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INFLUENCE OF CLADDING PANELS RETROFIT ON THE SEISMIC RISK OF AN EXISTING PRECAST BUILDING

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Abstract

Recent research and past earthquakes have shown that precast industrial buildings built in Italy up to the 1980s are characterized by a considerable seismic risk, particularly considering inadequate connections between precast elements. It is therefore necessary to implement seismic retrofit interventions both at a local and global level to protect working activities and human lives.

In this work, the influence of local interventions, particularly of cladding panels, on the evolution of the seismic risk of a reference precast industrial building is assessed. The effects of the retrofitting interventions were verified by comparing the performance of the retrofitted structure with those of the structure in its current state, by means of multi stripe analyses, in terms of demand-capacity ratio associated with each element and each connection of the structural and non-structural system.

Keywords: Precast Structures, Industrial Buildings, Seismic Risk

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1 INTRODUCTION

Single-storey precast buildings represent the most common type of building in the industrial sector in Italy. Before the Ministerial Decree of 3 June 1981, the seismic classification of the Italian territory envisaged its subdivision into two categories: non-seismic zone and seismic zone. Nowadays, a large part of the heritage of precast industrial buildings in Italy is characterized by inadequate structural and non-structural details [1, 2, 3, 4, 5, 6, 7, 8, 9]. This leads to the inadequate seismic behavior observed during past seismic events [10, 11, 12, 13, 14, 15, 16, 17].

The seismic risk associated with one-storey existing precast buildings has been studied in various works [18, 19, 20, 21, 15]. The results obtained confirm the need to implement retrofit interventions to protect working activities and human lives. The Italian building code defines three classes of interventions on buildings: local interventions, improvement interventions, full-adequacy intervention. In the context of industrial buildings, it is important to note that local interventions are preferred over the global retrofit (improvement and full-adequacy interventions) which would require the temporary closure of the industrial activity. For local interventions the code allows the safety check to be performed by referring only to the parts and/or elements affected by the intervention. However, it is interesting to question the effect of local reinforcement interventions on the seismic risk of the entire building.

The aim of this paper is to evaluate the influence of local strengthening interventions on the evolution of the seismic risk of a precast industrial building. For this purpose, the effect of the adjustment of the connections between the external RC cladding panels and the main structural elements of a case study building is studied. Response spectrum analysis was carried out to determine the safety level of the building in its current state. The effects of the retrofitting interventions were verified by means of multi stripe analyses in Opensees [22] and the results are presented as demand-capacity ratio (*D/C*).

2 REFERENCE BUILDING

The reference building (Figure 1) was derived from a previous research work [21]. It is a single-storey building built in the 80s in the municipality of L'Aquila (Italy). The structural details reflect the design criteria established by the standards of the time in seismic area [23].

2.1 Description of the reference building

The vertical bearing structure is made up of 16 precast columns. A double tapered beam with a net span equal to 20m is placed onto the top of each pair of columns, housed inside RC forks; the portal-to-portal distance is 6m. The secondary roof system is made by double-tee roof elements. The cladding is made by horizontally and vertically spanning RC panels arranged along the longitudinal and transverse directions, respectively.

The columns have a square cross-section 50cm x 50cm with 4+4 Φ14 longitudinal rebars and Φ 8 stirrups placed every 200mm. The horizontal cladding panels are connected to the columns by two bearing connections at the bottom of the panel (a bearing bolt laying on a steel bracket) and two retaining hammer-head anchor bolts at the top [11, 12]. The vertical cladding panels are connected to the beam by a hammer-head stripe connection [12, 24].

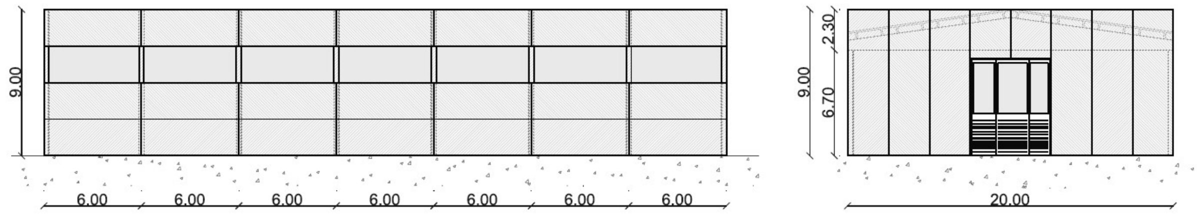

Figure 1: Longitudinal and transverse view of the reference building.

