Role of floor diaphragms on the seismic response of reinforced concrete frames

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Abstract: In existing Reinforced Concrete (RC) framed buildings, floor structural components (i.e. RC topping and joists) may play a crucial role in the seismic performance of the structure. The interaction between floor diaphragms and seismic-resistant frames can lead to different effects, depending on the relative stiffness and resistance of the elements belonging to the structures and on the adopted construction details. In this work, these aspects are deepened with reference to the institute "A. De Gasperi – R. Battaglia", located in Norcia, Italy, chosen as case study. The seismic response of the building is investigated through pushover analyses by adopting a multi-layered shell element approach, where the mechanical nonlinearity is evaluated by using the PARC_CL 2.1 crack model, implemented as user subroutine in Abaqus *FE* package. The obtained results highlight that the modelling of the diaphragm increases the flexural resistance of the beams, so determining an increase of the seismic bearing capacity for frames characterized by ductile failure modes. The modelling of diaphragms may also alter beam-column strength hierarchy and stresses' magnitude in beam-to-column joints, leading to anticipated brittle failures, that cannot be detected through the modelling of the bare frame.

Keywords: existing structures, diaphragm, pushover, finite element analyses, seismic actions

1. Introduction

Most of existing Reinforced Concrete (RC) framed buildings across Europe were designed to sustain gravitational loads only, without the adoption of construction details able to cope seismic actions. However, an adequate seismic detailing is fundamental to avoid brittle failure modes, such as the achievement of members' shear capacity and/or beam-column joint failures (CEN-EC8, 2004). In Italy, a larger part of existing RC buildings is characterized by the presence of primary strong frames arranged perpendicularly to the direction of RC joists used to realize the diaphragm. Frames parallel to the direction of joists are usually much more deformable and are often not equipped with beams connecting the columns, except for the peripheral sides. In these cases, the interaction between the elements forming floor diaphragms (i.e. topping and joists) and seismic-resistant frames may become significant, leading to different effects depending on the relative stiffness and resistance of the elements forming the bearing structure, and on the available construction details. In this paper, the interaction between the RC frames and the diaphragms – realised with RC joists, interposed hollow clay blocks and RC topping slab – is investigated to evaluate the effect of the diaphragm modelling on the seismic response of RC frames.

A literature review on the influence exerted by RC slab floors on the response of RC beam-to-column joints was recently carried out by Streppone (2022). From the experimental point of view, other studies investigated the role of RC diaphragms on the monotonic and cyclic/fatigue response of RC frames (Ahmed & Gunasekaran, 2014; Montuori et al., 2021; Ning et al., 2014). It was observed that the presence of RC floor slabs significantly enhanced the resistance of the whole system under cyclic/seismic loading compared to the counterpart without floor slabs, and the effect on beam-column joints was also investigated. In the context of numerical simulations, several modelling techniques are nowadays available to simulate RC frames subjected to seismic actions by adopting static and dynamic solutions (O'Reilly & Sullivan, 2019). Santarsiero & Masi (2020) investigated the role of RC floors by adopting a high-fidelity numerical approach with brick elements and discrete rebars. Similar outcomes were found by Masoudi & Khajevand (2020), who adopted a multi-layered shell element model in Opensees. The results available in the literature highlighted that detailed finite element models allow to realistically represent the interaction between floor slabs and frame elements, by also detecting damage evolution during the structural response under seismic action. In this work, the influence of floor diaphragms on the seismic capacity of a RC frame is investigated by performing pushover analyses. For this purpose, the institute "A. De Gasperi – R. Battaglia" located in Norcia (Lima et al., 2018) is chosen as case study.

The structure is modelled by adopting the PARC_CL 2.1 crack model, implemented in the Finite Element (FE) code Abaqus as user subroutine (Belletti et al., 2017). The study represents an extension of a previous work (Belletti et al., 2019), and aims to show the relevant role played by the structural components of the floor on deformation mechanisms and structural performances of the frames forming the building. For this reason, two different cases are considered: in the first one (corresponding to the real geometry of the case study building, with RC joists perpendicular to the main beams of the frame), the attention is simply focused on the strength and stiffening effect exerted by the RC topping. In the second case, RC joist direction is assumed parallel to the main beams, so to include their structural contribution explicitly. The obtained results highlighted that the presence of RC topping and joists of the floor and their collaboration with the frame enhance the global bearing capacity of the structure, in terms of maximum sustainable base shear. According to the suggestions of relevant Standards, it can be observed that RC topping increases the strength of the beams, since it acts as an additional collaboration flange at beam extrados.

