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Influence of bond-slip on numerical fragility curves of RC structural columns

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Abstract

In this paper results obtained from monotonic nonlinear static analyses performed on Reinforced Concrete (RC) columns 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 columns investigated.
The study is addressed, through parametric models and Monte Carlo simulations, to propose preliminary fragility curves for different damage states of the RC columns, including materials inherent uncertainties. Finally, the paper concludes comparing the proposed fragility curves with other similar ones published in the literature.

Keywords: bond-slip, damage states, fragility curves, material uncertainties, Monte Carlo simulations, RC columns

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1. Introduction

In recent years a new generation of methodologies for designing and assessing structures seismic performance including economic losses, based on a probabilistic approach, have been developed. Among these, the Pacific Earthquake Engineering Research (PEER) has developed the so-called Performance- Based Earthquake Engineering (PBEE) methodology (Moehle and Deierlein, 2004), that may be summarized as reported in Fig.1. According to the same method Applied Technology Council (ATC), on behalf of the Federal Emergency Management Agency (FEMA), has proposed guidelines containing the PBEE methodology applicable for both new and existing buildings. (ATC, 2018a, 2018b).

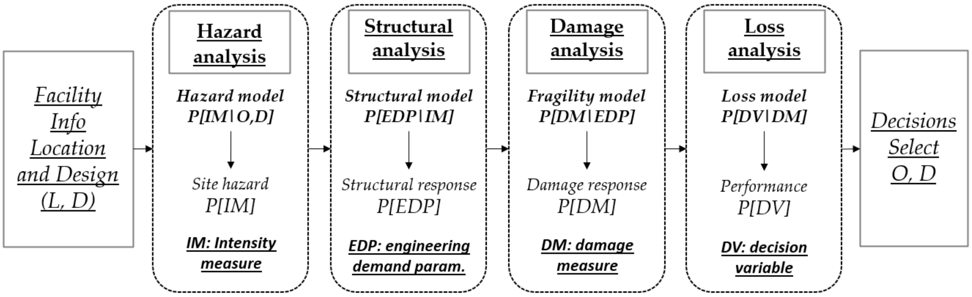

Fig. 1. Underlying probabilistic framework (Moehle and Deierlein, 2004)

As it can be seen in Fig.1, the PBEE methodology is divided into 4 phases. The first phase includes the description, definition and probabilistic quantification of the hazard by means of an Intensity Measure (IM) representative of the seismic action. In the second phase, with numerical analyses structural global and local response, including non-structural components, are performed monitoring an Engineering Demand Parameter (EDP). Afterwards, a Damage Measure (DM) is evaluated for each structural and non-structural element (or component) of the building. Finally, in the fourth phase, linking DM to a Decision Variable (DV), a loss analysis for each individual element (or component) may be evaluated, where usually DV is represented by the repair cost.

According to the PBEE methodology, element DM-EDP relationships play a central role in order to perform a loss analysis. DM-EDP relationships are frequently named element fragility curves, and they express the probability of having a damage level as a function of a certain EDP.

In this paper particular attention is paid to the construction of fragility curves of existing Reinforced Concrete (RC) elements, designed only for vertical loads having smooth longitudinal bars. As known, under seismic loads they are affected by significant bond-slip between bars and surrounding concrete. Bond-slips govern the structural response both at element and global level inducing a loss of stiffness strength and ductility (Braga, Gigliotti and Laterza, 2009; Laterza, D'Amato and Gigliotti, 2017).

FEMA P-58 database includes a wide variety of fragility curves of existing RC structural and non-structural elements typical of American construction practice. Of course, if one would apply the PBEE methodology to Italian buildings, specific fragility curves should be available. To date, there are several studies in the literature focusing on seismic response of existing RC buildings designed only for gravity loads with smooth bars. Among the others, in (Mohammad *et al.*, 2018) Incremental Dynamic Analyses (IDA) are conducted on Italian buildings. While an application of the entire PBEE methodology, always on Italian building typology, is presented in (Romano *et al.*, 2018). In this study the fragility curves are derived from experimental tests on existing RC sub-assemblages (Braga, Gigliotti and Laterza, 2009).

