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INFLUENCE OF BOND-SLIP ON NUMERICAL FRAGILITY CURVES AND STRUCTURAL RELIABILITY OF RC STRUCTURAL INTERNAL BEAM-COLUMN SUB-ASSEMBLY

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Abstract

In this paper results obtained from monotonic nonlinear static analyses performed on Reinforced Concrete (RC) internal beam-column sub-assembly are shown. Bond-slip phenomenon between steel longitudinal bars and surrounding concrete is also taken into account in order to predict the numerical response under lateral actions of the RC internal beam-column sub-assembly investigated.

The study is addressed, through parametric models and Monte Carlo simulations, to propose preliminary fragility curves for different damage states of the RC internal beam-column sub-assembly, including materials inherent uncertainties.

Keywords: Bond-Slip, damage states, fragility curves, structural reliability, material uncertainties, Monte Carlo simulations, RC internal beam-column sub-assembly.

1 INTRODUCTION

In recent years, structural engineering has increasingly focused on use and development of methodologies for assessing the structural performance based on a probabilistic approach. In this framework, structural or component performance is expressed through a reliability index, related to the failure probability that is the probability that capacity does not satisfy the corresponding demand. To this scope, to date several approaches for calculating structural reliability can be found in the scientific literature, including FORM (First-Order Reliability Methods), SORM (Second-Order Reliability Methods) and numerical methods (Monte-Carlo simulation) [1].

Seismic assessment including economic losses may be carried out following different approaches. For instance, methodologies for seismic risk assessment based on macro-seismic approach have been recently developed, based on an empirical approaches starting from seismic damage observed on several buildings stocks after seismic events ([2]–[4]).

On the contrary, recently the Performance-Based Earthquake Engineering (PBEE) approach has been developed by Pacific Earthquake Engineering Research (PEER) [5], based on a numerical and probabilistic approach for seismic risk assessment of buildings (Fig.1). Also, the Applied Technology Council (ATC), on commission of Federal Emergency Management Agency (FEMA), proposed guidelines containing the PBEE methodology applicable to both new and existing buildings ([6], [7]).

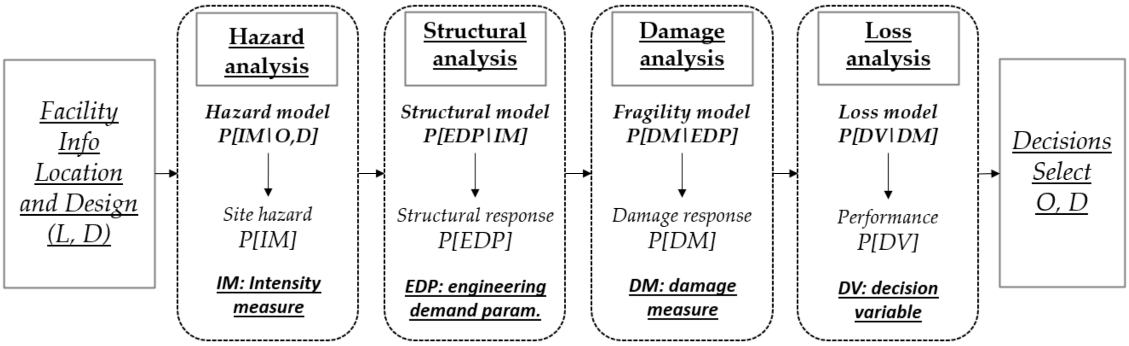


Fig. 1: PBEE schematization [5]

According to the PBEE approach (Fig. 1) Damage Measure – Engineering Demand Parameter (DM-EDP) relationships have a central role in performing the loss analysis. They are often called fragility curves, expressing the exceedance probability of a certain a damage level for a given EDP. The FEMA database P-58 [8] includes a variety of fragility curves of existing reinforced concrete structural and non-structural elements typical of American construction practice. Naturally, the PBEE methodology may be implemented for any type of buildings, requiring in this case specific fragility curves referred to the buildings typologies taken into account. To this scope, to date several studies exist in the literature focusing on the seismic response of existing Reinforced Concrete (RC) buildings designed only for gravity loads with smooth steel bars. As known, under seismic lateral excitations their response is dominated by bond-slips between longitudinal bars and surrounding concrete conspicuously reducing the elements hysteretic capacity ([9], [10], [11]).