Depending on the geometry and structural details of frame elements, this stiffening effect may alter the strength hierarchy of the system with respect to the case of the bare frame, leading to a change in the damage sequence in the structure, with a possible occurrence of soft floor mechanisms. For this reason, in case of existing structures, the common assumption of modelling only the bare frame structure, without explicitly including floor systems (and simply considering their effects as rigid constraint) may be not always on the safe side.

2. Description of the case study

2.1. Building geometry

The "A. De Gasperi – R. Battaglia" school building is characterized by a RC frame system, depicted in Figure 1. The diaphragms at each floor are formed by RC joists (with interposed hollow clay blocks) oriented along the X direction, and an upper collaborating 40-mm thick RC topping. However, in the performed numerical analyses, another floor arrangement, with RC joists aligned along Y direction, is also explored, to evaluate their possible influence on the structural response of the frame (refer to Figure 1, "modified plan layout", in green). In both the considered cases, RC joists are assumed to be 120-mm width and are reinforced with two 12-mm rebars. RC columns have a variable cross section along the height, with major local axis oriented along the Y direction, perpendicular to the original RC joists arrangement. The main beams of the frame, oriented along the Y direction, have a cross section equal 300 mm x 660 mm. In the original configuration of the building, transverse beams have a variable cross-section at different levels; most of the beams belonging to internal frame are 30-cm thick, while those belonging to the external frames are realized within the floor depth.

In this study, the dimensions of transverse beams are changed so to be compatible also with the modified orientation of floor joists and are kept the same (600 mm x 300 mm) in all the performed analyses. A sketch of the cross –sections of beams and columns, with the corresponding reinforcement layout at the first-floor level is reported in Figure 2. However, detailing varies at different floors. Beam-to-column joints are reinforced with ϕ6 mm stirrups with 8 mm spacing at the first floor, 10 mm at the second floor and12 mm at the fourth and fifth floors. For details refer to Lima et al. (2018). Inter-story height is 3.7 m at the first floor, 3.3 m for the other floors, and 2.5 m at the roof, as shown in Figure 1. In a previous study (Belletti et al., 2018) the non-linear response prediction of the building modelled by using beam elements and subjected to seismic action along X direction was analysed with and without the modelling of the diaphragm. The seismic response of the structure is investigated herein through PushOver (PO) analyses, by considering a lateral force distribution acting along Y direction and proportional to the seismic masses at each floor. The control point (CP) for the pushover analyses is shown in Figure 1d.

