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ASSESSMENT AND RETROFITTING OF A RC BUILDING THROUGH A MULTI-HAZARD APPROACH: SEISMIC RESISTANCE AND ROBUSTNESS

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Abstract

Most of the existing buildings in seismic prone regions have been built before the publication of modern design provisions against seismic events and progressive collapse. Nonetheless, some studies have highlighted the possible interaction between earthquake resistance and structural robustness, the latter being of interest to either individual extreme hazards (e.g., blast, impact, fire) or interacting hazards (e.g., landslides produced by seismic events). While retrofit strategies to improve the seismic performance of reinforced concrete (RC) structures have been widely investigated since many years, the topic of mitigation strategies against progressive collapse received very little attention. Progressive collapse can be described as a special type of structural collapse that involves several components of the structure as consequence of an initial localised damage. The present study aims at investigating whether and how much seismic retrofitting may improve not only the earthquake resistance but also robustness. A four-storey, five-bay, RC frame building designed according to Eurocode 2 is considered as a case study. The frame was assessed by evaluating: 1) the capacity of the structure to redistribute loads after a local damaging event; 2) the seismic capacity of the structure. Non-linear static analyses, i.e., PushDown and PushOver analyses, were carried out in OpenSees to evaluate the robustness and seismic resistance of the structure, respectively. The progressive collapse capacity was evaluated under two relevant column-removal scenarios, i.e., the sudden loss of an internal and an external column, while the seismic resistance was assessed under two load distributions, i.e., proportional to the first vibration mode and to the inertia masses. Subsequently, the impact of retrofitting with carbon fibre-reinforced polymers on both structural robustness and seismic resistance was evaluated. The use of the retrofit measure allowed, on the one hand, the removal of all the shear failures due to horizontal seismic actions and, on the other hand, to increase the robustness of the structure.

Keywords: Reinforced concrete buildings, structural robustness, retrofit operation, progressive collapse, non-linear static analysis.

1 INTRODUCTION

Some iconic cases, such as the collapse of the Ronan Point Building (London, 1968) [1], the Murrah Federal Building (Oklahoma City, 1995) [2], and the World Trade Centre (New York, 2001) [3] highlighted the high consequences of progressive collapse, in terms of loss of lives and properties, significantly increasing the interest of the research community in this topic [4]. Since the 1940s, many studies focused on this research area, widely investigating various aspects of the problem by performing components [*e.g.,* [5], [6], [7]] and large-scale experimental tests [*e.g.*, [8], [9], [10], [11]], numerical modelling and simulations [*e.g.*, [12], [13], [14], [15], [16], [17]] and investigating several aspects of the design against progressive collapse [*e.g.,* [18], [19]]. These studies allowed to build up an increasing understanding of the structural response in progressive collapse scenarios, along with the definition of possible design strategies. However, design guidelines and codes [[20], [21], [22]] have been introduced only in recent years, and most of the existing buildings worldwide do not incorporate design provisions to achieve structural robustness.

Besides, existing reinforced concrete (RC) structures are often vulnerable to seismic actions, as demonstrated by many events worldwide [*e.g.,* [23]]. However, in the last few decades, there has been a significant effort from the research community in order to address these issues, and many seismic retrofit strategies are currently available and implemented in practice [*e.g.*, [24], [25], [26], [27], [28], [29]].

In contrast, a very limited number of research studies focused on the development and investigation of retrofit strategies to avoid progressive collapse for both RC and steel structures. Among the few studies addressing this problem, Vieira *et al.* [7] experimentally investigated the effectiveness of the Textile-Reinforced Mortar and Near-Surface-Mounted reinforcement techniques when applied for the strengthening of existing RC frames against disproportionate collapse, showing an increase of the ductility by a factor of 1.95. Shayanfar *et al.* [30] numerically investigated the use of several combinations of additional rebars and Carbon Fiber Reinforced Polymer (CFRP) layers for the retrofitting of 2-storey RC frame showing the improved catenary effects and the reduction of vertical displacements. Jinkoo *et al.* [31] investigated the effect of prestressing tendons on the progressive collapse performance of a RC structure by performing non-linear static and dynamic analyses investigating a 6- and a 20-story RC structure subjected to a sudden column loss scenario.

