A PARAMETRIC STUDY ON RC EXISTING BUILDINGS TO COMPARE DIFFERENT ANALYSIS METHODS CONSIDERED IN THE EUROPEAN SEISMIC CODE (EC8-3)

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ABSTRACT :

In the paper a parametric study on Reinforced Concrete (RC) frames representative of structural types widely present in the European building stock is carried out. Buildings designed only for vertical loads are considered, detailed adopting the simulated design procedure as described in (Masi, 2003) and introduced in the European code for the assessment and retrofitting of existing buildings (CEN, 2004). Structural types having different number of storeys (3 cases: 2, 4 and 8 storeys), presence and position of masonry infills (3 cases: bare, infilled and pilotis frames) are considered. The seismic response is evaluated by means of the linear and non-linear static analysis methods provided in the seismic code. The different performances obtained adopting different seismic action levels (4 cases: very low to high seismic zone), behaviour factors (4 cases, only for linear analysis), knowledge levels and confidence factors (3 cases), concrete strength (3 cases: 10, 18 and 28 MPa), are computed. The results are compared to understand the coherence of the analysis methods provided in the code, that is if they lead to equivalent results in respect of the limit states verification taking also into account the different complexity and accuracy of the adopted method. The role of some parameters on the performances is emphasized and an outline of the future developments of the study aimed at upgrading the current version of the code is provided.

KEYWORDS: Seismic codes, Assessment, Existing buildings, Reinforced Concrete, Methods of analysis

1. INTRODUCTION

In Italy as well as in other European countries a large part of the residential building stock, particularly in terms of volume, has Reinforced Concrete (RC) structure. Moreover, many of these buildings have been designed only for vertical loads without anti-seismic design criteria. For this reason the assessment and retrofitting of these buildings is essential to realize an effective mitigation strategy of seismic risk.
Safety verifications of existing RC structures are conditioned by the uncertainties in the estimation of the in-situ material properties (concrete and steel), the knowledge of the structural system and reinforcing details. The European seismic code EC8-3 (CEN, 2004) defines three different Knowledge Levels (KL) depending on the amount and quality of the information available regarding geometry, details and materials. The achieved KL is fundamental in order to choose the admissible type of analysis and the appropriate Confidence Factor (CF) value. According to EC8-3 the seismic action effects may be evaluated using some methods, among which the Linear Dynamic Analysis (LDA) with “q-factor” approach, widely used in the professional practice, and the Non Linear Static Analysis (NLSA) whose adoption is rapidly increasing.
In this work a wide parametric analysis has been carried out and the results, in terms of Demand/Capacity ratios (D/C), obtained applying the LDA and NLSA methods, are reported and compared in order to evaluate their relative coherence. The analyses have been applied to some structural types, representative of typical RC existing buildings, defined by means of a simulated design procedure, set up in Masi (2003). The seismic response has been evaluated by adopting four levels of seismic intensity, three values of concrete strength and the three knowledge levels defined by EC8-3. Further, four different values of the behaviour factor (q = 1.5, 2, 2.5, 3), when the LDA with “q-factor” approach is performed, have been considered. The different performances have been analysed and compared performing safety verifications at the Limit State of Significant Damage. The results have been also examined to evaluate the coherence of the two analysis

methods under study keeping in mind their different complexity and accuracy.

2. BUILDING TYPES

Making reference to the Italian construction standards after the 1971 (Masi et al., 2004) some 3D structural types (Moment Resisting Frames), representative of RC frame buildings widely present in the Italian and European building stock, have been defined. The selected types have 2, 4 and 8 storeys, with an interstorey height equal to 3m, representative of low-, mid- and high-rise buildings, respectively. The buildings have rectangular plan shape with global dimensions 10×15m, and constant bay length equal to 5m in both directions. In the exterior frames the beams are 30×50cm (Rigid Beam, RB) while in the interior frames there are no beams and the columns are connected through a RC slab strip with dimensions 22×20cm (No Beams, NB). According to the typical characteristics of the frame building structures, the presence and the position of infill masonry walls along the exterior frames have also been considered, thus obtaining the types BF (Bare Frame), IF (Infilled Frame), and PF (Pilotis Frame) (Figure 1).