In this paper, according to the PBEE methodology, fragility curves of internal beam-column sub-assemblages representative of existing RC buildings built in Italy before '80s with smooth bars are proposed. Monotonic analyses including bond-slips are performed, where uncertainties of material properties (concrete and reinforcing steel) are considered through Monte Carlo simulations. For this purpose, fragility curves tool is at first extended in order to provide the probability of having a certain section condition (such as yielding in tension or compression, maximum moment) for a given section EDP. Afterwards, fragility curves are derived by varying a given EDP. The results obtained are shown with and without bond-slips and referred to different axial stress ranges.

2 NUMERICAL SIMULATIONS

In this study a internal beam-column joint representative of an existing RC building built in Italy before '80s and reinforced with smooth bars is considered (Fig. 1a). The internal joint was tested in [9], where a set of typical internal and external existing RC joints were tested under lateral displacements. More details about this experimental campaign and related results may be found in [9].

Numerical simulations are conducted by means of a Finite Element Model (FEM) of an internal beam-column joint implemented into OpenSees software v. 3.3.0 [12][13] through Python language [14]. ForceBeamColumn fiber elements [15] with a HingeRadau integration method ([16]–[18]) are used, assuming each element elastic in the central region with a fiber length (plastic hinges) only at the element ends (Fig. 1b). Moreover, the nodal panel is considered as a rigid by means of rigid elements. Non-linear stress-strain relationships are considered for materials. In particular, Concrete04 [19] is assigned to concrete fibers, and Steel02 [20] for longitudinal steel in the case of Full-Bond (FB) assumption. Whereas, a multilinear material is assumed when a Model with bond-Slip (MS) is taken into account. The instability of longitudinal bars and the possible interaction between flexural and shear capacity are neglected in this study.

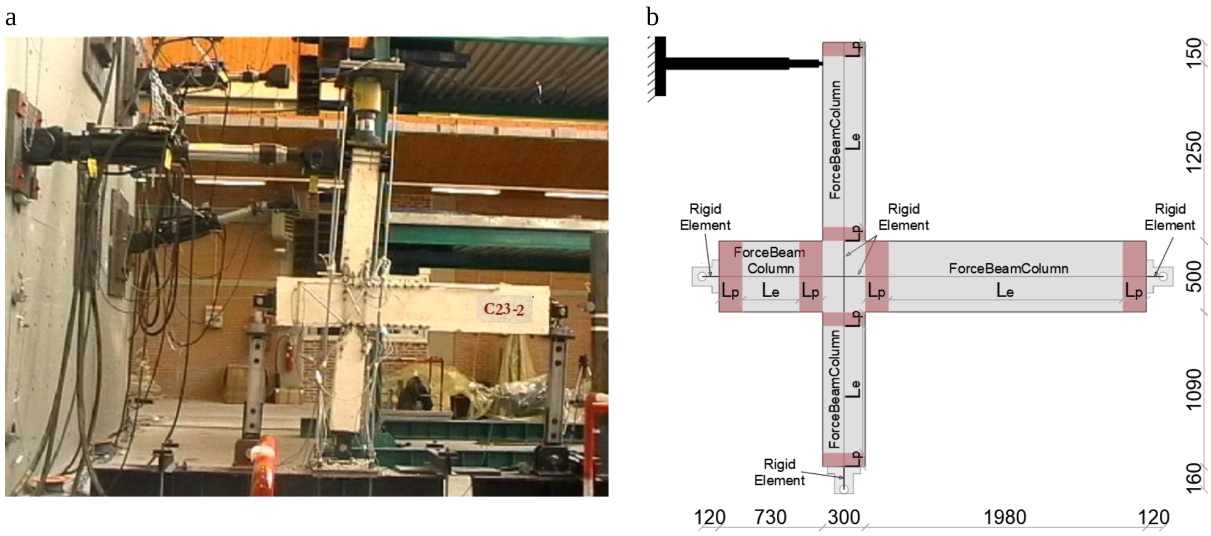


Fig. 2. Internal beam-column sub-assembly: (a) experimental test [9] (b) FEM schematization

Slip phenomenon is considered through a numerical model modifying steel stress-strain relationships of each longitudinal smooth bar for incorporating the Bond-Slips of the longitudinal bars with respect to the surrounding concrete [21]. The model was validated in [22] and subsequently developed to also account for steel hardening [23]. In [24] the model was further extended considering an exponential field for slips, including also a cyclic formulation.