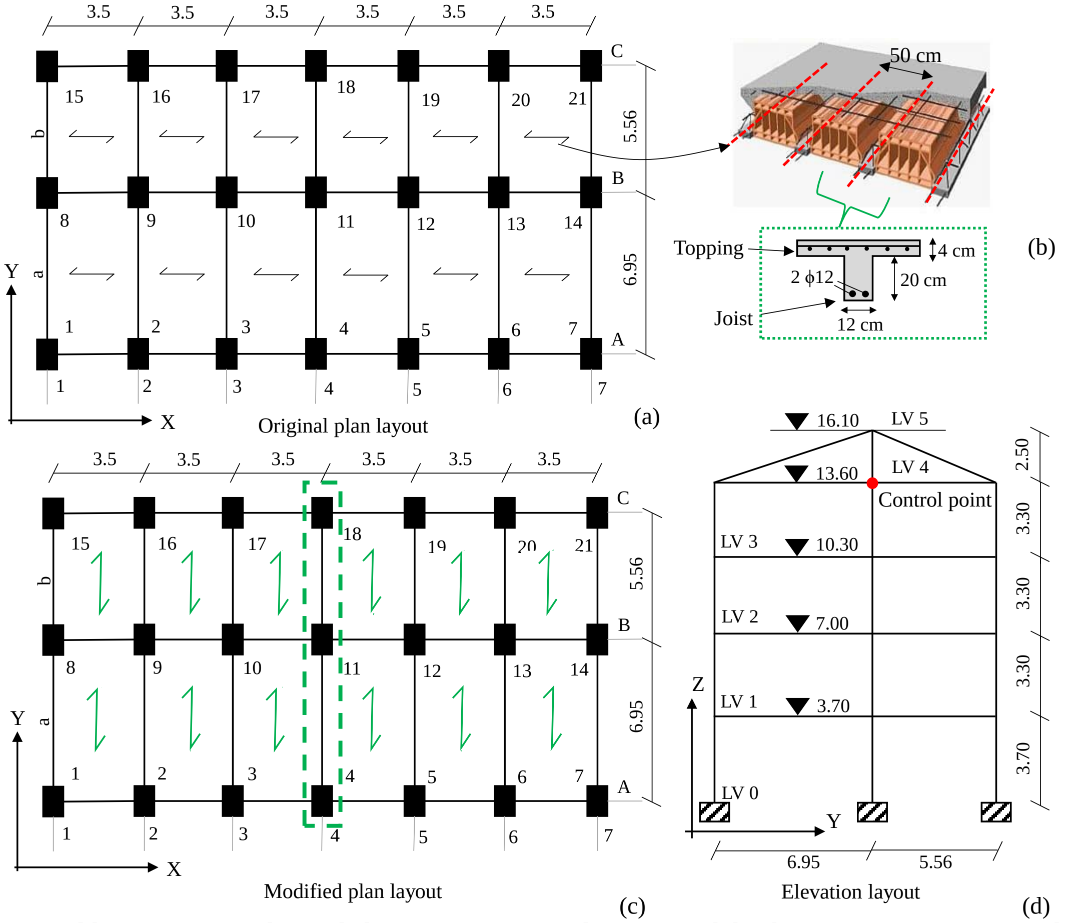


Figure 1 Building geometry: plan and elevation views: (a) plan view and diaphragm orientation assumed for Case C1 and Case C2, (b) composite floor adopted for the diaphragm, (c) plan view and diaphragm orientation assumed for Case C3, (d) section view. Dimensions in [m] and [cm].

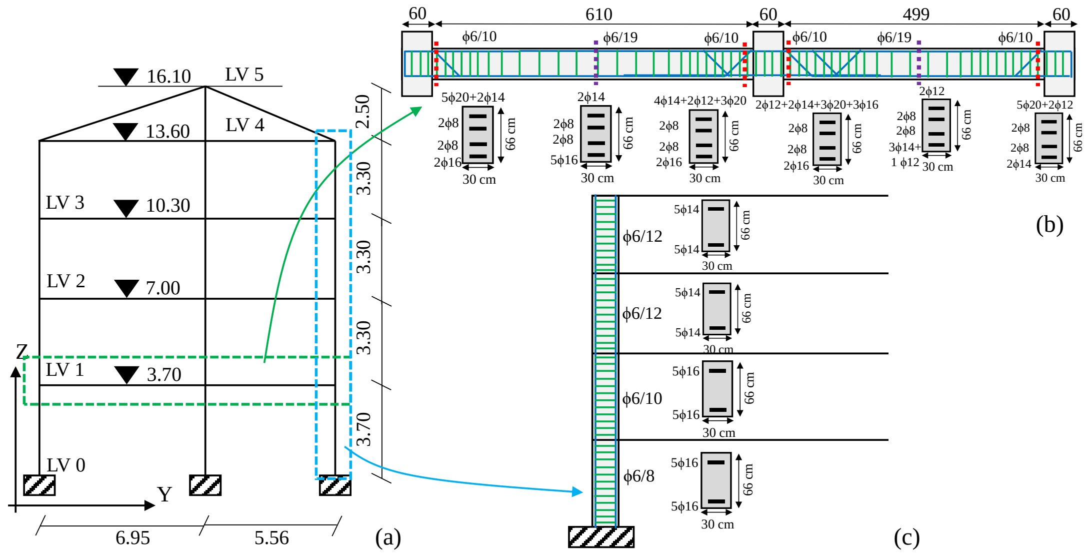


Figure 2 (a) Section view of the frame and reinforcement layout of the (b) beams and (c) of the column. Not to scale.