2.2 Level of safety in the as-is condition

The level of safety in the as-is condition was assessed in accordance with Italian technical standards [26] through the coefficient ζ_E, defined as the ratio between the actual capacity and the demand required in the design of a new construction with the same characteristics. The value of ζ_E was evaluated for the columns, the roof element-to-beam connections and the cladding panels-to-structure connections. The demand was assessed through a modal response spectrum analysis. The main characteristics of the adopted finite element model are reported in Figure 2.

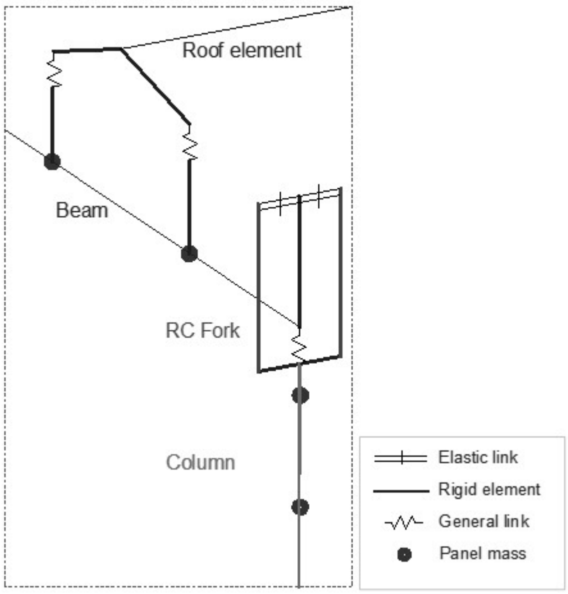

Figure 2: Modeling scheme for the finite element model used in the response spectrum analyses (MidasGen [26])

The main structural elements were modeled using beam type elements while the cladding panels were modeled only in terms of mass in correspondence with their connections with the main structural elements. To account for the cracking in the columns, a stiffness reduction factor equal to 0.27 was calculated as the ratio between the flexural stiffness at yield and the gross stiffness. As for constraints, the columns are fixed at the base while the roof element-to-beam and beam-to-column connections have been modeled using lumped springs ("general link") with a stiffness equal to 20'000 kN/m [27] and 64'000 kN/m [28], respectively. To capture the force on the RC fork, the "elastic link" type was inserted between the beam and the RC fork. The design spectrum in acceleration was defined with a behavior factor (q) equal to 1.5, assuming the topographic category T_1 and soil type C [25].

In determining the ζ_E coefficients, the demand at the roof element-to-beam connections was increased by 30% and for fragile mechanisms (e.g., shear failure), a safety factor of 1.25 was used [25]. The results (Table 1) highlight the deficiencies of the connections between the panels and the main structure with a safety coefficient ζ_E equal to 16% and 41% for the vertical and horizontal panels respectively. For the roof element-to-beam connections and columns the coefficient ζ_E is respectively 92% and 53%.

	ζ_E		**Demand**		
Connection	$F_{,X}$	$F_{,Y}$	$F_{,X}$ (kN)		$F_{,Y}$ (kN)
Roof element - Beam	0.92	5.31	45.22		7.82
			$F_{,X}$ (kN)	F_a (kN)	d (m)
Panel – Column horizontal cladding	2.52	0.41	6.52	2.38	0.064
Panel-to-Beam Vertical Cladding	0.67	0.16	7.30	2.93	0.160
Element	Bending	Shear	Bending (kNm)		Shear (kN)
Column	0.53	1.10	344.20		57.70

Table 1: Current levels of safety and demand in terms of force (from analysis: $F_{x/y}$; analytically: F_a) and displacement. Notes: x and y indicate the longitudinal and transverse direction, respectively.

3 RETROFIT MEASURES AND NON-LINEAR MODELING

In this section the retrofit measures selected for the local retrofit of cladding panels are presented. The selected retrofit consists of placing additional steel brackets (two at the bottom and two slotted brackets at the top) between each horizontal panel and the columns and one slotted bracket between each vertical panel and the beam.

The two-steel brackets at the bottom of the horizontal cladding panels are designed to carry the entire weight of the panel while the steel brackets arranged at the top of the panels (vertical and horizontal) are designed to counteract the overturning mechanism. Additionally, these top connections must accommodate the displacement demand in the panel plane. To this end, on the panel side, the steel angle is slotted for +/- 64 (mm) and +/- 160 (mm) respectively for the horizontal and vertical panels.