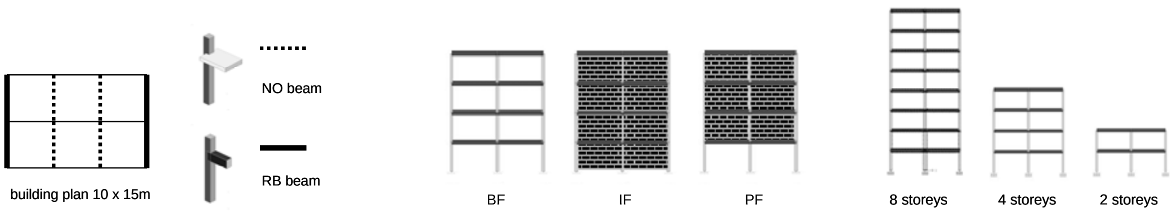


Figure 1. Characteristics of the structural types under study

The simulated design of the selected structural types has been performed taking into account only gravity loads, making reference to the codes in force, the available handbooks and the typical current practice of the period (Masi, 2003). According to standards of the '70s, simulated design has been performed adopting the allowable stress method in the safety verifications, and considering mechanical properties of materials typical of the period: medium quality concrete C 20/25 and steel with grade close to S400 type. The columns have been designed for axial load only and adopting the minimum requirements provided in the Italian code of the period. The beams have been designed on the basis of the simplified model of continuous beam resting on simple supports.

3. METHODOLOGY

A wide parametric study on the 3D models defined in section 2 has been carried out. For each model the Limit State of Significant Damage (LS-SD) has been verified considering four values of seismic intensity as provided by the Italian seismic code (OPCM 3274, 2003), that is a_g = 0.35g (Zone 1), a_g = 0.25g (Zone 2), a_g = 0.15g (Zone 3), and a_g = 0.05g (Zone 4). Further, three concrete strength values (f_{cm} = 10, 18, 28 MPa), one steel strength (f_{ym} = 400MPa), and three confidence factor values (CF = 1, 1.2, 1.35) respectively associated to the knowledge levels KL3, KL2, and KL1, have been considered. Two analysis methods have been adopted according to the EC8 (CEN, 2003a) provisions:

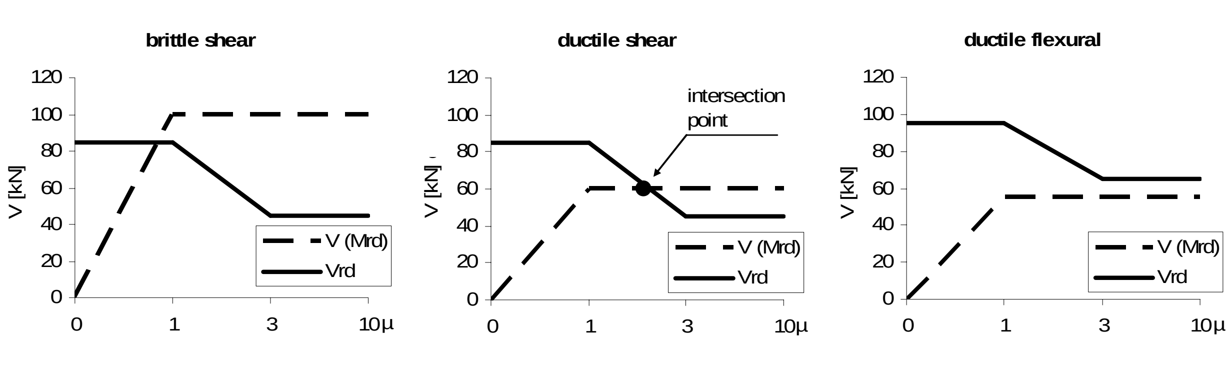
- Linear Dynamic Analysis (LDA) with "q-factor" approach;
- Non Linear Static Analysis (NLSA).

