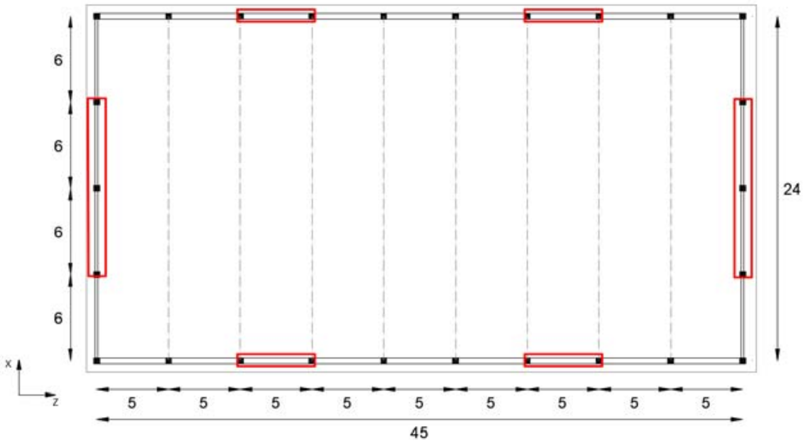


Figure 1. Plan view of the case-study structure.

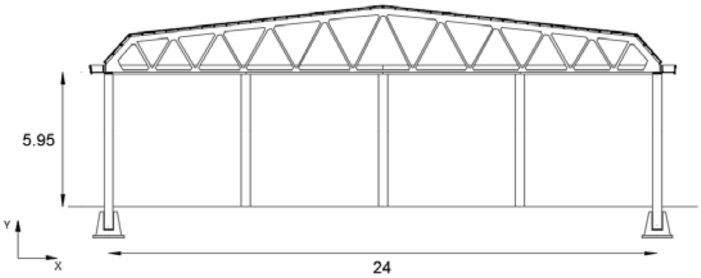


Figure 2. Transversal view of the case-study structure.

Nonlinear modeling

A tri-dimensional nonlinear model is developed in OpenSees (PEER, 2007), for both the bare and the infilled structural layout. Given the building structural typology, both principal and secondary beams do not belong to the system in charge to resist seismic forces; thus, they are modeled as elastic elements, by means of the *ElasticBeamColumn* element, assuming mean cross-section dimensions due to their peculiar and/or variable shape. Columns are represented as simple cantilever members, made up of two sub-components working in series: i) the elastic element and ii) the plastic hinge at the base. To achieve this configuration, a *zerolength* element is implemented at the base of each elastic column, providing the plastic flexural response. The assigned moment-rotation backbone is characterized by three linear branches, i.e., the elastic branch up to the attainment of the yielding point (θ_y, M_y), the hardening branch up to the capping (θ_{cap}, M_{cap}), and the softening branch up to the ultimate chord rotation, θ_u, corresponding to a null residual bending moment; moreover, cyclic degradation in terms of stiffness and strength is taken into account. Numerical implementation of such plastic response is obtained by means of the *Modified Ibarra-Medina-Krawinkler Deterioration Model with Peak-Oriented Hysteretic Response*, defining the θ_y according to Fardis and Biskinis (2003), θ_{cap} and θ_u according to Haselton (2006) and the degradation parameter λ through the expression from Ibarra, Medina and Krawinkler (2005). Roof covering is not explicitly modeled, and it is taken into account only in terms of seismic mass and gravity load. However, the presence of the 5 cm thick top covering upon them allows the implementation of a rigid diaphragm constraint, making the floor slab indefinitely rigid in its own plane and enabling a stiffness-based redistribution of seismic forces among the underlying elements. With a view to the seismic assessment of the building, a crucial aspect is represented by the connections between beams and columns. As concern secondary beams, the link is assumed as a perfect hinge and simulated in the numerical model through the *equalDOF* constraint; this assumption is not deemed to compromise the results, since secondary beams are only attached to the perimeter of the structure without any connection to the deck. On the contrary, friction connections between principal beams and columns have to be carefully considered. A *zerolength* element is adopted to reproduce the behavior of such connections; in particular, horizontal response (along X and Z directions) is defined as elastic-perfectly plastic, assuming the elastic stiffness of the neoprene pad to characterize the initial branch and the friction force, evaluated according to the Coulomb formulation, to define the plateau. In this application, the axial force acting on the pad is associated to the gravitational loads, and the neoprene-concrete friction coefficient is evaluated follows the indication of Magliulo et al. (2011), as a result of a wide experimental campaign. In the vertical direction, the axial stiffness of the pad is assigned, whereas rotation around Z axis is left free. For the sake of simplicity, rotations about X and Y axis are fully constrained.