Although the above-mentioned studies (and a few more) investigate retrofit strategies for progressive collapse resistance, the knowledge level in this field is still very limited and there is a significant need for additional studies in this direction. Moreover, the vulnerability of these structures against multiple hazards offers the opportunity for integrated retrofit strategies that otherwise would often not be economically sustainable if directed toward the improvement of the structural response against a single hazard.

Within this context and given the high vulnerability of RC buildings towards the two considered hazards (*i.e.,* seismic actions and progressive collapse scenarios), the present study investigates the influence of the seismic retrofitting based on the use of CFRP on the structural robustness of a case study RC structure.

2 CASE-STUDY STRUCTURE AND FINITE ELEMENT MODELLING

A five-storey, six-bay by four-bay RC building, designed according to Eurocode 2 [32] and only to gravity loads, and previously investigated by the authors [[33], [34], [35]] was considered for case study purposes (see Figure 1). The building has a constant inter-storey height of 3 m and span lengths along the *x*- and *y*-directions equal to 5 m. Columns have a 400 × 400 mm^2 square section, whereas beams are 300 × 500 mm^2 at each floor. Concrete class C20/25 and steel reinforcement bars B450C were employed in the design. Uniform longitudinal reinforcement consisting of and 6ϕ18 and 8ϕ18 were used for columns and beams, respectively. The same transverse steel reinforcement of ϕ18 stirrups with 200 mm spacing and a concrete cover of 40 mm were used for both beams and columns.

A two-dimensional Finite Element (FE) model of the external frame in *x*-direction was developed in OpenSees [36]. A spread plasticity approach with displacement-based fibre formulation was used, and each cross-section was discretised in 120 fibres: one hundred fibres relating to the confined concrete (*i.e.*, the concrete core) and twenty for the concrete cover. The stress-strain concrete behaviour was simulated through the uniaxial Kent-Scott-Park concrete model [37] (*i.e.*, 'Concrete01' in OpenSees), while a uniaxial bilinear model with kinematic hardening set to 0.01 was adopted to simulate the steel behaviour (*i.e.*, 'Steel01' in OpenSees). The characteristic cylinder compressive strength of concrete, f_{ck}, was set to 20 MPa. The characteristic yield strength, f_{yk}, and Young's modulus of the reinforcing steel, E_0, were set equal to 450 MPa and 200 GPa, respectively. The loads were applied as concentrated loads on the beams following a discretisation of the structure in which each beam element was subdivided into 5 parts. Masses were concentrated at beam-column intersections while beam-column joints were modelled as rigid. Geometric non-linearities in the form of both large displacements/rotations and P-Delta effects were considered by means of a total corotational transformation.

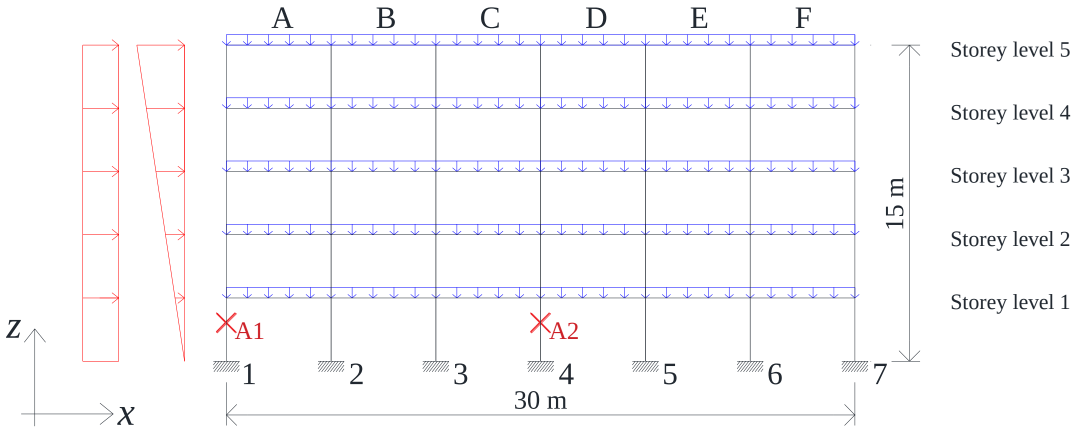

Figure 1: Reference frame model

3 PROGRESSIVE COLLAPSE ANALYSIS METHODS

The present Section investigates three different strategies for progressive collapse simulations. As shown in Figure 1, two different column removal scenarios were considered, *i.e.*, scenario A1 and A2. Loads were applied according to the accidental load combination, as per UFC [21] guidelines:

$$Q_{bd} = 1.2\ DL + 0.5LL \tag{1}$$

where DL and LL indicate the dead and live loads, respectively equal to 3 kN/m^2 and 2 kN/m^2.