The NLSA analysis has been performed also in case of knowledge level KL1, even if this is not allowed by EC8-3. The elastic response spectrum provided in the EC8 (CEN 2003a) for ground type B, has been used. In LDA analysis the ordinates of the elastic spectrum have been reduced adopting four values of the behavior factor (q = 1.5, 2, 2.5, 3). The FE code used for both linear dynamic and non-linear static analyses is SAP2000 (1995). In NLSA a macro-modeling based on lumped plasticity has been adopted. At both ends of each structural member (beams and columns) a bending moment – rotation relation has been defined through a bi-linear curve described by the values of the yielding moment (M_y) and chord rotation (θ_y), and of the ultimate chord rotation (θ_u). θ_y and θ_u have been evaluated according to EC8-3. Moment values have

been computed according to EC2 (CEN, 2003b) considering a parabola–rectangle diagram for concrete under compression, characterized by maximum and ultimate strength equal to f_{cm}/CF, no tensile strength, strain at peak stress ε_{co}=0.002, and unconfined ultimate strain ε_{cu}=0.0035. An elastic–perfectly plastic steel stress–strain diagram is considered, characterized by a maximum strength equal to f_{sm}/CF and ultimate strain ε_{su}=0.01. Stiffness properties of members have been taken equal to the corresponding stiffness of the uncracked members, and assuming the concrete modulus of elasticity (E_c), computed according to EC2, as a function of the concrete strength f_{cm}. As regards the infills, in RC existing buildings, they usually are made of two layers of hollow brick masonry with a total thickness t_w equal to about 200 mm and scarce mechanical characteristics. Each masonry panel has been modelled by using an equivalent diagonal strut, whose area is determined by multiplying the panel thickness t_w by an equivalent width b_w. The expression due to Mainstone (1974), relevant to rectangular masonry panels, has been used to compute b_w.
The NLSA has been performed using two lateral force distributions, having a "uniform" and a "modal" pattern, as provided by EC8-3. In both analysis methods, vertical loads (G_K and Q_K = 2.0 kN/m^2, respectively the characteristic values of dead and live loads) and seismic action (E) are combined in the following way: $\gamma_I E + G_K + \psi_2 Q_K$, where γ_I is the importance factor assumed equal to 1, and ψ_2 is the combination coefficient for variable action assumed equal to 0.3.
As regards the safety verifications performed in the LDA, the demand values D obtained from the analysis, have been compared to the corresponding strength capacities C. In the NLSA safety verifications have been performed checking that demands due to the seismic action do not exceed the corresponding capacities in terms of deformations for ductile elements and in terms of strengths for brittle elements. Capacity in brittle elements has been computed in terms of cyclic shear resistance according to EC8-3, adopting the expression (1) that takes into account the reduction of the shear capacity with the plastic part of the chord rotation ductility (μ_Δ^{pl}):

$$V_{Rd}=\frac{1}{\gamma_{el}}\left\{\frac{h-x}{2L_V}\min(N;0.55A_c f_c)+(1-0.05\min(5;\mu_\Delta^{pl}))\cdot\left[0.16\max(0.5;100\rho_{tot})(1-0.16\min(5;\frac{L_V}{h}))\sqrt{f_c}A_c+V_w\right]\right\} \quad (1)$$

It is worth noting that the above evaluation of shear capacity could be, in general, not trivial. In the present work, the simplified approach suggested in Mpampatsikos et al. (2008) has been adopted. Although the shear failure should be always regarded as a brittle mechanism, two different behaviors, based on the procedure described in Mpampatsikos et al. (2008), can be considered, named "brittle shear" and "ductile shear". In case of brittle shear failure (Figure 2a), in Eq. 1 μ_Δ^{pl} has been assumed equal to zero, while in case of ductile shear failure (Figure 2b) a specific value of μ_Δ^{pl} has to be computed (intersection point in Figure 2b).
Really, in the structures under study, members show generally a brittle shear failure, with the exception of the columns of the two upper storeys where a ductile flexural failure (Figure 2c) can be predicted. Finally, when applying Eq. 1, it must be noticed that the shear strength for beam members is slightly sensitive to concrete strength f_c variation, while column shear strength results more dependent on f_c values.



(a) (b) (c)
Figure 2. Different failure mechanisms: brittle and ductile shear failure, and ductile flexural failure.