In this study Monte Carlo simulations are performed to consider uncertainties related to material properties. As for bars, AQ50 steel is used usually used in Italy in '50s-'70s [25]. In particular, an average yield stress f_{ym}=370,9 MPa with a Coefficient of Variation (CV) of 0,09 are considered. Whereas, for concrete [26] a compressive strength f_{cm}=19.53 MPa with a

CV=0.37 are assumed. Finally, the residual bond strength is evaluated according to the formulation proposed in CEB-FIP (2008). Fig. 3 shows in the histograms form for concrete and steel the strength recurrence sampled by means of Monte Carlo simulations. In the same graphs, the Probability Density Function (PDF) considering a normal and log-normal distribution is plotted, too.

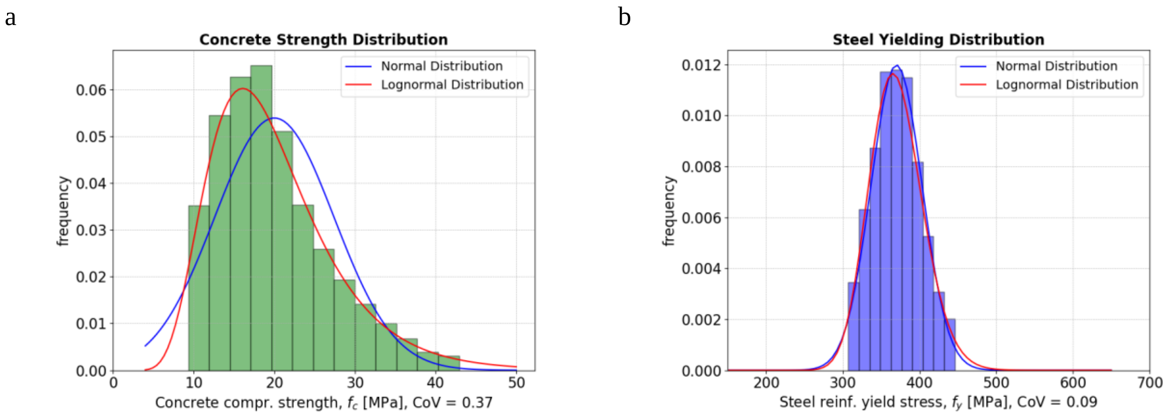


Fig. 3. Material theoretical distribution: (a) Concrete Compressive Strength, (b) Steel Yielding

Table 1 report elements sections details of the internal beam-column joint considered in this study. Numerical investigations are carried out for different axial load ratio ranges ν (10-20%, 20-30%, 30-40%), without considering any P-Δ second-order effect.

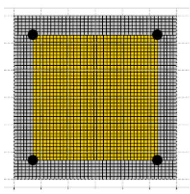
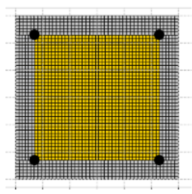
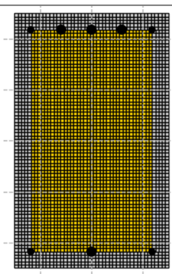
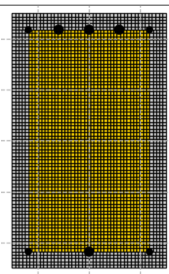
		Lower column	**Upper column**	**Left beam**		**Right beam**	
Length	L [m]	1.25	1.25	0.85		2.1	
Section	B [m]	0.3	0.3	0.3		0.3	
	H [m]	0.3	0.3	0.5		0.5	
Top reinforcement	φ [mm]	18	18	12	18	12	18
	n	2	2	2	3	2	3
Bottom reinforcement	φ [mm]	18	18	12	18	12	18
	n	2	2	2	1	2	1
Fiber layout							