A FE model in Seismostruct [38] was previously developed by the authors [35] for the case study frame, allowing the comparison of the numerical results and increasing confidence in the modelling strategy. The comparison was initially carried out in terms of modal properties showing a good match, *i.e.*, the first vibration periods were 0.35 sec and 0.37 sec for the Seismostruct and OpenSees model, respectively, while the second and third vibration periods were coincident. A further comparison was carried out investigating the behaviour under the progressive collapse scenarios. In particular, the results of Incremental Dynamic Analysis (IDA) carried out in the previous study [35] were compared with the PushDown Analysis (PDA) carried out in this study. The Dynamic Amplification Factor (DAF) is used in the PDAs in order to simulate the dynamic effects. Figure 2 shows three different approaches used to carry out the PDAs, allowing the identification of the most appropriate one in terms of convergence and analysis' accuracy. The following approaches are evaluated:

- The *'Approach A'* is characterised by two Analyses (see Figure 2(a)). *'Analysis 1'* consists of a standard load control static analysis of the 'undamaged' structure, allowing the definition of the reaction force (R) of the column where the column loss is successively simulated. During *'Analysis 2'* the column removal is simulated by two steps. *'Step 1'*, represented in Figure 2(a), allows simulating the presence of the column before the removal. The equivalent upward force F is applied to the frame, which entity corresponds to the vertical load previously detected on that column. In this Step, the gravity loads and the force F, are monotonically increased until the target value. In the following *'Step 2'*, represented in Figure 2(a), a downward force F (equal to R in terms of values and opposite in terms of direction) is applied to simulate the column removal. Moreover, the loads on the beams adjacent to the removal are amplified with the DAF. This second set of loads are thus gradually applied.
- The *'Approach B'* is characterised by a single step (see Figure 2(b)), consisting in a displacement control analysis with the application of design loads amplified by the DAF. This approach, differently from *'Approach A'* neglects the initial condition before the collapse but allows an easier implementation of the analysis. In addition, the displacement control strategy allows the convergence of the analysis for larger displacements hence better simulating the catenary effects.
- The *'Approach C'* is characterised by two Steps (see Figure 2(c)). *'Step 1'* represented in Figure 2(c) consists of a standard load control static analysis of the 'damaged' structure with the application of the design loads. In the following *'Step 2'* a displacement control analysis is performed with the design loads amplified by the DAF.

The present Section of the work investigates the intensity of the DAF parameter by the comparison of the results of the PDAs and IDA. For this reason, the DAF was considered unitary during these preliminary analyses.

The results of the PDAs were obtained in terms of α-θ curves, in which α represents the design load multiplier and θ the vertical drift of the control point. Specifically, α was obtained, according to Eq. 2, as the ratio between the sum of the reaction values, relative to the base constraints, and the sum of the loads applied to the structure. The vertical drift was obtained as the ratio between the vertical displacement of the control point, v, and the beam length, L, according to Eq. 3; the control point was coincident with the upper joint of the removed column.