4. RESULTS

4.1. Linear Dynamic Analysis (LDA)

Some results obtained using the LDA with “q factor” approach are summarized in this section describing, for sake of brevity, only the results relevant to the mid-rise (4 storey) structural types. However, the main reported conclusions can be extended also to the low- and high-rise buildings. Whichever seismic zone is considered, the safety verifications in every structural types, at the Limit State of Significant Damage, are not satisfied, that is at least one member has D/C > 1. In Figure 3 the mean, maximum and minimum D/C values, separately for beams and columns, and considering q = 3 and Zone 1 (a_g=0.35g), are displayed as a function of the concrete strength $f_d = f\,(KL, f_c)$. Figure 3 highlights a somewhat different trend of D/C values between beams and columns. As regards the columns, D/C values (for the same KL), decrease going from f_c10 to f_c28, as it could be expected. On the contrary, the D/C values (for the same KL) in the beams increase with the concrete strength, thus showing an anomalous trend. Two are the main factors influencing these results:

a) as a consequence of the shape of the considered response spectrum, the frames with f_c28 concrete, being stiffer, are generally subjected to higher seismic effects than the more flexible frames with f_c10 concrete;
b) the shear resistance computed using Eq. 1 increases with the concrete strength, but V_{Rd} values for beam members are not very sensitive to f_c variation.

In beams the role of factor a) is dominant so demand increases more than capacity when increasing f_c, while the contrary happens for columns.

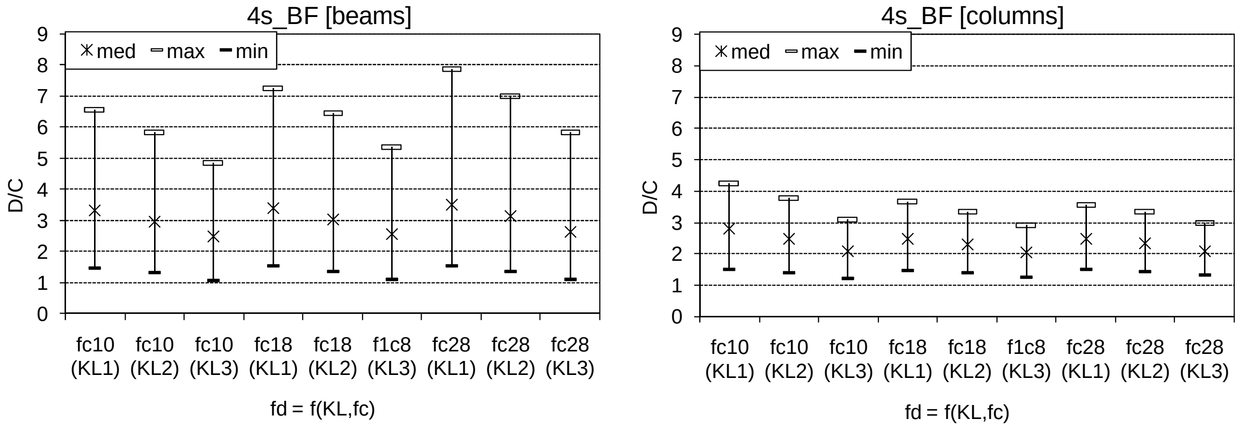


Figure 3. Maximum, mean and minimum D/C values from LDA (Zone 1, q = 3).

Results of the analyses comparing different configurations of masonry infills are shown in Figure 4, in terms of mean values of D/C ratios. Both for beams and columns, the D/C values in Bare Frame (BF) types are higher than those ones in Infilled Frame (IF) and Pilotis Frame (PF) types. Slightly higher differences appear in columns, where D/C mean values decrease from 2-3 (BF type) to 1-1.5 (IF type). Also as a consequence of the linearity of the analysis method, Pilotis Frame (PF) types show values close to those ones computed in IF types.