Masonry infills are inserted in the frames by means of two concentric diagonal truss elements, following the modeling approach by Liberatore et al. (2018); each truss reacts only in compression, so as to have a single truss working for each direction of the seismic action. Compression behavior is defined according to Decanini, Liberatore and Mollaioli (2014), considering a quadrilinear force-displacement backbone with a null residual strength (Figure 3). Three points are needed in order to define the response: F corresponding to the first macro-cracking, FC corresponding to the maximum strength and U corresponding to the ultimate deformation. The maximum strength is taken as the minimum among the possible failure mechanisms (diagonal tension, sliding shear, corner crushing and diagonal compression). For each truss, the behavior is implemented by means of the *Concrete 01* material.

Seismic mass of the whole deck, including beams, is concentrated in the center of the mass of the floor, whilst masses of columns and masonry infills are distributed along the vertical members.

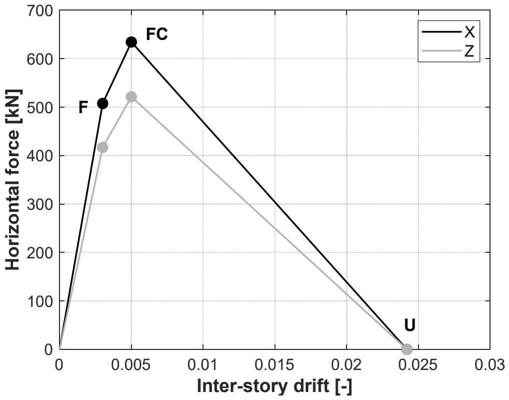


Figure 3. Horizontal force-drift ratio backbones for masonry infills in X and in Z direction.

Seismic assessment

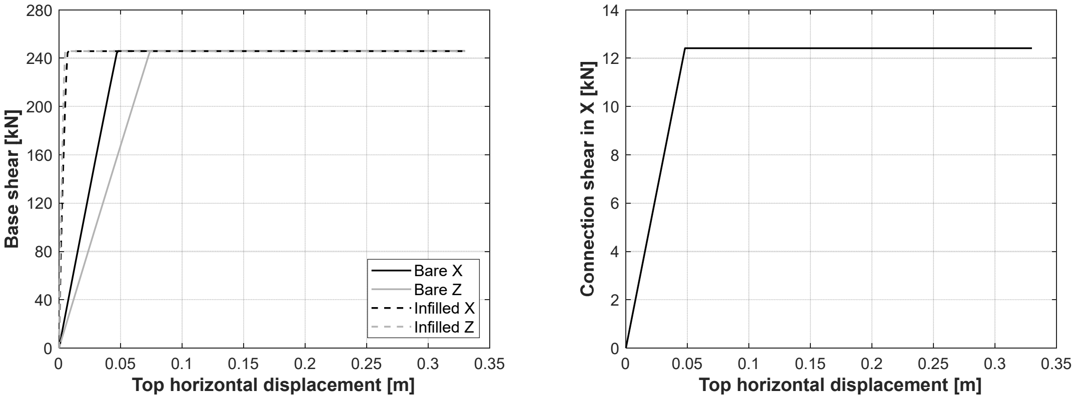
The evaluation of the seismic response of the case-study building is carried out performing both static and dynamic nonlinear analyses. First, in order to obtain a dynamic identification, a modal analysis is also performed on both the bare and the infilled structure. Outcomes are reported in Table 1, in terms of vibrational periods (T) and mode translational direction. From a comparison between the two considered models, the lowering of structural periods in the infilled case is evident, and justifiable in the strong stiffening contribution of the cladding. Furthermore, it is interesting to note that the translational modes in the infilled model are reversed with respect to the bare model; this is ascribable once again to the stiffening contribution of the infills, which is significantly higher in the longitudinal direction with respect to the transversal direction.