In this work, according to the PBEE methodology, fragility curves of existing RC columns, subjected to axial and bending moment, are derived including bond-slip of longitudinal bars and representative of buildings built in Italy before '80s. Monotonic analyses on RC cantilevers are carried out, where the materials properties uncertainties are considered through Monte Carlo simulations. To this scope, at first curve providing the probability of having a certain element condition, in terms of strength and deformability, are shown. Then, elements fragility curves are derived. The results are shown with and without bond-slips and referring to different intervals of axial load.

2. Bond-slip modelling

In the this work, a simplified model modifying the steel stress-strain for incorporating bond-slip of longitudinal bars with respect to the surrounding concrete is used (Braga *et al.*, 2012). The model has been validated in (D'Amato *et al.*, 2012), and subsequently further developed for taking into account also steel hardening (Braga *et al.*, 2015) and for simulating the cyclic behavior (Caprili *et al.*, 2018).

3. Numerical Simulations

Monotonic analyses on RC cantilevers are conducted by using OpenSees software (McKenna et al 2000, Mazzoni et al. 2006) with Python programming language (Zhu, McKenna and Scott, 2018).

In order to consider the materials properties uncertainties Monte Carlo simulations are performed. Mechanical parameters for longitudinal steel are representative of typical Italian values adopted within a 30-years interval, from 1959 to 1980, according to the work presented in (Verderame *et al.*, 2011). As for concrete, variability reported in (Masi, Digrisolo and Santarsiero, 2014) from 1961 to 1971 is considered. Respectively, for steel a mean yield strength f_{ym}=356.5 MPa with a coefficient of variation (CV) equal to 0.19 are assigned. While, for concrete, a compressive mechanical strength f_{cm}=19.53 MPa with a CV=0.37 are considered. Finally, the residual bond strength is evaluated in accordance with the formulation proposed in CEB-FIP (2008). Fig. 2 plots, in the form of histograms, of recurrence of the strength sampled with the Monte Carlo simulations for the concrete compressive strength, steel yielding and residual bond strength. In the same graphs the Probability Density Function (PDF) considering a normal and log-normal distribution are plotted. Histograms are referred to a number of about 50 samplings.

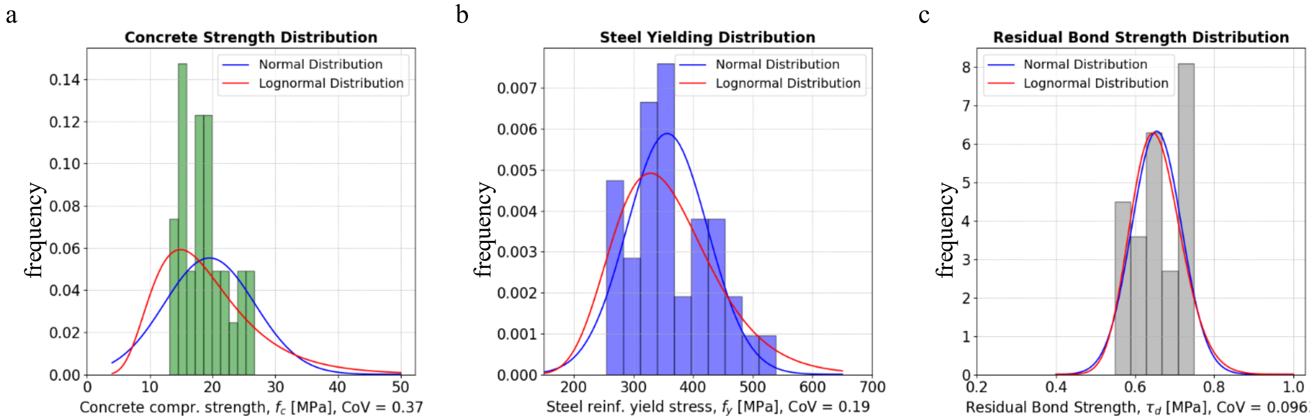

Fig. 2. Material theoretical distribution: (a) Concrete Compressive Strength, (b) Steel Yielding (c) Residual Bond Strength