$$\alpha = \frac{\sum_i R_i}{\sum_i R_i\left(Qbd\right)} \tag{2}$$

$$\theta = arctg\frac{v}{L} \tag{3}$$

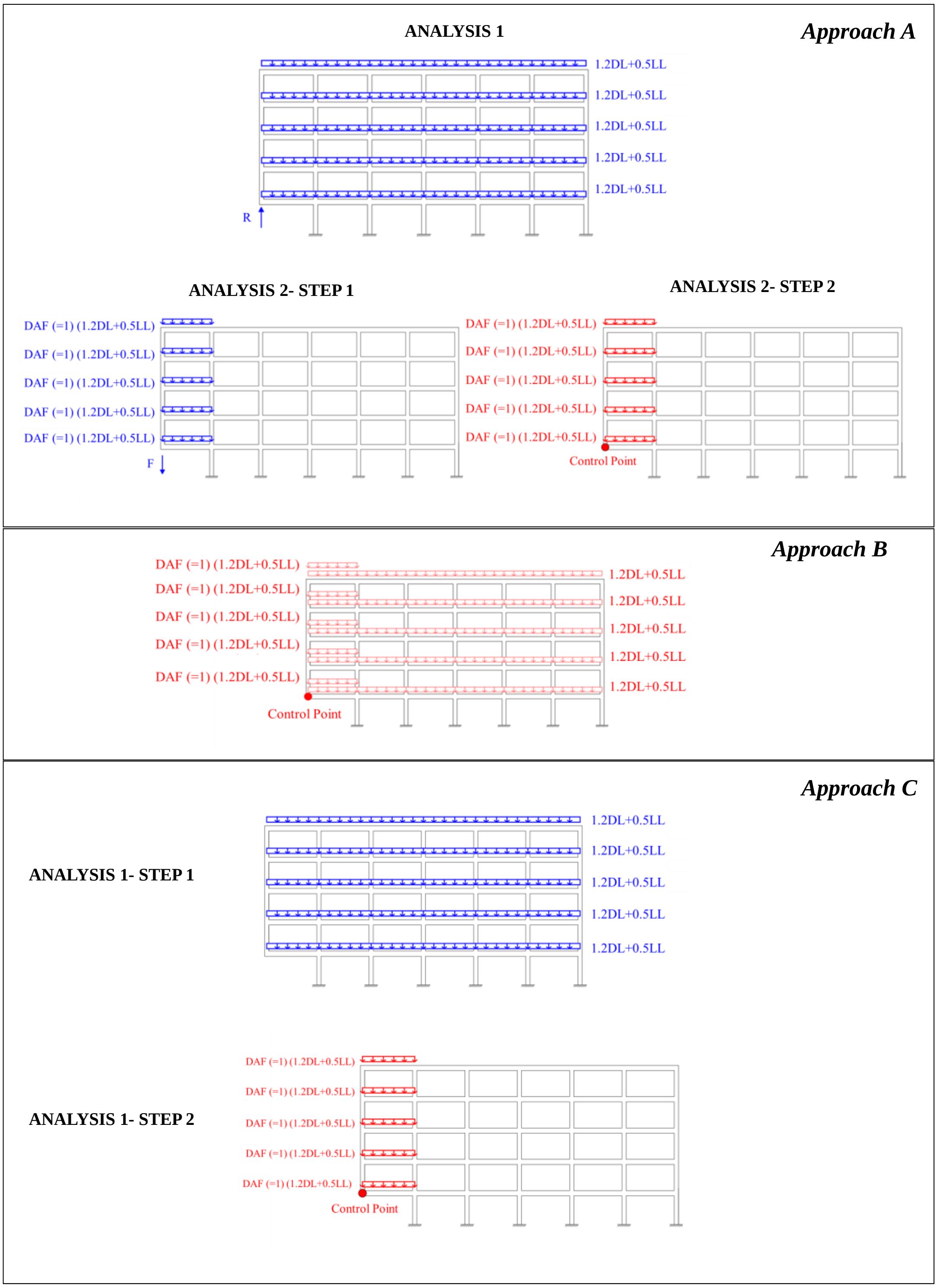

Figure 2: Approaches used for PushDown Analysis (PDA) (scenario A1).

The comparison between PDAs and IDA is shown in Figure 3. Through these results, the specific DAF (DAF_{sp}) was assessed as the ratio between the values of the load multipliers, α, obtained from both analyses, as follows:

$$DAF_{sp} = \frac{\alpha_{PDA}}{\alpha_{IDA}} \tag{4}$$

The results of the PDAs overestimated the design load multiplier, α, overestimating the capacity of the structure. This is related to the lack of consideration of the dynamic amplification during these analyses since the DAF was set equal to 1. However, this preliminary assumption allowed the evaluation of the DAF (DAF_{sp}) based on the results of the PDAs and IDA according to Eq. 4. The DAF_{sp} for the three Approaches and for the two removal Scenarios is shown in Figure 4. Figure 4 shows that Approaches A and C provide DAF values lower than the unit for high values of θ. The Approach B, on the other hand, led to a more realistic estimate of the DAF. Indeed, its value is always higher than the unit for both scenarios up to large vertical drift values and is aligned with DAF values obtained from other studies in the literature [40]. Approach B, with the corresponding value of the DAF, was chosen as the analysis method for the following part of the study. The DAF was set equal to 1.16, corresponding to a value of plastic rotation of 0.03 rads, as suggested by UFC (table 4-1); the yield rotation was calculated based on the recommendations of the Eurocode 8 [39].