Model	1° mode		2° mode	
	T	Direction	T	Direction
	[s]	[-]	[s]	[-]
Bare	1.60	Z	1.35	X
Infilled	0.35	X	0.32	Z

Table 1. Modal analysis outcomes.

Nonlinear static analyses

Figure 4a shows the outcomes of pushover analyses in terms of top horizontal displacement-base shear, on both the bare (solid lines) and the infilled (dashed lines) structures, in each horizontal direction in plan, i.e., X in black and Z in grey. The resulting curves are characterized by two branches: the former elastic and the latter perfectly-plastic. The global failure of the structure is caused by the activation of the sliding of the principal beams on columns head, after the attainment of the horizontal resistance provided by friction connections. The force value corresponding to the plateau coincides with the sum of the horizontal strengths of all the beam-to-column friction connections; indeed, it is found that the activation of friction occurs for all the connections at the same time, thanks to the rigid roof diaphragm. Figure 4b shows the record of the horizontal force acting on a friction connection in the X direction during the analysis on the bare model; if compared with the corresponding global pushover curve, it can be easily observed that the plateau branch is attained at the same analysis step, i.e., the same top displacement. Furthermore, pushover curves allow to detect that loss of support global failure always occurs before the yielding of the columns base. Thus, the structure (both bare and infilled) collapses in the elastic field without showing permanent damage due to plastic deformations. As expected, the infilled layout is stiffer than the bare one; moreover, from the curves can be easily derived that, for the bare case, the Z direction represents the weak direction, whilst, for the infilled case, an inverted trend can be seen, even if in this latter case results in the transversal and in the longitudinal directions are very similar.



a) b)

Figure 4. a) Pushover curves for both the bare and the infilled models; b) horizontal force in a lateral friction connection of the bare model in X direction against top displacement.

Nonlinear dynamic analyses

Seismic inputs are developed in the framework of the RINTC project, in which this work has been conceived and carried out (Iervolino, Spillatura and Bazzurro, 2017; RINTC Workgroup, 2018). A multi-stripe approach is selected in order to perform a nonlinear dynamic assessment of the structure in seismic conditions. Ten increasing intensity measures (IM) are selected, identified by ten increasing return period ([10,50,100,250,500,1000,2500,5000,10000,100000] years). A soil type C is assumed, so as to consider a burdensome amplification due to soil conditions. According to the Conditional Spectrum Method (Lin, Haselton and Baker, 2013a,b), proper bi-directional time-histories are obtained starting from an accurate hazard analysis of the site; indeed, for each return period, a target spectrum can be defined together with a value of the spectral acceleration corresponding to the first period of vibration of the structure ($S_a(T_1)$). For the bare structure, the first period is assumed equal to 2.0 s, whilst for the infilled model it is 0.5 s. Spectral acceleration for both the modeling cases are reported in Table 2. Then, ground motions are selected from Italian and American seismic database (Itaca, NGWest), and properly scaled in order to have a spectrum matching the target spectrum defined for each intensity level. For each intensity level, twenty accelerograms are provided, for a total of two hundreds nonlinear dynamic analyses.

$S_a(T_1)$ [g]										
IM	1	2	3	4	5	6	7	8	9	10
Bare	0.011	0.026	0.049	0.080	0.124	0.184	0.270	0.379	0.572	1.077
Infilled	0.077	0.181	0.322	0.503	0.754	1.129	1.733	2.481	3.810	7.630

Table 2. Conditional spectral acceleration corresponding to the first period of vibration for the bare and the infilled model.