Cantilevers numerical simulations are implemented with the *ForceBeamColumn* fiber element (Spacone, Filippou and Taucer, 1996) with a *HingeRadau* integration method (Scott and Fenves, 2006; Scott, 2011; Scott and Ryan, 2013). This type of fiber element considers a plastic hinge length near the two end-nodes, i and j, while the element is considered linear elastic in the central region. Non-linear stress-strain relationships are considered for the materials. In particular, *Concrete04* is assigned to the concrete fibers (Mander, Priestley and Park, 1989), and *Steel02* (Mazzoni et al. 2006) for longitudinal steel with the full-bond assumption. Whereas, in the case of bond-slips a *Multi-linear* material is assumed. The instability of longitudinal bars and the possible interaction with shear capacity are neglected in this study.

Analyses are conducted by referring to cantilevers having 1.60 m length, with two different sections: one of 30 cm x 30 cm with 4φ16, and one with a 30 cm x 50 cm having 8φ16. Each column element is analysed for different intervals of axial load ration ν (10-20%, 15-30%, 25-40%). No second-order effects due to P-Δ were is considered in all the analyses performed.

4. Results

From the analyses performed, by observing the stress-strain state of the section fibers, it is possible to determine the M-θ relationships for the different element conditions. Analyses are carried out for the two sections 30 cm x 30 cm and 30 cm x 50 cm, in Full Bond (FB) and considering the Bond-Slip (SM).

4.1. Moment-Rotation relationships

Fig. 3 illustrates, as example, the result of a single analysis obtained for the 30 cm x 50 cm section with an axial load ratio of ν=30%. The figure legend shows all the element conditions considered, that are: concrete cracking, steel compressive and tensile yielding, partial and total concrete cover failure, maximum flexural moment, concrete core failure, bars steel failure, 85% of the moment peak.

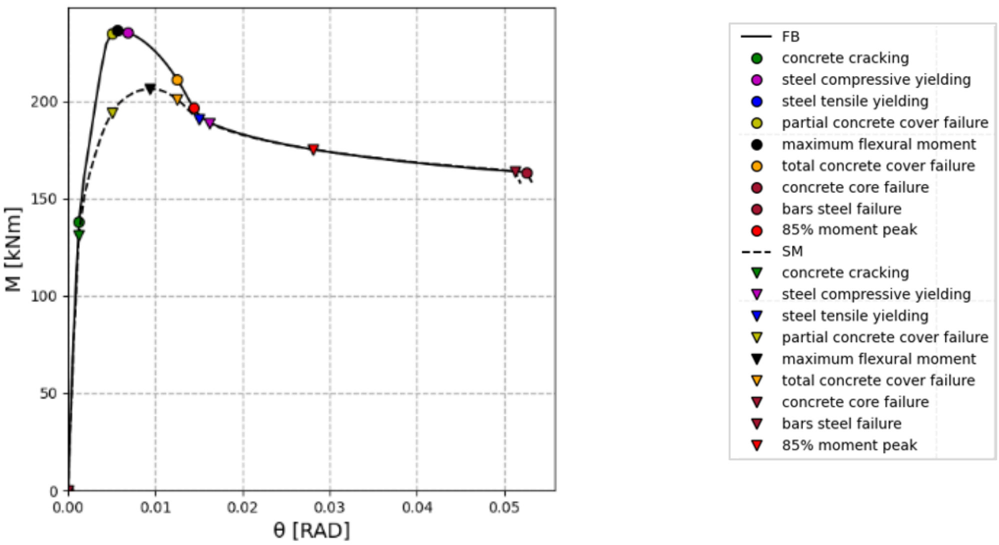

Fig. 3. Moment – Rotation section 30x50 and ν=30%

Starting from the results obtained, curves providing the probability of having a certain element condition (as defined before) may be derived. As example, Fig. 4 refers to the results obtained for the section 30 cm x 30 cm. For simplicity, the curves refer only to three element conditions, steel compressive (M_{yc}) and tensile yielding (M_{yt}), and maximum flexural moment (M_{max}). As one may note that, for a given probability, with the bond-slips a moment always lower than the one obtained with the full-bond assumption is obtained.