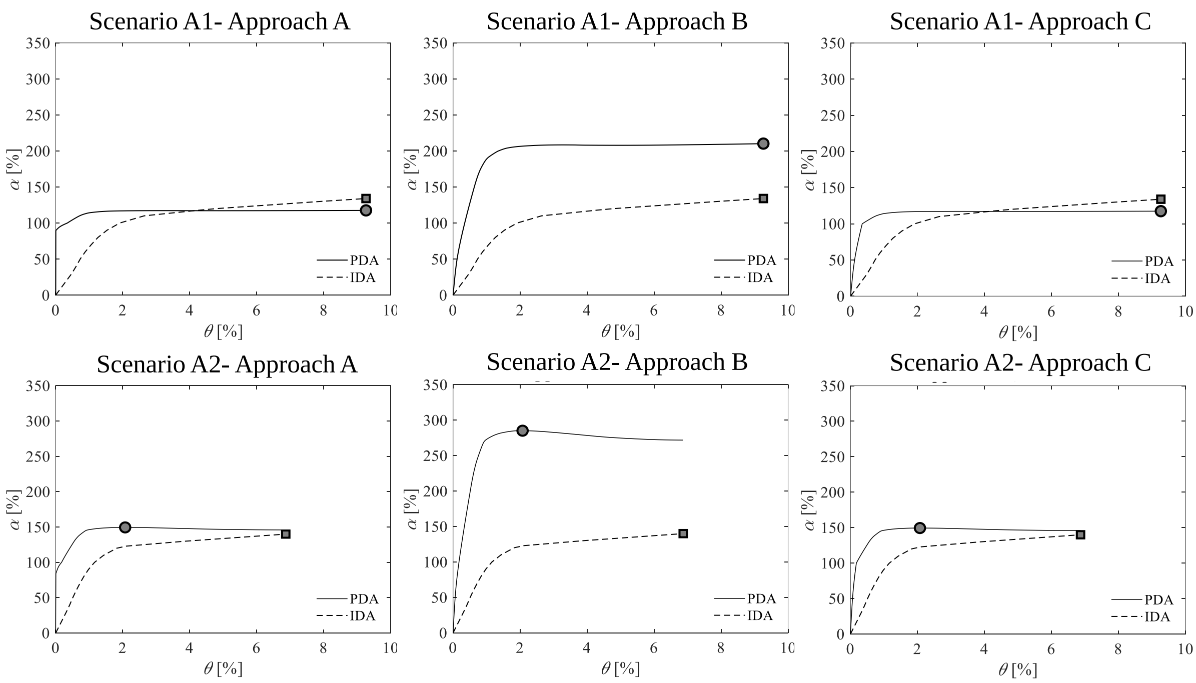

Figure 3: Comparison between PushDown analysis (PDA) and Incremental Dynamic Analysis (IDA).

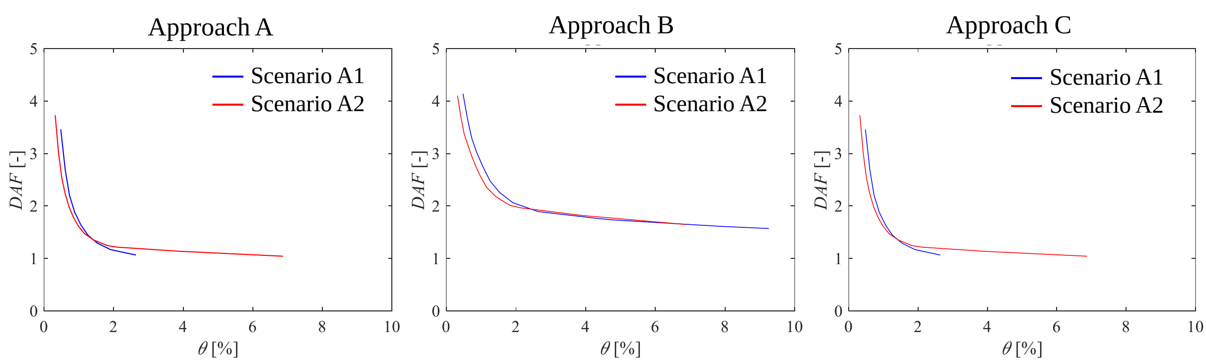

Figure 4: Dynamic Amplification Factor (DAF) assessment.