The out-of-plane design force (F_a) of the top connections can be determined by applying the formulation expected for non-structural elements [25]:

$$F_a = \frac{S_a W_a}{q_a} \tag{1}$$

where S_a is the seismic coefficient applicable to non-structural element; W_a and q_a are the weight of the element and its behavior factor respectively. The seismic coefficient may be calculated using the formula proposed by EN 1998-1 [30].

$$S_a = \alpha S \left[3\left(1 + \frac{z}{H}\right) \Big/ \left(1 + \left(1 - \frac{T_a}{T_1}\right)^2\right) - 0.5 \right] \tag{2}$$

Where α is the ratio of the design ground acceleration (a_g) on soil type A to the acceleration of gravity (g); S is the soil factor; T_a and T_1 are respectively the fundamental period of vibration of the panel and of the building in the relevant direction; z is the height of the non-structural element above the level of application of the seismic action and H is the building height.
However, the strength values (F_a) determined analytically are much lower than the corresponding values obtained from the analysis; therefore, the values obtained from the analysis were directly used. S275 steel and 8.8 grade bolts were adopted.

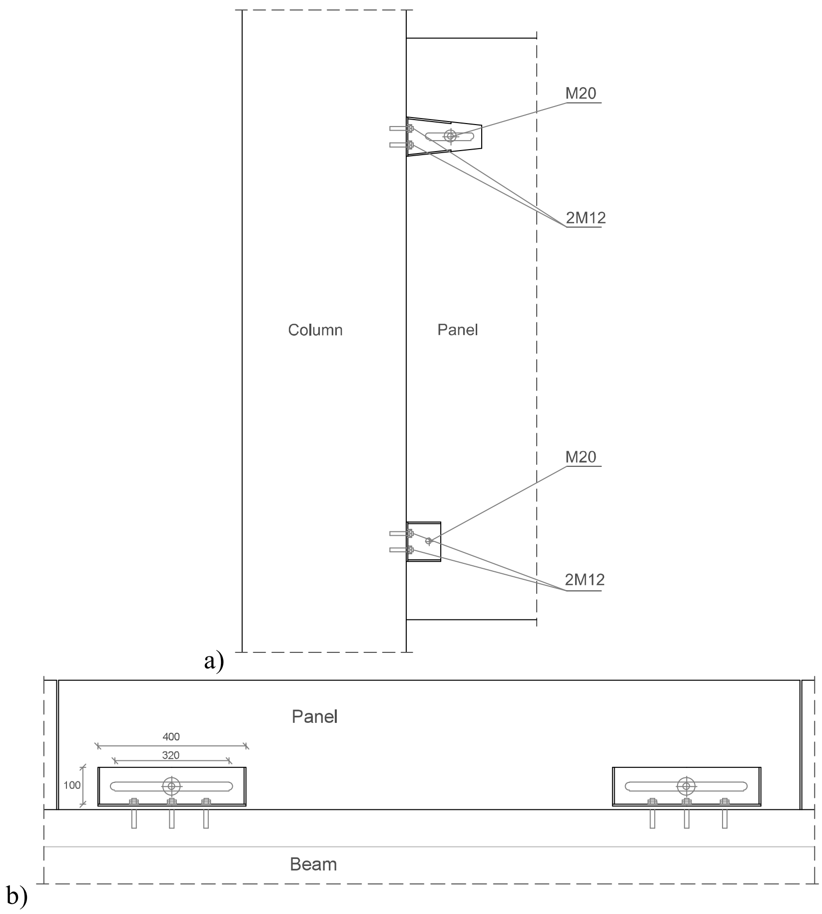

Figure 3: Details of the new connections for the (a) horizontal and (b) vertical panels.

3.1 Nonlinear modelling and analysis

The refined 3D model was created in OpenSees [22] according to the assumptions and approaches adopted in [21]. The new connections have been placed in the model by means of "Zero-Length" elements in parallel to other "Zero-Length" elements resembling the behaviour of the existing connections. The steel bracket at the bottom connections of the horizontal panels have been modeled with an "elasto-plastic" hysteresis material with stiffness (k) equal to 25'000 kN/m and yield strength (F_y) equal to 36.6 kN while those at the top have been modeled with "elasticPPgap" hysteresis material (gap=0.064 m, k=36'600 kN/m, F_y=36.60 kN), in the panel plane, and "elastoplastic" hysteresis material (k=22'000 kN/m, F_y=48.56 kN), in the out-of-plane of the panel. Similarly, the steel brackets connecting the vertical panels to the beams have been modeled with "elasticPPgap'" hysteresis material in the panel plane (gap=0.16 m, k=25'000 kN/m, F_y=42.74 kN) and "elastoplastic" hysteresis material in the out-of-plane of the panel (k=22'000kN/m, F_y=48.33 kN).