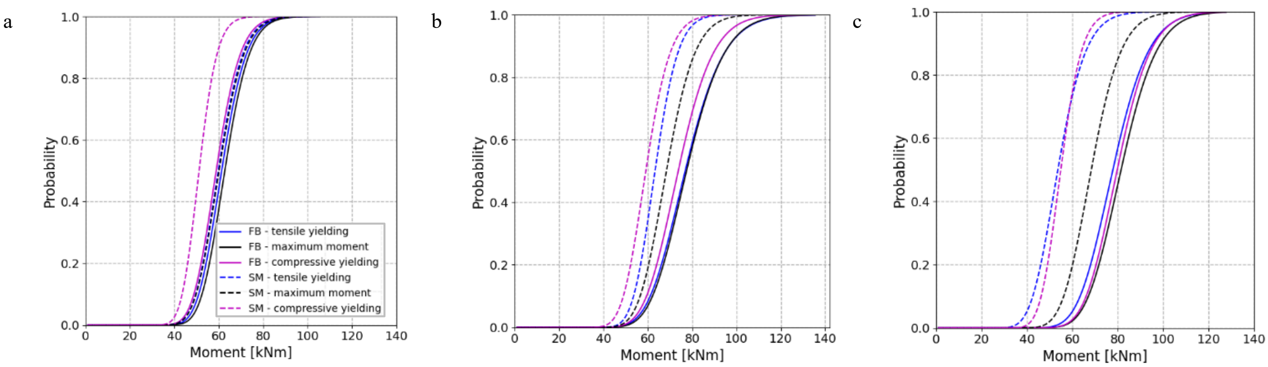

Fig. 4: Probability of having $M_{yt} \leq M^*$, or $M_{yc} \leq M^*$, or $M_{max} \leq M^*$: Section 30 cm x 30 cm (a) ν=10-20%, (b) ν=15-30%, (c) ν=25-40%

The effects of bond-slips on the Inter-story Drift Ratio (IDR) are plotted in Fig. 5. One may note that, for a given probability, the IDR corresponding to the three element conditions considered in the case of full-bond is always lower than the one with bond-slips. This highlights that the bond-slip delays the steel yielding point. In addition, within a loss analysis the curves obtained considering with bond-slip (Fig. 5) would provide a certain benefit, since the probability of obtaining a certain IDR results lower with respect the one provided with the full-bond assumption. Of course, this conclusion will be better investigated in future since a definition of damage levels with bond-slip should be clearly defined within a seismic risk framework, including also a correlation with the repair cost required. These aspects will be better investigated in future.