4 PROGRESSIVE COLLAPSE AND SEISMIC ANALYSIS

4.1 Progressive collapse resistance assessment

Progressive collapse analyses were carried out to assess structural robustness under the two column-removal scenarios previously discussed and shown in Figure 1. Unlike previous studies [[33], [34], [35]], in which a building perimeter frame was considered, the structural system is herein supposed to be an internal primary frame. This led to an increase of design loads applied on beams equal to 23 kN/m according to Eq. 1.

The α-θ curves for the two scenarios and obtained with the PDAs are shown in Figure 5. For both scenarios, Figure 5 shows that the structure lack of robustness with shear failures of the beams corresponding to α values of 33.05 and 47.61, respectively, for scenarios A1 and A2. The degrading model of Biskinis and Fardis [41] was used for shear checks.

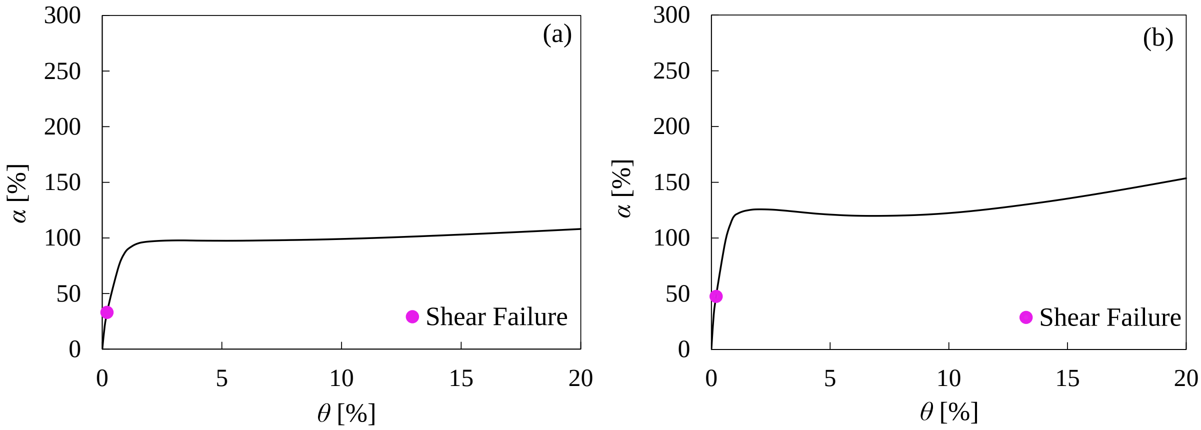

Figure 5: Pushdown analysis results: (a) scenario A1, (b) scenario A2.

4.1 Seismic safety assessment

In order to evaluate the seismic capacity of the structure, global and local checks were carried out through non-linear static analysis performed according to NTC 2018 [42]. The construction site was coincident with L'Aquila (Abruzzo, Italy), the reference life of the structure, V_R, was 50 years and soil and topographic category were *B* and T_1, respectively; the PGA was equal to 0,301g. Two different load distribution, such as the MODE and a MASS distribution, were considered. Local checks showed that almost all elements, vertical and horizontal, were subjected to brittle failures, as underlined by the demand/capacity ratios collected in Tables 2 and 3 (Bold values in the tables indicate demand/capacity ratios greater than 1). These checks were carried out step-by-step, and they were performed in correspondence of a single step, *i.e.*, that related to the identified performance point. The global checks compared the required inelastic displacement, $S_{d,inel}$ with the ultimate displacement of the structure, d_u. This comparison was performed in the ADRS plane, allowing the comparison between the demand spectrum and the capacity curve. The intersection between the demand spectrum with the extension of the elastic branch provided the performance point. The results are shown in Figure 6, with the relative demand/capacity ratios.