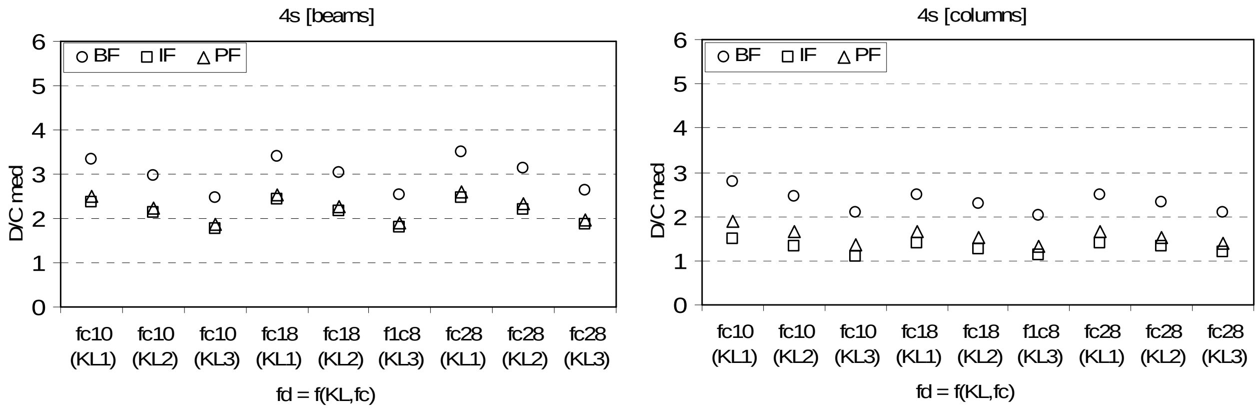


Figure 4. Role of infill configuration on mean D/C values from LDA (Zone 1, q = 3).

In figure 5 the percentages of not verified beam and column member sections, for BF, IF and PF types, are shown. The percentages of not verified beams are higher than those ones obtained for the columns. Further, percentages on columns show higher variations with respect to different infill configurations.

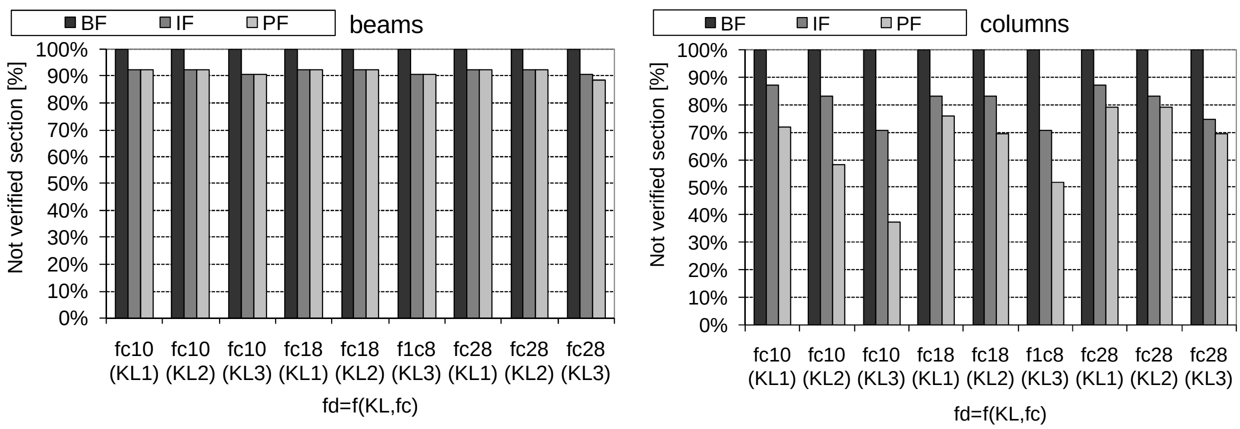


Figure 5. Role of infill configuration on percentages of not verified sections from LDA (Zone 1, q = 3).

4.2. Non-Linear Static Analysis (NLSA)

Some results obtained using the NLSA, together with some comparisons with the LDA results, are summarized in this section, again reporting only the performances of the mid-rise (4 storeys) structural types. Also adopting the NLSA method, whichever seismic zone is considered, the safety verifications at the Limit State of Significant Damage, in every structural types, are not satisfied. In Figure 6 the mean, maximum and minimum D/C values for beams and columns, and considering Zone 1 (a_g=0.35g), are displayed as a function of concrete strength $f_d = f\,(KL, f_c)$. For both members (beams and columns) the D/C values decrease from KL1 to KL3 (considering the same value of concrete strength f_c); on the contrary an unclear trend appears going from f_c10 to f_c28 but considering the same KL.