Seismic assessment is conducted considering a global collapse condition (GC), which identifies the failure of the whole building; because of the peculiar vulnerability provided by friction connections, it is detected when the first loss of support phenomenon occurs. Thus, the maximum recorded relative beam-column displacement in correspondence of friction joints is assumed as seismic demand (D) for each ground motion applied, whilst the capacity (C) is represented by the maximum available support length; this can be evaluated as the sum of the effective support span and the shear deformation of the neoprene pad under the friction force. In Figure 5, Demand-Capacity ratios (D/C) are reported for all the considered intensity levels and for both the assessed models. It is worth highlighting that the seismic assessment is performed in both the horizontal directions in plan; for each ground motion, the maximum value of the Demand-Capacity ratios between the two directions is considered.

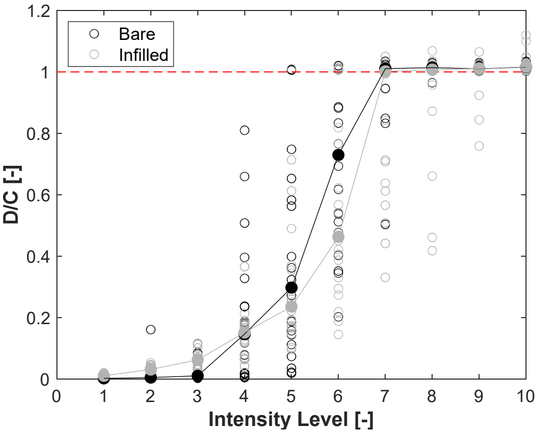


Figure 5. Demand-Capacity ratios for the global collapse condition.

The filled markers connected by the solid line provide the D/C median results. Two opposite trends can be observed. For low intensity levels, the infilled structure is characterized by higher D/C, since the higher stiffness and, consequently, the higher values of the acting seismic forces; as the seismic intensity rises, beam-to-column relative displacements become significant, and the bare structure shows a higher vulnerability. It is found that the bare model response is characterized by a phase shift between the absolute displacements of the end nodes of the zero-length element simulating the friction joint. As opposite to the infilled model, in the bare structure, the absolute displacements recorded at the superior and the inferior nodes of the connection element are strongly out-of-phase; this involves an increase in the relative displacements between principal beams and columns, favouring the depletion of the capacity length.

Conclusions

In this work, an exhaustive seismic assessment is carried out on a typical existing Italian RC precast building. The structure is conceived as a one-story building, with cantilever columns and principal beams in the transversal direction; these horizontal members are simply supported on the columns head by means of a single-layer unreinforced neoprene pad, relying only on friction for the transmission of horizontal loads. As common in industrial buildings in the past, infill cladding is made up of 25 cm thick masonry walls, covering the whole height of the columns. Two tri-dimensional nonlinear models are developed in OpenSees, one neglecting the presence of masonry infills and the other explicitly considering the cladding, in order to detect the role of masonry infills in the seismic vulnerability of the building. Nonlinear static analyses are carried out, allowing detecting the effect of cladding on the structural behavior under horizontal forces and the predominant failure mode. Nonlinear dynamic analyses are also performed, according to a multi-stripe approach, for ten increasing intensity levels. The following findings can be drawn:

- Masonry infills provide a significant contribution in terms of lateral stiffness to the structure, leading to a sharp reduction of vibrational periods and, as a consequence, high seismic forces.
- A damage state corresponding to the global collapse condition, identifiable as the total disruption of the building, is considered for the seismic assessment. The highest source of seismic vulnerability is represented by the beam-to-column friction connections, whose activation leads to premature failure of the building, due to loss of support phenomena.
- The detected failure mode occurs before the yielding of plastic hinges at the base of the structure. This means that, except the small hysteresis provide by friction, the failure is brittle.
- Outcomes for the global collapse condition show that, in IM with low intensities, the infilled structure is more vulnerable than the bare one. The increase in stiffness brings to a significant increase in seismic forces, activating the sliding of the beam upon the neoprene pad. Contrary, for high seismic intensities, the bare structure shows higher D/C than the infilled one; it is found that a considerable out-of-phase oscillation characterizes the deck with respect to the columns heads. This is responsible for the increase in the relative displacement between beams and columns.

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