Table 1: Elements Details

3 RESULTS

The numerical model adopted for the internal beam-column joint has been firstly validated by comparing it with experimental results. Fig. 4 depicts a comparison in terms of lateral force – displacement at upper column top relationship (F-Δ) between numerical and experimental results [9], [22] are reported. Results are referred to both Full-Bond (FB) and Bond-Slip (BS) of longitudinal bars. Moreover, in the same graph the chord rotation (drift) measured between the lower column base and upper column top is indicated, too. By monitoring the fibers stress-strain state of the ends sections, it is possible to determine the achievement of several section states (i.e. limit state), such as concrete cracking (when ε_{ct} is measured at the external deformation of the cover fibers), steel compressive and tensile yield strength (when ε_{sy} is measured in the reinforcement bars), partial and total failure of the concrete cover (when ε_{co} and ε_{cu} is measured at the external or internal deformation of the cover fibers), maximum strength (when the internal beam-column sub-assembly is at maximum force), concrete core failure (when ε_{cu} is measured within the confined core), steel bar failure (when ε_{su} is measured in the reinforcement bars). As one may note in Fig. 4, bond-slips effects significantly reduce the lateral stiffness of the internal beam-column joint obtaining a good agreement with the experimental results.

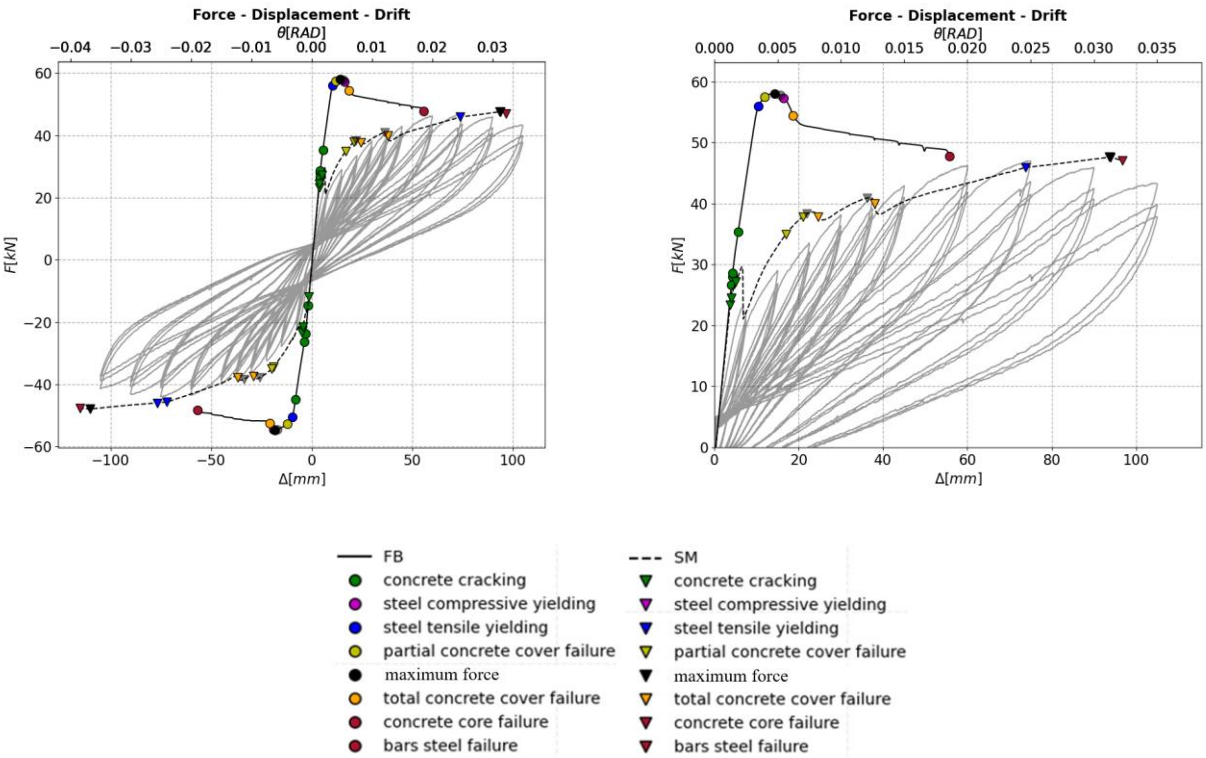


Fig. 4: comparisons with experimental results

From the results obtained, the cumulative probabilistic capacity distribution referring to some particular element conditions can be derived. In Fig.5, for simplicity's sake, the representative element fragility capacity curves refer only to three element conditions, the steel yielding strength in compression (F_{yc}) and in tension (F_{yt}), and maximum yielding force (F_{max}). As one may note that, for a given probability, with the Bond-Slips a force always lower than the one obtained with the Full-Bond assumption is obtained. In this figure three axial load ratio intervals are considered, that are: ν=10-20%, ν=20-30%, ν=30-40%. It is easy to note that when the axial load ratio values increase the tension yielding of longitudinal bars do not occur (Fig.5 b, c).

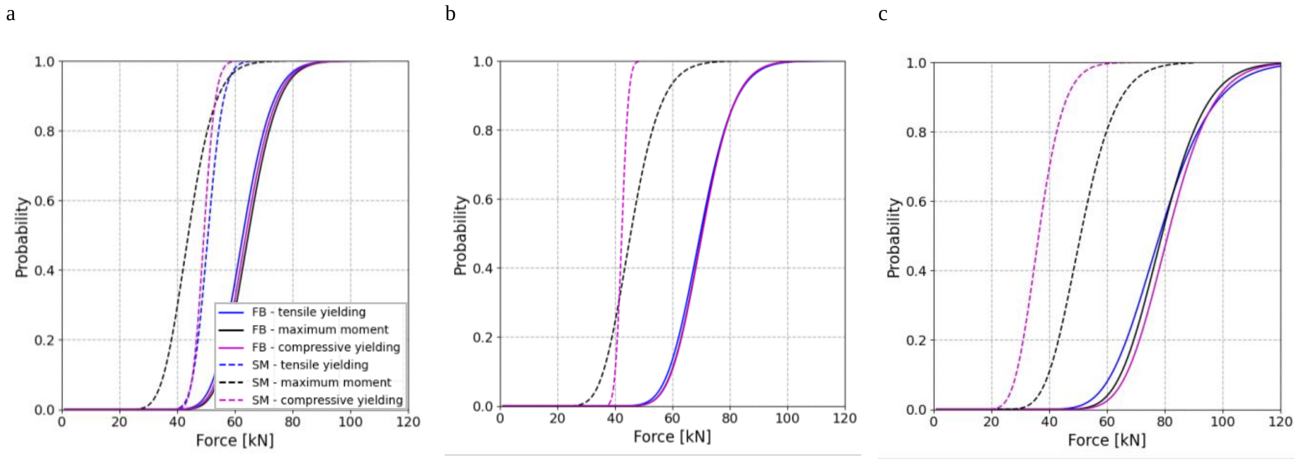


Fig. 5: Probability of having $F_{yt} \leq F^*$, or $F_{yc} \leq F^*$, or $F_{max} \leq F^*$: (a) ν=10-20%, (b) ν=20-30%, (c) ν=30-40%

For completeness a reliability evaluation is also conducted in order to show how bond-slips may affect the reliability index. To this purpose, by assuming an equal demand probability distribution for both the Full-Bond and Bond-Slip models, in Fig.6 the reliability index is calculated with the FORM approach [27]. It is clear to observe that Bond-Slip significantly reduces the reliability of the internal beam-column sub-assembly with respect to the Full-Bond assumption.