2.2. Mechanical properties and applied loads

The mechanical properties of concrete and steel derived from in-situ tests are reported in Table 1. The loads equivalent to the contemporary static combination by considering self-weights, permanent and variable floor loads are applied to beams. The corresponding seismic masses at each floor level are reported in Table 2.

Table 1 Mechanical properties.

Concrete			Steel			
f_c [MPa]	f_{ct} [MPa]	E_c [MPa]	f_y [MPa]	f_u [MPa]	E_s [MPa]	ε_u [%]
25.2	2.0	22000	375	450	200000	4

Table 2 Seismic masses at each floor level.

LV 1 [tons]	*LV 2* [tons]	*LV 3* [tons]	*LV 4* [tons]	*LV 5* [tons]
320	317	326	299	116

3. Numerical modelling

The nonlinear behaviour of beams, columns and floor elements is modelled through a multi-layered shell element approach, by adopting the PARC_CL 2.1 crack model (Belletti et al., 2017), implemented into Abaqus (2018) FE package as User MATerial (UMAT) subroutine. The model is based on a fixed crack approach, and assumes the reinforcement as smeared within the hosting concrete element. The model is developed for elements subjected to plane stresses: for this reason, the thickness of each element can be subdivided into layers, each one formed by concrete with or without steel reinforcement. The PARC_CL 2.1 crack model has been used to model punching failure (Belletti et al., 2019) and structural robustness assessment (Martinelli et al., 2022). It has been recently updated by including rheological phenomena (Vecchi et al., 2022). Three modelling cases are investigated concerning the above-described case study (Figure 3). The first one represents the selected bare frame case C1, (Figure 3a), which is representative of the whole system response - analysed in the previous work by Belletti et al. (2019) -, in virtue of the building regularity and loading distribution. RC beams and columns of the frame are modelled through S4 shell elements (Abaqus, 2018), and steel reinforcement is smeared in the host concrete elements though adequate reinforcement ratios. RC columns are fixed at base nodes. The other two considered cases allow to investigate the influence of floor components on the structural response of the frame. Case C2 corresponds to the real floor arrangement (depicted in black in the upper part of Figure 1a), with RC joists perpendicular to the main beams. For this reason, their presence is neglected in numerical analyses and only the RC topping is included in the model, by using non-linear S4 shell elements with smeared reinforcement (Figure 3b). Case C3 corresponds instead to the modified building configuration (depicted in green in the lower part of Figure 1c), with joists oriented along the Y direction of the building and parallel to the main beams. In this last case, both RC topping and joists are included in the model. Joist reinforcement is simulated by means of nonlinear truss elements, properly connected to the corresponding bottom nodes of shell elements modelling the web of RC joists (Figure 3c). The nomenclature adopted in the study is summarized in Table 3 for sake of clarity. Both in C2 and C3 cases, the part of transverse beams close to midspan are included in the analyses in a simplified way, by simulating their behaviour through linear elastic shell elements. This latter modelling choice allows to reduce the time and memory required for analyses without affecting the accuracy of the response that is dependent on plastic hinges formation at the beams' ends.

Table 3 Modelling cases analysed in the study.

Item	*C1*	*C2*	*C3*
RC topping slab	-	x	x
RC joist	-	-	x
Transverse RC beams	-	x	x