As regards the seismic input, two-hundred three-dimensions accelerograms were selected and grouped into ten increasing seismic intensity levels characterized by return periods equal to 10, 50, 100, 250, 500, 1000, 2500, 5000, 10000, 100000 years. Non-linear time-history analyses were performed by adopting mass and initial stiffness-proportional Rayleigh damping model. In addition, during the analyses, a check of the state of the plastic hinges inserted in the model was performed to assess failure of connections or elements.

4 RESULTS AND DISCUSSIONS

The results of the analyses are presented herein in terms of demand-on-capacity ratio (D/C) for two performance levels: Usability Preventing Damage (UPD) and Global Collapse (GC). The first refers to cladding panels while GC refers to roof elements and columns.

Figure 4 shows the beneficial effect of the retrofit intervention on the seismic response of the cladding panels. For the horizontal panels, the UPD is reached at intensities 2 and 5 for the as-is structure and the retrofitted structure, respectively; while referring to the vertical panels, UPD moves from intensity 3 to intensity 4. For the GC of the roof elements, the intervention does not alter the roof element falling from the support, while, considering the columns, we observe a slight reduction in the demand on the retrofitted structure compared to as-is conditions. This aspect might be related to the lower global acceleration demand due the higher fundamental period of vibration of the building with cladding panels (see for instance the mean acceleration spectrum of the ground motions of intensity 5 in Figure 5).

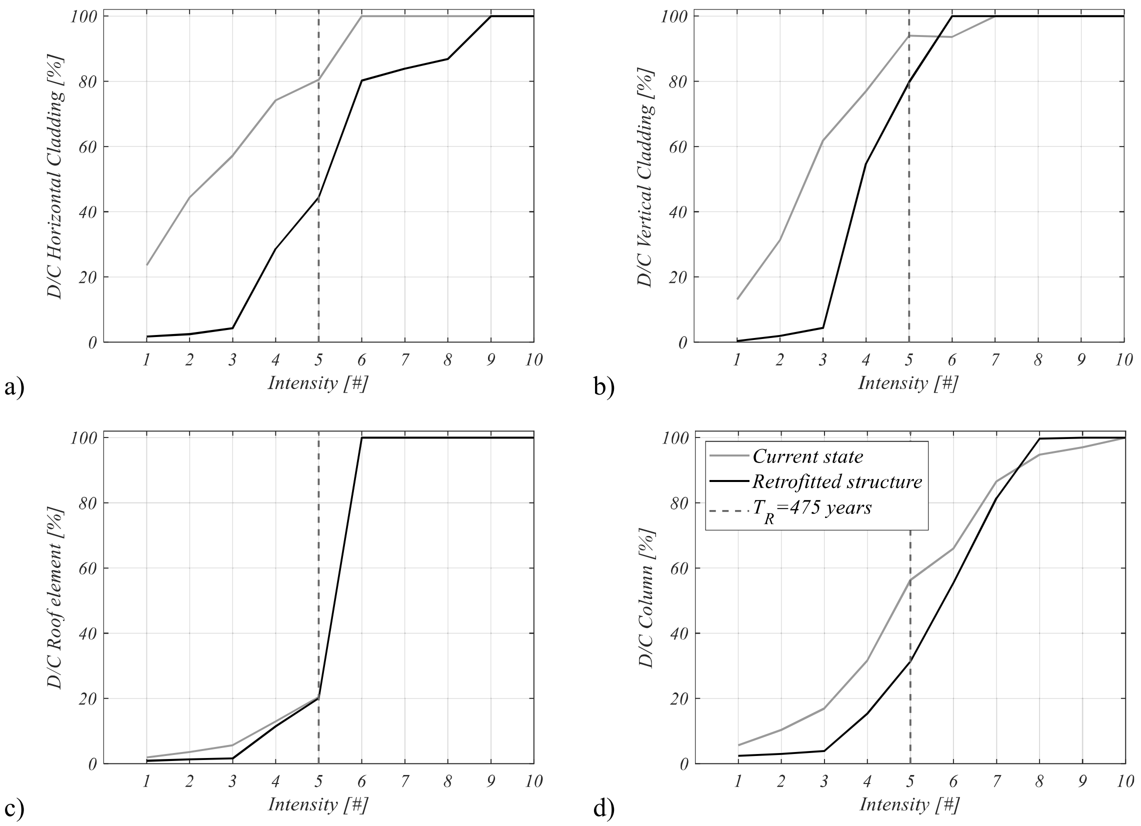

Figure 4: Demand-capacity (D/C) ratio for (a) horizontal and (b) vertical cladding panels, (c) roof elements and (d) columns. The dashed vertical line represents the seismic intensity for the life safety limit state.