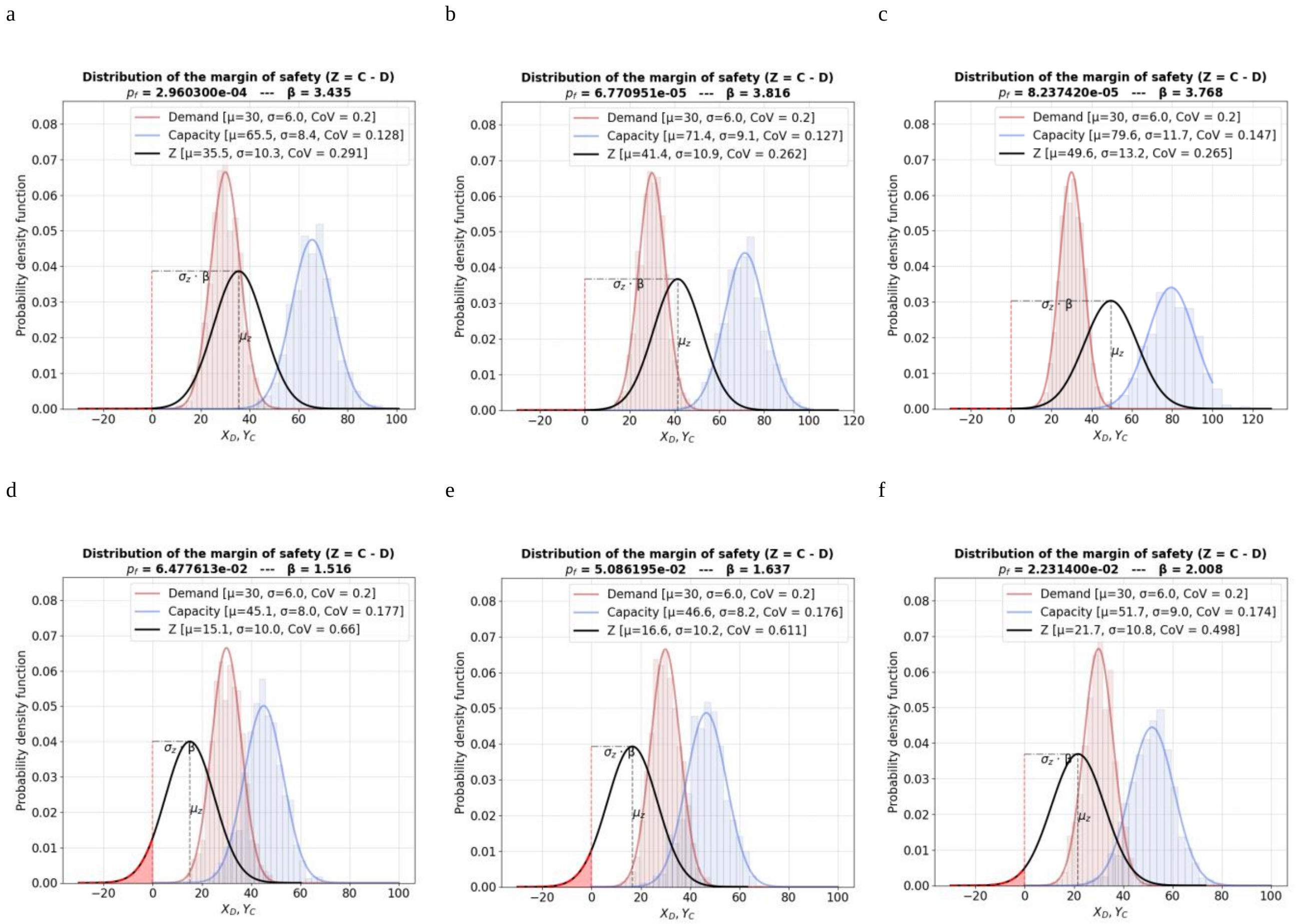


Fig. 6: Reliability index: Full-Bond (a) ν=10-20%, (b) ν=20-30%, (c) ν=30-40%; Bond-Slip (d) ν=10-20%, (e) ν=20-30%, (f) ν=30-40%

The effects of Bond-Slips on the Inter-story Drift Ratio (IDR) are illustrated in Fig. 7. It can be seen that, for a given probability, the IDR corresponding to the three element conditions considered in the Full-Bond case is always lower than that with Bond-Slip. This shows that the Bond-Slip delays the yield point of the steel. Furthermore, in the context of a loss analysis, the curves obtained by considering the Bond-Slip (Fig. 6) would provide a certain advantage, since the probability of obtaining a certain IDR is lower than with the Full-Bond hypothesis. Of course, this conclusion will be better studied in the future, as the definition of damage levels with Bond-Slip should be clearly defined within a seismic risk framework, including a correlation with the required repair cost. These aspects will be further explored in the future.