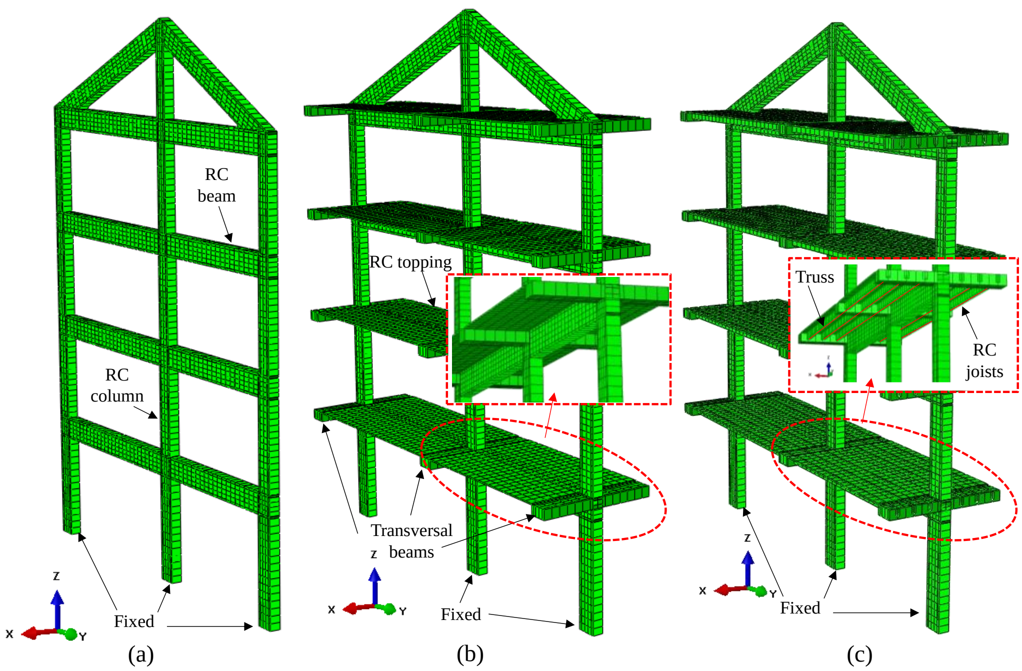


Figure 3 Modelling cases with solid shell views: (a) Case C1, (b) Case C2, and (c) Case C3.

4. Results and discussion

4.1. Global response

The results of PO analyses are plotted in Figure 4 in terms of total base shear vs horizontal displacement of the control point for the different modelling cases. It can be observed that the reference case C1 (without the explicit modelling of floor diaphragm) is characterized by lower stiffness and bearing capacity compared to the other cases. The highest stiffness and bearing capacity are associated to case C3, due to the presence of both RC joists and topping, while case C2 (with the only presence of RC topping) shows an intermediate behaviour. For sake of brevity, further discussions are only focused on the two limit cases C1 and C3. For these two cases, the sequence of ductile and brittle damages - detected during numerical analyses - is reported on the corresponding capacity curves (Figure 4). In the bare frame (case C1), a ductile behaviour is observed, with the formation of plastic hinges in the beams at all levels (except for level 4, which is stiffer due to the presence of the roof), followed by the yielding of longitudinal rebars at the base of the columns of the lower level. In more detail, the first plastic hinge forms at beam extrados (for negative moments) at LV1, followed by the appearance of another plastic hinge at beam extrados also at LV2, both in the right bay of the frame (near column 18, refer to Figure 1). It should be however observed that these hinges do not appear near the external column but are placed in a backward location, corresponding to the section where longitudinal rebars are bent-up to increase the shear resistance of the member (refer to Figure 2b). This latter observation could be relevant for simplified modelling of the frame by adopting beam elements and concentrated plasticity at beam's ends because the discrepancy between the real position of plastic hinges and the modelled one, could play a relevant role. For increasing lateral loads, the following damage sequence is detected: yielding of longitudinal rebars at beam intrados (for positive moments), first at level LV1 and then at level LV2; yielding of longitudinal rebars at beam extrados (for negative moments) at level LV3; yielding of the stirrups in beam-to-column joints at levels LV2 and LV1; yielding of longitudinal rebars at the base of the columns at level LV1. Finally, the ultimate strain is achieved in longitudinal rebars at beam extrados (in correspondence of plastic hinges location), both at levels LV1 and LV2 (refer to the failure mode contour in Figure 5a). As can be inferred from Figures 4-5, the presence of the RC diaphragm (topping and joists, case C3) significantly modifies the damage sequence and the failure mode of the frame, leading to a soft-storey mechanism involving the lower level of the building (refer to the failure mode contour in Figure 5b).

Due to the increased strength of the beam (which is related to the presence of the floor topping, acting as a collaborating flange at beam extrados), the first plastic hinges appear in this case at beam intrados (for positive moments) in the left bay of the frame, first at level LV1 and subsequently at level LV2, as depicted in Figure 4c (near column 4, refer to Figure 1). Also in this case, the position of plastic hinges is influenced by the setting of longitudinal rebars, which are bent-up to increase shear resistance. The formation of plastic hinges at beam extrados (for negative moments) at levels LV1 and LV2 takes place almost at the same time as rebar yielding at the base of the columns. Later, the yielding of the stirrups of beam-to column joints at levels LV1 and LV2 takes place, followed by the formation of a plastic hinge at beam intrados (for positive moment) at level LV3. Finally, a soft-story mechanism takes place at level LV1, after the failure in compression of concrete struts in the central beam-to-column joint (refer to Figure 5b).