Floor level	Column Line													
	Mass distribution							Mode distribution						
	1	2	3	4	5	6	7	1	2	3	4	5	6	7
1	**2.03**	**2.07**	**2.14**	**2.19**	**2.25**	**2.32**	**2.04**	**1.59**	**1.81**	**1.93**	**2.03**	**2.12**	**2.21**	**1.89**
2	**1.60**	**2.20**	**2.14**	**2.09**	**2.04**	**1.98**	**0.90**	**1.80**	**2.25**	**2.24**	**2.20**	**2.16**	**2.12**	**0.98**
3	**1.15**	**1.86**	**1.77**	**1.68**	**1.58**	**1.48**	**1.02**	**1.46**	**2.16**	**2.07**	**1.98**	**1.88**	**1.78**	**1.17**
4	0.70	**1.39**	**1.32**	**1.25**	**1.19**	**1.13**	0.74	**1.04**	**1.76**	**1.67**	**1.58**	**1.50**	**1.42**	0.91
5	0.02	0.81	0.79	0.75	0.71	0.63	0.60	0.18	**1.12**	**1.05**	0.99	0.93	0.81	0.65

Table 1: Columns' demand/capacity ratios.

Floor level	Beam Span											
	Mass distribution						Mode distribution					
	A	B	C	D	E	F	A	B	C	D	E	F
1	**2.36**	**2.24**	**2.25**	**2.25**	**2.24**	**2.28**	**2.24**	**2.18**	**2.21**	**2.23**	**2.25**	**2.28**
2	**2.20**	**2.06**	**2.02**	**1.97**	**1.92**	**1.89**	**2.32**	**2.18**	**2.14**	**2.10**	**2.06**	**2.03**
3	**1.82**	**1.70**	**1.66**	**1.61**	**1.56**	**1.53**	**2.05**	**1.90**	**1.85**	**1.80**	**1.74**	**1.72**
4	**1.36**	**1.33**	**1.29**	**1.26**	**1.23**	**1.19**	**1.58**	**1.51**	**1.46**	**1.42**	**1.37**	**1.32**
5	**1.36**	**1.33**	**1.29**	**1.26**	**1.23**	**1.19**	**1.13**	**1.01**	**1.00**	0.98	0.98	0.89

Table 2: Beams' demand/capacity ratios.

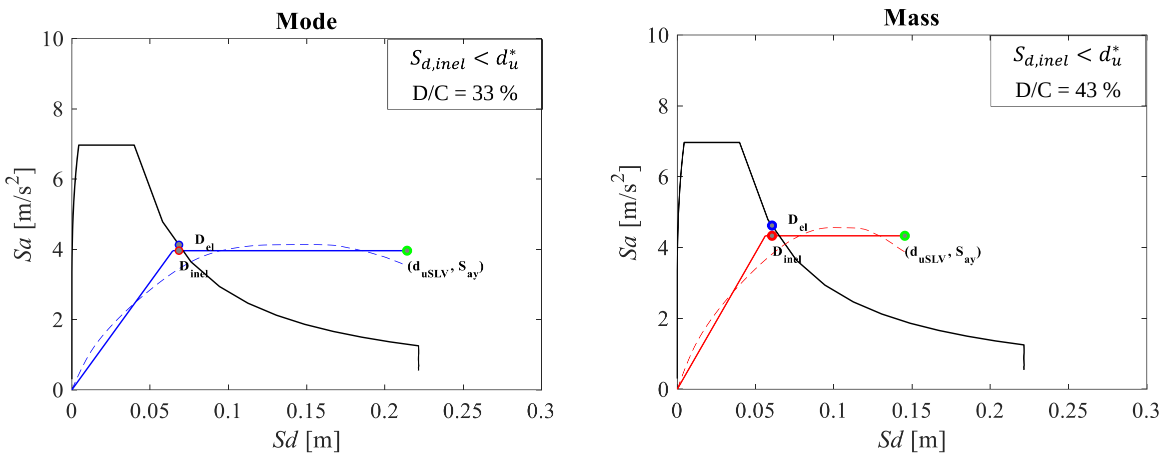

Figure 6: Global checks

5 INFLUENCE OF SEISMIC RETROFITTING ON STRUCTURAL ROBUSTNESS

The analysis results outlined the need to adopt a retrofit measure in order to prevent brittle failures occurring as a consequence of both considered hazards. In this study, the application of CFRP was investigated as it represents a widely used strategy to improve the seismic performances of RC frames. The impact of this retrofit strategy on the structural robustness was successively evaluated. The CFRP were applied differently on vertical and horizontal elements so as to eliminate brittle failures. A single bandage ply was used for the beams, while a different composition was considered for the columns. Five layers of CFRP plies, coinciding with the limit set by CNR-DT 200 R1/2013 [43], were used for the first floor level; three and two layers of plies were used for second and third floor level, respectively, while a single layer of ply was applied for fourth and fifth floor level. The analysis results showed that the application of CFRP allowed the removal of all the shear failures due to the seismic action, obtaining values of demand/capacity ratios lower than the unit.