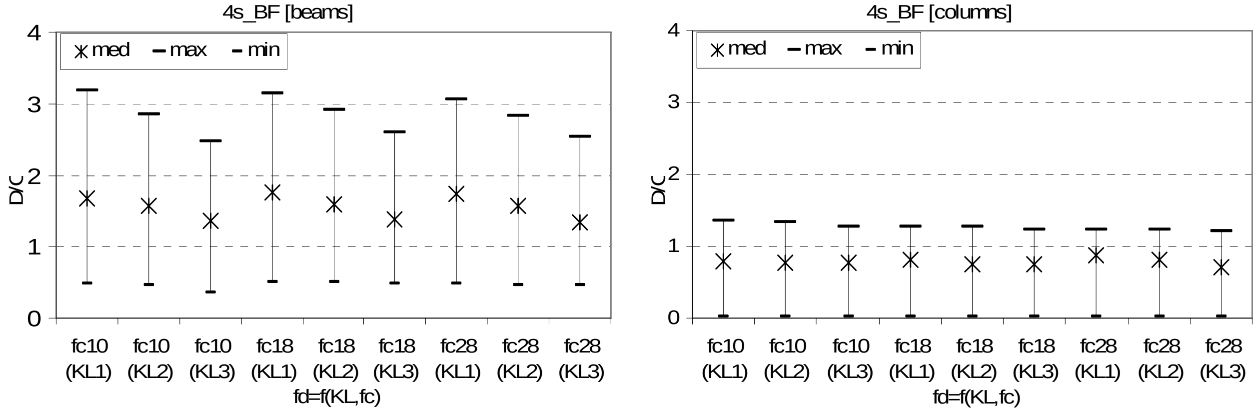


Figure 6. Maximum, mean and minimum D/C values from NLSA (Zone 1).

Generally, D/C values obtained from NLSA are lower than those ones obtained from LDA. For example, in the beams the max D/C value from NLSA is around 3, increasing up to 8 in LDA (Fig. 3); still, as for the columns with concrete strength f_c28 and KL3, the D/C mean values are equal to 2.0 and 0.7, respectively for LDA and NLSA.

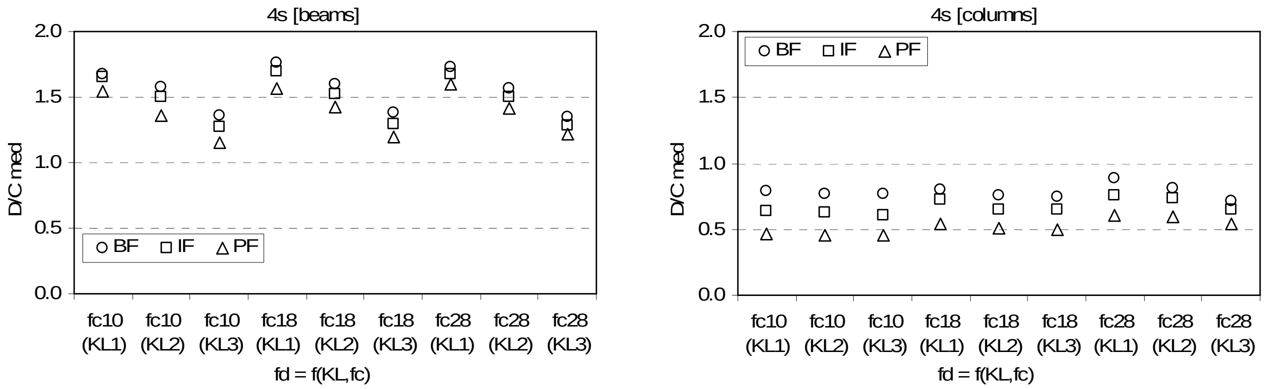


Figure 7. Role of infill configuration on mean D/C values from NLSA (Zone 1).