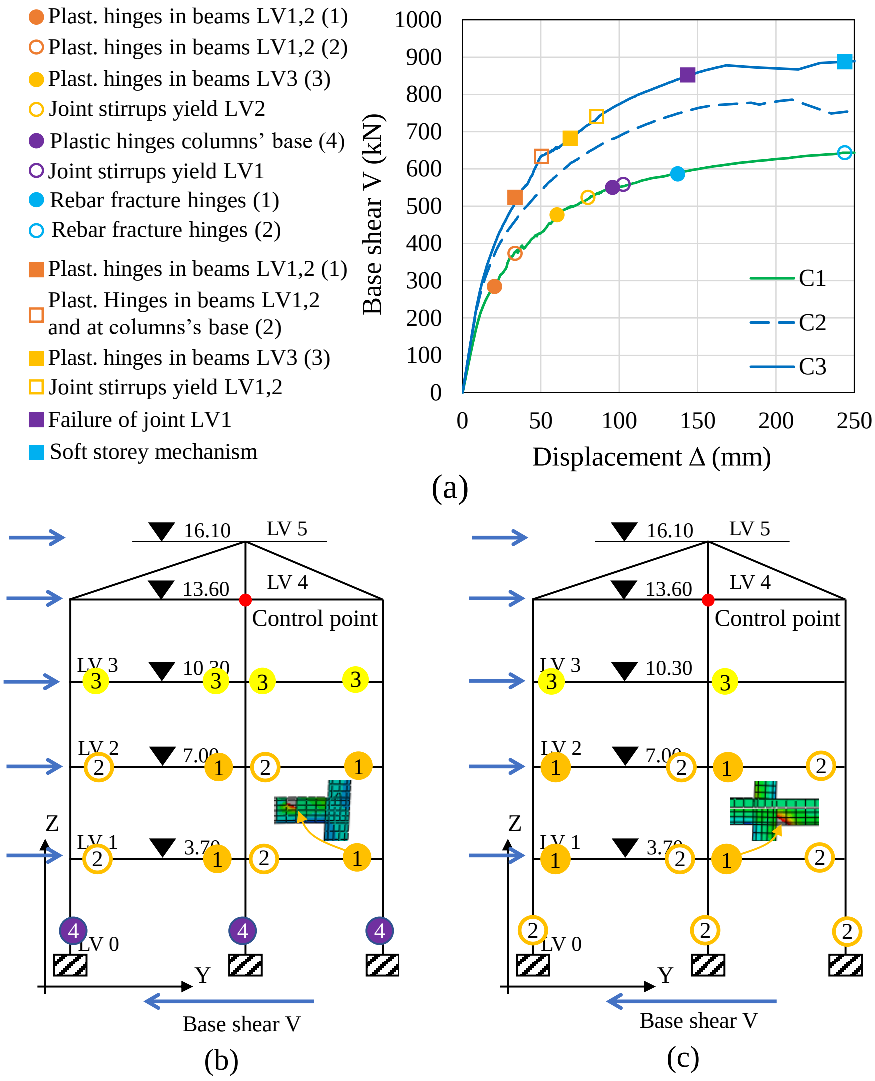
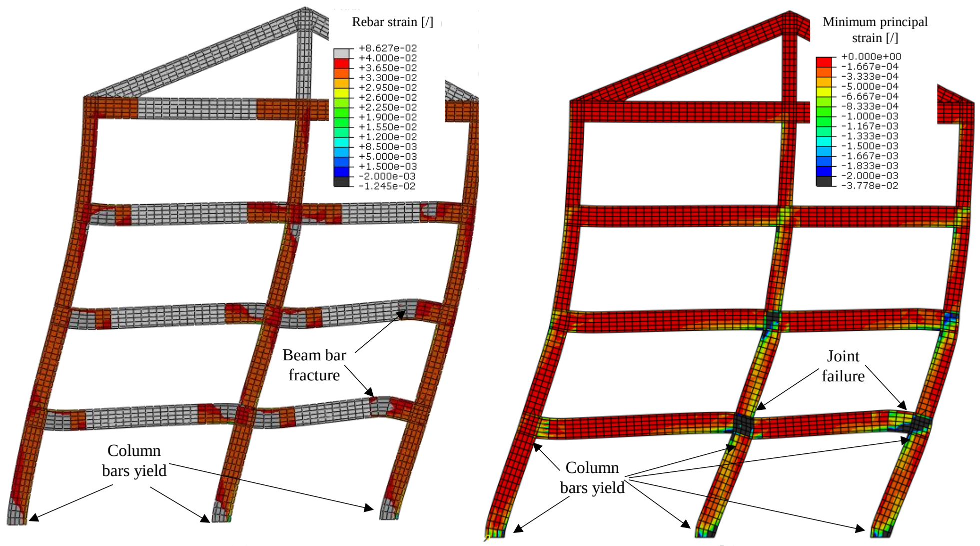