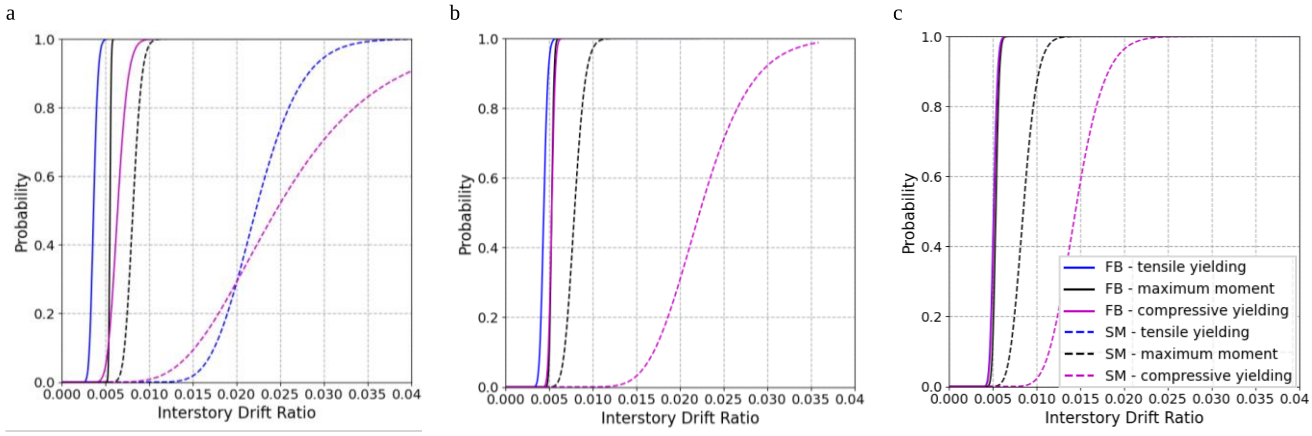


Fig. 7. Probability of having $IDR_{Fyt} \leq IDR^*$, or $IDR_{Fyc} \leq IDR^*$, or $IDR_{Fmax.} \leq IDR^*$: (a) ν=10-20%, (b) ν=20-30%, (c) ν=30-40%

4 EVALUATION OF FRAGILITY FUNCTIONS

In order to derive the fragility curves, it is necessary to identify the damage states [28] for elements in which Bond-Slip occurs. In this study, three damage states (DS1, DS2, DS3) are assumed for low-detail elements reinforced with plain bars, which are identified as a function of the damage state of the cover and core.

Numerically, the damage state can be expressed through the deformation state of the section of the elements. In particular, with regard to the damage state of the concrete cover, it is assumed that: DS1 is reached when the external unconfined concrete deformation ε_{c0} is measured; DS2 corresponds to cover failure (when ε_{cu} is measured at the external deformation of the cover fibers); and DS3 is assigned when the ultimate deformation of the confined concrete is obtained (when ε_{cu} is measured within the confined core).

Fragility curves are derived by knowing the parameters θ and β, that may be derived with the maximum likelihood criterion, where the likelihood function is the following [29], [30]:

$$\mathcal{L}(\vartheta,\beta) = \prod_{j=1}^{m} \binom{k_j}{n_j} \Phi\left(\frac{\ln\left(\frac{EDP}{\vartheta}\right)}{\beta}\right)^{k_j} \left[1 - \Phi\left(\frac{\left[\ln\left(\frac{EDP}{\vartheta}\right)\right]}{\beta}\right)\right]^{n_j-k_j} \qquad j = 0, \dots, m \tag{1}$$

where the binomial probability distribution is assumed for calculating the probability of observing k_j analysis with a damage equal or greater than a specific value; n_j is the total number of analyses.

Hence, the parameters θ and β for each fragility function are obtained by maximizing the logarithm of the likelihood function, which is expressed with:

$$(\hat{\vartheta},\hat{\beta}) = arg\,max \sum_{j=1}^{m} \left\{ ln\binom{k_j}{n_j} + k_j ln\left[\Phi\left(\frac{\ln\left(\frac{EDP}{\vartheta}\right)}{\beta}\right)\right] + (n_j - k_j) ln\left[\Phi\left(\frac{\ln\left(\frac{EDP}{\vartheta}\right)}{\beta}\right)\right]\right\} \quad j = 0,\ldots,m \qquad (2)$$

where (θ, β) correspond, respectively, to the median value and the standard deviation of the logarithm of EDP, allowing to have the most likely fragility curve.