Results of the analyses comparing different configurations of masonry infills are displayed in Figure 7 in terms of mean values of D/C ratios, showing that $(D/C)_{med}$ values computed for the members of PF types are lower than the respective values in the BF and IF types. Such an apparently anomalous trend can be explained by examining also the results in Figure 8 where the percentages of not verified member sections across the entire structure of BF, IF and PF types, are compared. In general terms, Fig. 8 shows that the number of not verified column sections obtained from NLSA is significantly less than that one obtained from LDA. Specifically, the number of not verified sections in the columns of PF type is rather low and significantly lower than in BF and IF types, emphasizing a less favourable redistribution of action effects and less widely spread energy dissipation across the entire structure in the PF type. For this reason, considering mean values of D/C ratios, lower values for the PF type are generally computed. On the contrary, the max value of D/C ratios are generally higher in PF types; for example considering a low seismic intensity (zone 4), $(D/C)_{max}$ is equal to 1.17 and 0.81, respectively for PF and IF type.

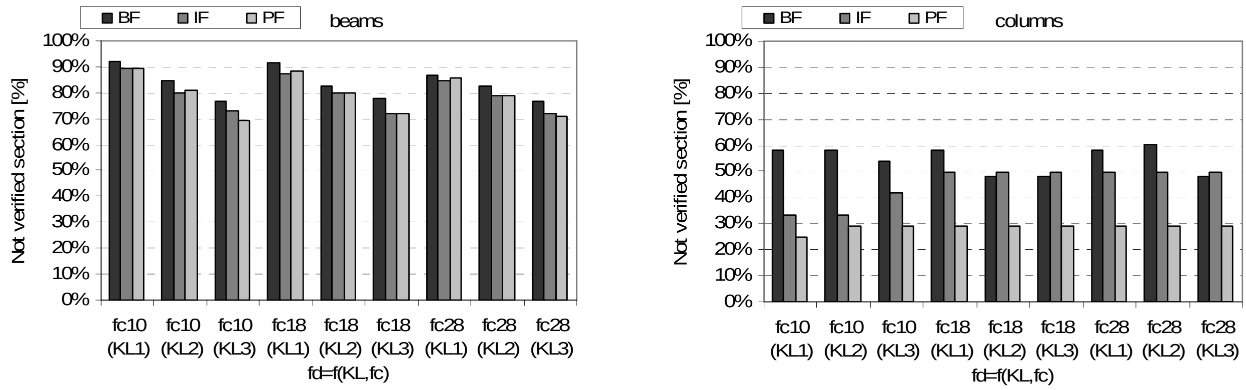


Figure 8. Role of infill configuration on percentages of not verified sections from NLSA (Zone 1).

In Figures 9 and 10 some results obtained from LDA and NLSA are shortly compared in terms of percentages of not verified elements at each storey for the 4-storey structural types (BF, IF and PF types), considering Zone 1, f_c18 and KL3, and assuming q = 3 in the LDA. As already said, for both methods, the evaluation of the seismic performance at the Limit State of Significant Damage is always not satisfied. However, it is worth noting that the number of not verified elements provided by LDA is greater than that one provided by NLSA. Specifically, examining the results from LDA relevant to the BF type, all the members are not verified at every storey; while, adopting NLSA, lower percentages of not verified sections are obtained, particularly for the columns at the two upper storeys where a ductile flexural failure was predicted. Similar results are found for IF and PF types.