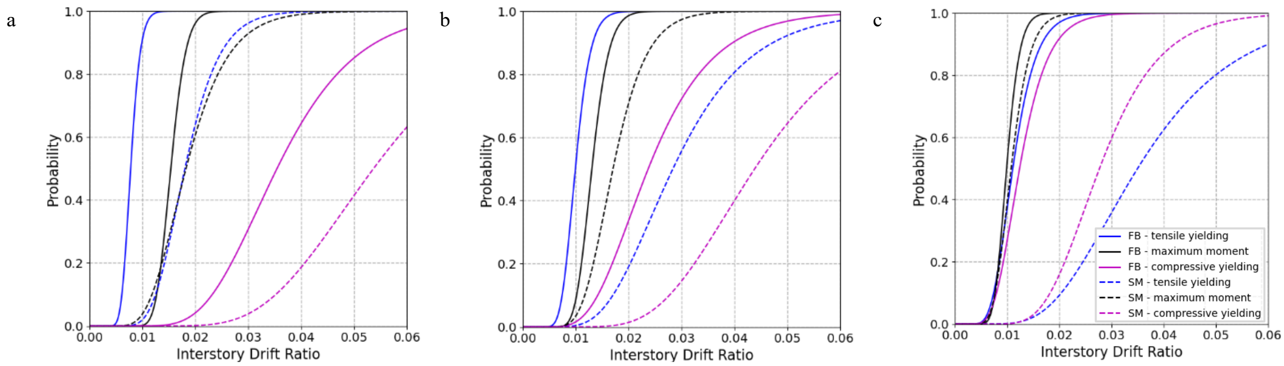

Fig. 5. Probability of having $IDR_{Myt} \leq IDR^*$, or $IDR_{Myc} \leq IDR^*$, or $IDR_{Mmax.} \leq IDR^*$: Section 30 cm x 30 cm (a) ν=10-20%, (b) ν=15-30%, (c) ν=25-40%

4.2. Rotational ductility

With the Monte Carlo simulations it is possible to evaluate as well the rotational ductility of the cantilevers investigated, with and without the bond-slip of longitudinal bars. Fig. 6 reports μ_θ by the sampling number, and for three values of axial load ratio (10%, 20% and 30%). The graphs plot the average of μ_θ, and the values $\mu_\theta \pm \sigma$, where σ is the standard deviation of the μ_θ values. One may clearly note that the bond-slip significantly reduces the rotational ductility. Also, the higher the axial load ratio the lower the μ_θ and the dispersion with respect to the average value.

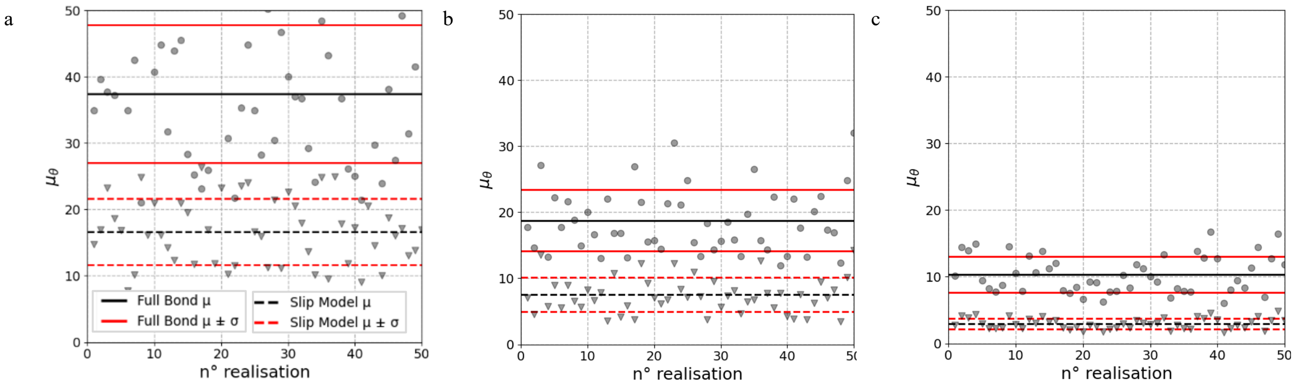

Fig. 6. Ductility Rotation μ_θ: Section 30 cm x 30 cm (a) ν=10%, (b) ν=20%, (c) ν=30%

5. Definition of damage states

In order to derive the fragility curves, it is necessary to identify the damage states for the elements where bond-slip occurs. In this study, according to the work proposed in (Cardone, 2016), three damage states (DS1, DS2, DS3) for poor-detailed columns reinforced with smooth bars are assumed, identified as a function of the concrete crack width and the concrete cover damage state.