Fig. 8 plots the fragility curves obtained for existing RC internal beam-column sub-assembly designed for vertical loads for the concrete damage states, considering both Full-Bond (continuous curves) and the Bond-Slip (dashed curved) and three intervals of axial load ratio. From the results obtained, it can be stated that the Bond-Slip phenomenon induces a rightward shift of the fragility curves, due to greater deformability of internal beam-column sub-assembly.

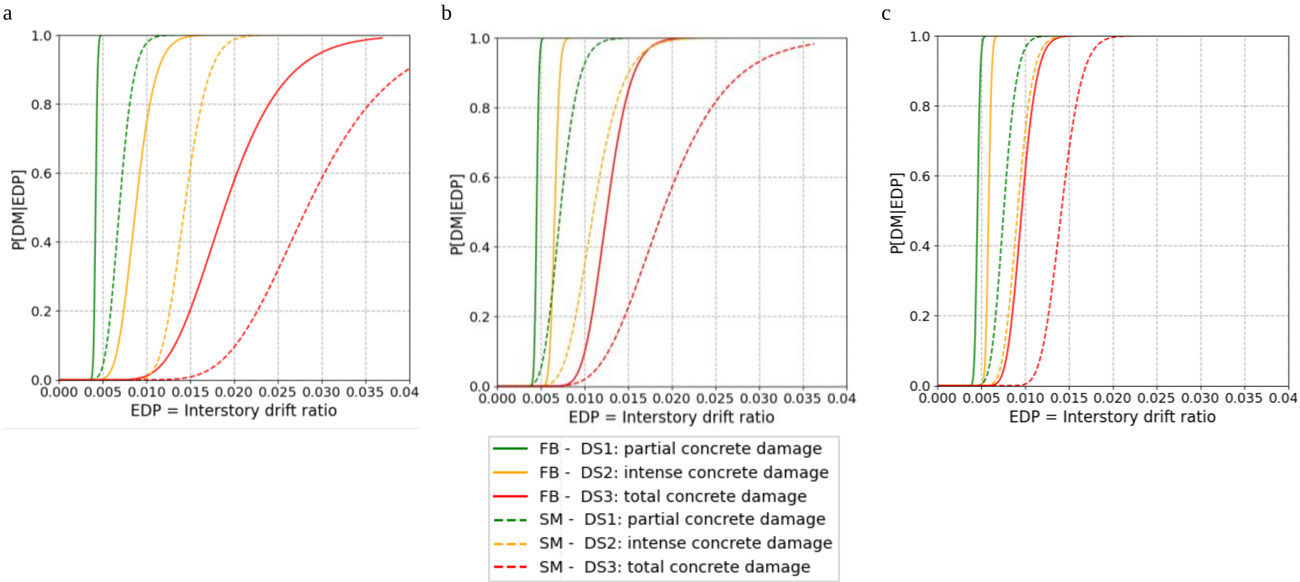


Fig. 8. Fragility functions for the damage level of concrete: (a) ν=10-20%, (b) ν=20-30%, (c) ν=30-40%

5 CONCLUSIONS

In this work, monotonic non-linear static analyses have been performed on an existing RC internal beam-column sub-assembly with smooth bars, in order to propose preliminary fragility curves to be implemented within the PBEE framework. The internal beam-column sub-assembly considered is representative of typical existing RC buildings constructed in Italy before the 1980s, and designed for vertical loads with smooth bars. The fragility curves have been derived including the Bond-Slip phenomenon [21], and taking into account the materials uncertainties by means of Monte Carlo simulations.

The results obtained confirm that, according to the PBEE methodology, the Bond-Slip has a relevant impact on the structural analysis, modifying the response of the elements in terms of both strength and deformability. In addition, it has been shown that the phenomenon of slip significantly reduces structural reliability. Furthermore, it is clearly illustrated that, for the element analyzed, Bond-Slips can significantly modify the fragility curves of the concrete damage state compared to those derived with the Full-Bond hypothesis.

The proposed method may be adopted in the future for the development of fragility curves of other structural components to be implemented within the PBEE methodology in order to perform seismic risk analyses.

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