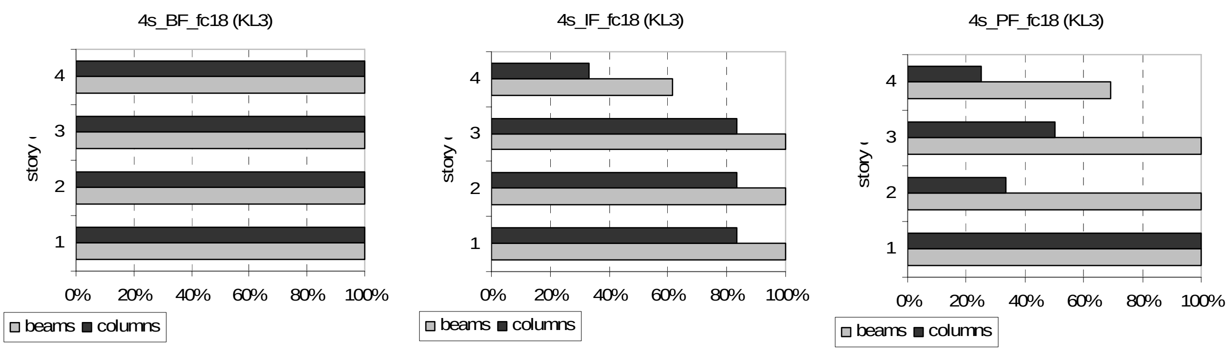


Figure 9. Results from LDA (Zone 1, q=3): percentages of not verified members at each storey.

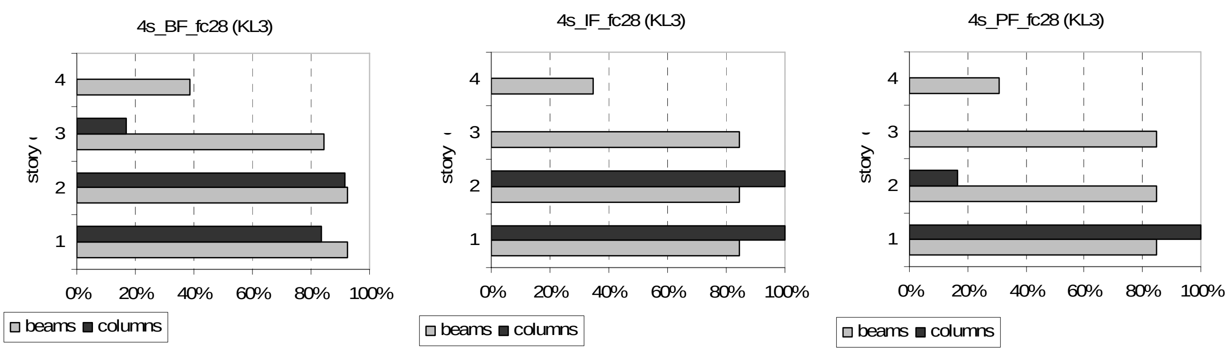


Figure 10. Results from NLSA (Zone 1): percentages of not verified members at each storey.

5. CONCLUSION

A wide parametric study on structural types with different number of storeys, and presence and position of

masonry infills, representative of post-1970 RC existing buildings, has been carried out. Structures designed only for vertical loads have been considered detailed through a simulated design procedure (Masi, 2003) where reference to the codes in force, the handbooks and the current practice of ‘70s in Italy, is made. The performances at the Limit State of Significant Damage (LS-SD) adopting four seismic intensities, three concrete strength values, and three Knowledge Levels, have been evaluated using two analysis methods provided in the European seismic code (CEN 2003a, 2004), that is Linear Dynamic Analysis (LDA) and Non Linear Static Analysis (NLSA). The results have been examined mainly in terms of Demand/Capacity (D/C) ratios in the structural members, and after compared mainly to understand the relative coherence of the two analysis methods.
Comparing the results from the two methods of analysis under study, it arises that both the number of not verified elements and the mean values of D/C ratios obtained from LDA are higher than those ones obtained from NLSA. As regards the outcome of the assessment, the results have shown that safety verifications are always not satisfied with both analysis methods, whichever parameter values are adopted, and also in case of the lowest seismic intensity (Zone 4, peak ground acceleration $a_g = 0.05g$). Main cause of these results can be ascribed to the capacity models for assessment provided by EC8-3 (CEN, 2004) for beam and column members under shear. Adopting the expression provided to calculate the cyclic shear resistance (Eq. 1, see section 3) most of the members show a brittle failure. Moreover, such expression is ductility-dependent, then an evaluation of the non linear behaviour of the member sections is required also for LDA. For this reason future developments of the study aim at thoroughly investigating the performance of RC members under cyclic shear, also to provide a reliable as well as simpler procedure to recognize ductile and brittle members.

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