Numerically, the damage state may be expressed through the section deformation state at the column base. In particular, as for the concrete cover damage state it is assumed that: DS1 is reached when the unconfined concrete external fiber strain ε_{c0} is measured; DS2 corresponds to the beginning of the concrete cover loss (when ε_{cu} is measured at the cover external fiber strain); and DS3 is assigned when the complete loss of the concrete cover is reached (when ε_{cu} is measured in all the cover fibers).

6. Evaluation of fragility functions

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 (Porter, 2001; Baker, 2015):

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

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:

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

where $\left(\hat{\vartheta},\hat{\beta}\right)$ correspond, respectively, to the median value and the standard deviation of the logarithm of *EDP*, allowing to have the most likely fragility curve.

Fig. 7 plots the fragility curves obtained for existing RC columns designed for vertical loads for the concrete cover damage states, considering both full bond and the bond-slip and three intervals of axial load ratio. One may note that the fragility curves may be quite comparable only in some case, demonstrating that the bond-slip may significantly affect the fragility curves referred to the concrete damage state. Anyway, the effect of the bond-slip anticipates the unconfined concrete failure.

Finally, Fig. 8 shows the comparison of the proposed fragility curves considering both full bond and bond-slip with similar ones published in the literature (Cardone, 2016), derived from a data set of experimental results carried out on RC columns/sub-assemblages designed only for vertical loads and reinforced with smooth bars. As one may observe, the results are in good agreement for all the damage states, validating the approach proposed in this study in order to derive fragility curves for existing RC columns with smooth bars.

7. Conclusions

In this work monotonic nonlinear static analyses performed on RC columns have been performed in order to proposed preliminary fragility curves to be implemented within the PBEE methodology. Columns considered are representative of those realized for buildings built in Italy before '80s, and designed for vertical loads with smooth bars. Fragility curves are derived including bond-slip phenomenon, according to the work proposed in (Braga *et al.*, 2012), and taking into account material uncertainties by means of Monte Carlo simulations.

Results obtained confirm that, according to the PBEE methodology, bond-slip has a relevant impact in the Structural Analysis, modifying the response of the elements in terms of both strength and deformability. Moreover, it is clearly illustrated that, as for the columns analyzed, bond-slips may significantly modify the fragility curves of concrete damage state with respect to the one derived with the full-bond assumption. It is observed, that the effect of the bond-slip anticipates the unconfined concrete failure.

The method proposed, preliminarily validated with comparisons with curves experimentally derived and already published in literature, may be adopted in the future for developing fragility curves of other structural components and beam-columns sub-assemblages to be implemented within the PBEE methodology, in order to perform seismic risk analysis.