Additionally, this retrofit measure had a beneficial effect, also improving the structural robustness of the frame under both column removal scenarios, as shown in figure 7. The first brittle failure, for both cases, occurred for a higher multiplier value than in the case of the existing structure. In scenario

A2, the retrofitted structure is able to avoid progressive collapse and at the same time was characterised by little damage, as low drift values were reached. A lower increase of structural performance was observed for scenario A1 where shear failures are observed for α values just above 100% *i.e.*, α = 101.64, yet achieving structural robustness under the considered design loads. In this case, however, the activation of the alternative load path required large displacements, *i.e.*, exceeded 10% of vertical drift, corresponding to a damage level significantly greater than in the previous scenario.

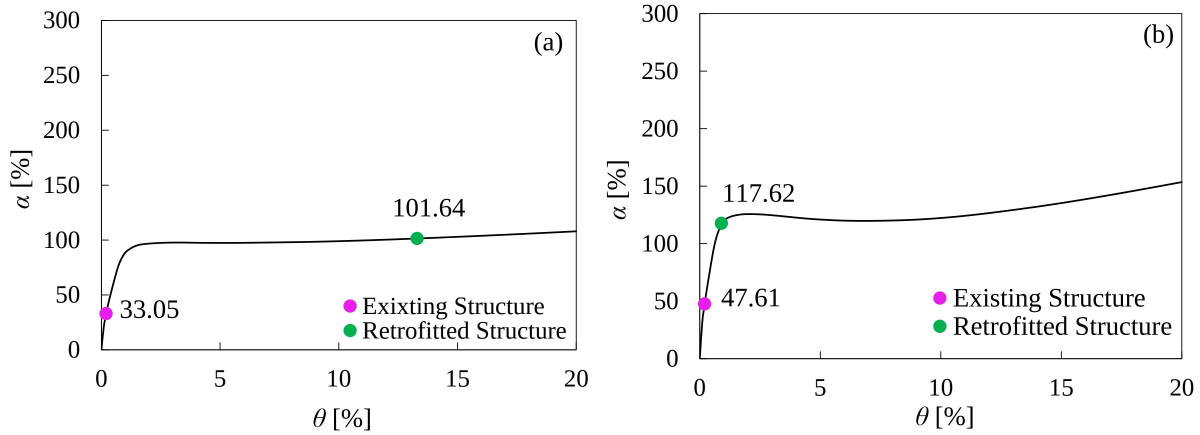

Figure 7: Impact of retrofit measure on structural robustness: (a) scenario A1, (b) scenario A2.

6 CONCLUSIONS

In this study, the structural capacity of RC framed structure through a multi-hazard approach was investigated. The impact of a retrofit measure, by the application of Carbon Fibre Reinforced Polymer (CFRP), on structural robustness was assessed. Non-linear static analyses were carried out to evaluate the structure's seismic capacity and its resistance to progressive collapse. The results of the analyses showed the need for retrofitting in order to eliminate brittle failures and to increase the structural capacity. The study also provided some insights into the methodologies for progressive collapse analysis using non-linear static procedures. Three Approaches (*i.e.*, A, B and C) were investigated and compared with the results of non-linear dynamic analysis previously performed on the same case study structure. The following conclusions can be drawn: *(i)* among the alternative options for progressive collapse resistance assessment, a more realistic estimate of the dynamic amplification factor was obtained with Approach B; *(ii)* the seismic safety and structural robustness are limited by premature brittle failures and *(iii)* the use of CFRP as a retrofit measure was efficient for the case study structure, leading to the removal of all the shear failures due to the seismic action. On the other hand, the introduction of this retrofit measure allows obtaining a positive result also in terms of structural robustness.

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