Figure 4 Pushover curves for seismic loading +Y applied to the different modelling cases and main events.

4.2. Influence of floor modelling on the behaviour of frame elements and of beam-to-column joint

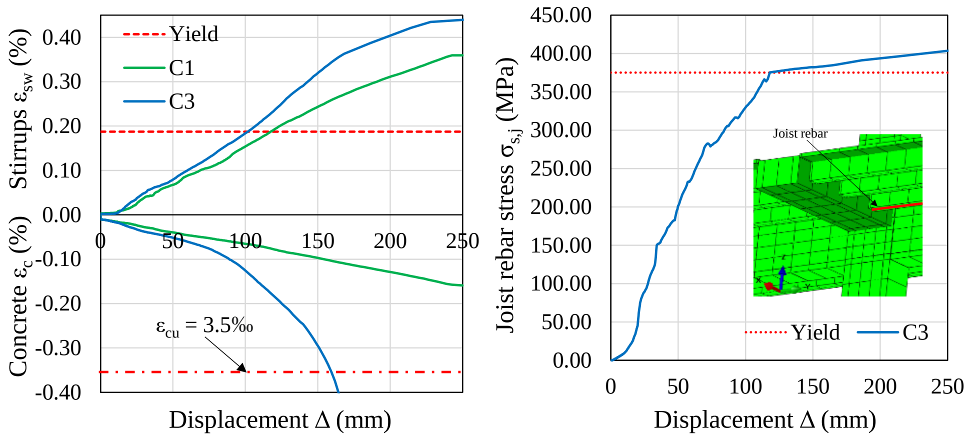
The response of beam-to-column joints is crucial to adequately redistribute the applied loads between resisting members during a seismic event and may affect the failure modes and strength hierarchy of the whole system. Within this context, Figure 6a reports both stirrup strains and concrete compression strains in correspondence of the central beam-to-column joint at level LV1 for the two limit cases C1 and C3, as a function of the control point displacement. The curves highlight that in both cases, stirrups in beam-to-column joint are yielded but do not reach the ultimate strain value.

On the contrary, when explicitly modelling floor elements (case C3), concrete strains are significantly higher with respect to the bare frame (case C1), and exceed the limit value of 3.5 ‰ - which is reasonable for unconfined members - for a displacement value of about 150 mm. As can be seen from Figure 4, this displacement approximately corresponds to the onset of the soft-storey mechanism at level LV1. The stress state in the longitudinal rebars of RC joists placed near the main beam at level LV1 is depicted in Figure 6b. Rebars are yielded, so confirming their contribution in increasing the lateral resisting behaviour of the RC frame, and in reducing the stress state in the beam. The attention is focused on the right bay of the frame.