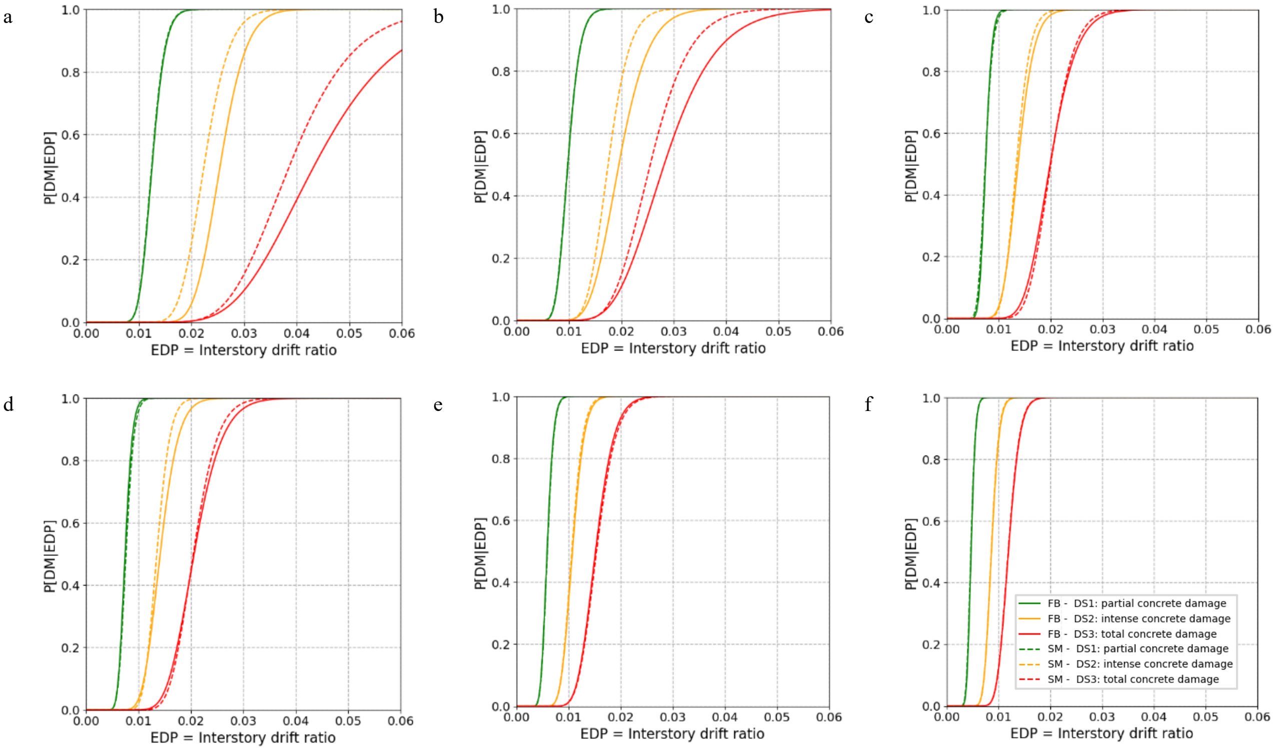

Fig. 7. Fragility functions for the damage level of concrete: Section 30 cm x 30 cm (a) ν=10-20%, (b) ν=15-30%, (c) ν=25-40%; Section 30 cm x 50 cm: (d) ν=10-20%, (e) ν=15-30%, (f) ν=25-40%

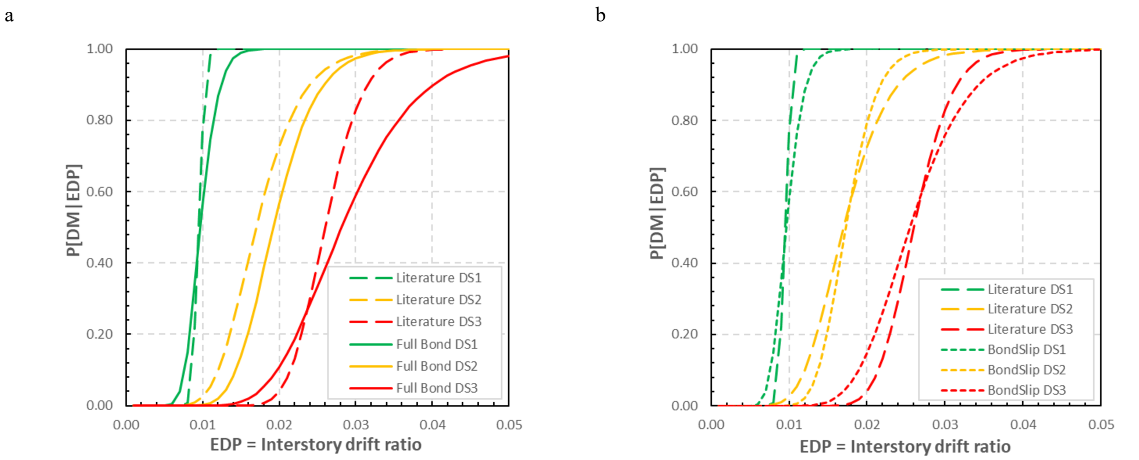

Fig. 8. Comparison of the proposed curves with similar ones published in the literature: (a) Full Bond model, (b) Bond Slip model

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