Figure 6 shows the envelope for UPD of the cladding panels and GC for the columns and roof elements. The purpose of the retrofit intervention was related to the seismic risk reduction of the cladding system, and it successfully contributed to reducing it, although the effectiveness of the new connection of the vertically spanning panels was not optimal. This aspect needs to be further investigated. Regarding the GC, the envelope curve highlights a slight reduction of the D/C ratio.

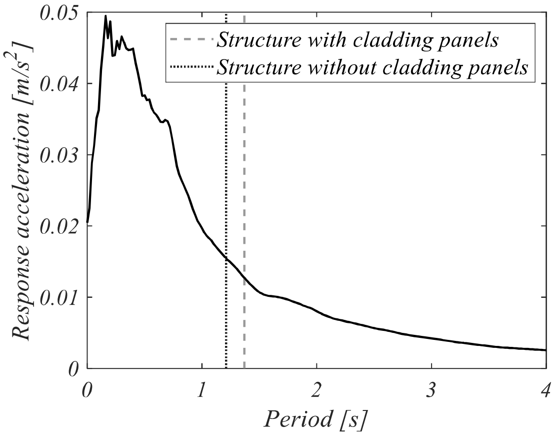

Figure 5: Mean elastic response spectrum (5% damping) for the ground motions of intensity 5. The vertical lines mark the fundamental period of the structure with and without cladding panels.

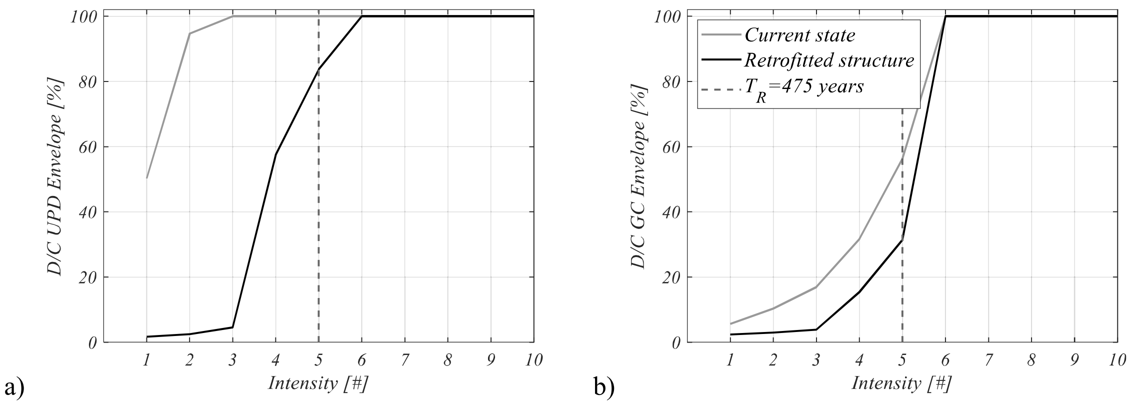

Figure 6: Demand-capacity (D/C) ratio envelope for (a) UPD (vertical and horizontal cladding panels failure) and (b) GC (failure of roof elements and columns). The dashed vertical line represents the seismic intensity for the life safety limit state.

5 CONCLUSIONS

This paper investigated the change in seismic risk of a typical precast industrial building following a local retrofit intervention of the connections between the cladding panels and the main structure. The investigated interventions considered the placing of new steel backets, in parallel with the existing connections. The new connecting elements have been designed in accordance with the Italian building code thorough a response spectrum analysis considering as target the life safety limit state, indeed the failure of the panel may cause injuries.

Multi-stripe analyses were carried out to compare the performance of the retrofitted structure with the as-is conditions. The results have shown that the new cladding panel connections led as expected to a delay of the panel connection failure although not so effective for the vertically spanning cladding panels. On the other end, considering the global collapse related to the falling of roof elements no evident benefit was observed, while for the global collapse related to column's failure, a slight reduction of the demand was recorded probably due to the lower acceleration demand associated with the higher fundamental period of the building with cladding panels compared with the building with failed cladding panels.

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