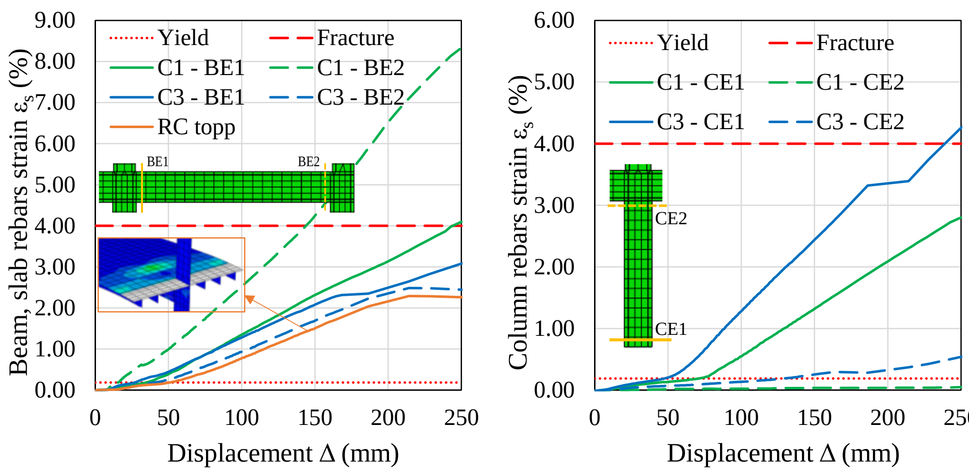
(a) (b)

Figure 5 Failure modes contours at ≈ 250 mm for: (a) C1, (b) C3 cases. Deformed shape x10.



(a) (b)

Figure 6 Results and contours at joint vs CP displacement: (a) stirrups and concrete strains in the central beam-to-column joint, (b) joist rebars stress.



(a) (b)

Figure 7 Strains in the longitudinal reinforcement vs. CP displacement for: (a) beam of the right bay of the frame; (b) central column, at level LV1.

The graph shows that, for case C1, rebar strains are much higher at section BE2, near the external column, where the ultimate strain value is also achieved for a displacement almost equal to 150 mm, according to Figure 5a. On the contrary, in case C3, rebars appear to be yielded but remain far from the ultimate strain value and their trend is very similar in correspondence of both beam ends. The strain distribution in the longitudinal reinforcement of the topping is also shown, to further prove its collaborating effect on the beam. Finally, Figure 7b reports the strain distribution in the longitudinal reinforcement of the central column at level LV1. The curves highlight the formation of a plastic hinge at the base of the column in both cases C1 and C3, but the presence of the floor slab determines an increase of the strain values, which causes the failure at the base of the column, associated the soft-floor mechanism highlighted in Figure 5b.

5. Conclusions

In this study, the role of floor elements (RC topping and joists) on the seismic response of frame structures is investigated by performing nonlinear finite element analyses. The "A. De Gasperi – R. Battaglia" school building located in Norcia is chosen as case study and an internal primary frame is extracted and modelled by using multi-layered shell elements. The response of the bare frame is compared to that obtained by including floor elements in the FE model. Two different floor arrangements are considered, with joists respectively parallel (original layout) and perpendicular to the X direction (modified layout). In the first case, only the RC topping is modelled, while in the second one, both RC topping and joists are considered. The main conclusions are reported as follows:

- Pushover analyses show that the resisting contribution of the RC diaphragm considerable enhances both bearing capacity and lateral stiffness of the resisting system. If both RC topping slab and joists (C3) are included in the FE model, the maximum capacity increases of about 40% compared to the reference case study (C1 case).
- Due to the presence of RC topping and joists, the strength hierarchy of frame elements may be modified, and the global failure mechanism may shift from ductile (Case 1) to brittle (Case 3). The topping acts as a collaborating flange working at beam extrados and this led to a change in the distribution of plastic hinges in the beam itself (from extrados to intrados). Moreover, while in the bare frame case the cross-section dimensions and reinforcement layouts of the elements correspond to a situation with "strong columns and weak beams" leading to a ductile global response, the modelling of RC joists and topping slab leads to the failure of some beam-to-column joint in compression at level LV1, associated with the formation of a soft-storey mechanism.
- The current assumption of modelling the bare frame in existing RC structures, by only considering the effect of floor diaphragms as rigid constraint, is not always on the safe side, and the interaction between the beams and the floor elements should be inserted, at least in a simplified way, in numerical analysis.
- The paper shows that the refined modelling by adopting multi-layered shell elements, allows to evaluate all the ductile and brittle phenomena occurring during loading without the need of a-priori hypotheses and a-posteriori controls of failure modes non considered in the model.

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