Progressive collapse fragility of European reinforced concrete buildings

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ABSTRACT: Structural safety is generally assessed without consideration of abnormal load conditions that may give rise to global system collapse after local failure in one or a few components. Particularly in the case of high-risk structures, Eurocode 1 recommends a systematic risk assessment of the structure, considering either identified threats or unspecified damaging events. Nonetheless, a comprehensive probabilistic assessment of European structures is strongly needed. In such a context, this paper presents the outcomes of fragility analyses performed on reinforced concrete framed buildings, proposing a set of fragility models that can be used for probabilistic assessment and management of the risk of progressive collapse. Gravity-load designed and earthquake-resistant building structures were considered and respectively designed in accordance with Eurocodes 2 and 8. Fiber-based finite element models were developed and analyzed under sudden removal of one or more columns, allowing structural performance and damage propagation to be evaluated. Based upon statistics and probability distribution functions for material properties, geometry, and design loads of the building class under study, a Monte Carlo simulation was performed to generate both 2D and 3D models. Structural performance was assessed by incremental-mass nonlinear dynamic analysis, capturing the attainment of limit states either at sectional or global levels. Probability distribution functions were then fitted to fragility points in order to provide fragility functions at multiple damage states for their use in progressive collapse risk assessment. The analysis results show the significant impact of seismic design rules and secondary beams on progressive collapse fragility.

1. INTRODUCTION AND RESEARCH METHODOLOGY

Conventional procedures for structural design and assessment do not consider safety against abnormal load conditions produced by, for instance, impact, explosion or human error. Abnormal loads on structures are caused by extreme events, which have a low probability of occurrence and high potential of huge losses. Abnormal load conditions can induce a disproportionate collapse, which is a particular type of progressive collapse consisting in a catastrophic propagation of damage from a localized portion of the structure to the whole structural system or a significant part of it (Adam et al., 2018). Assessing structural safety in such conditions requires computationally expensive simulations of the extreme structural response. This has typically motivated a deterministic nature of building codes (e.g.: GSA, 2013; DoD, 2013), research and engineering practice in the field (e.g.: Bao et al., 2008; Tsai and Lin, 2008; Shi et al., 2010; Parisi and Augenti, 2012; Brunesi and Nascimbene, 2014). By contrast, extreme loads and structural capacity are affected by large uncertainties that should be quantified and controlled for an effective risk management,

1

particularly in the case of critical infrastructures. This has recently produced the beginning of probabilistic research on building structures subjected to blast loads or sudden column loss (e.g.: Parisi, 2015; Li et al., 2016). In Europe, this new research trend is in line with EC1 (2006) that, particularly in the case of Class 3 structures, recommends a systematic risk assessment, considering either identified threats or unspecified damaging events. Therefore, a comprehensive probabilistic assessment of European structures is not only advisable but also strongly needed. To this end, fragility models may be used to predict the conditional probability of progressive collapse given that local damage has occurred in a portion of the structure.

In this paper, a framework for progressive collapse vulnerability assessment of European reinforced concrete (RC) framed buildings is presented. The analytical procedure makes use of fragility analysis proposed in earthquake engineering (Porter et al., 2007) and progressive collapse simulation techniques (Brunesi and Nascimbene, 2014). The flowchart shown in Figure 1 outlines the main modules of a probabilistic mechanics-based procedure used for analytical derivation of fragility curves that define the conditional probability of exceeding a damage level given the gravity load intensity.

After that capacity models and nonlinear analysis methods were identified as discussed below, the generation of the building population implied modeling of random variables (RVs) in terms of material properties, geometrical parameters, and design loads. Based on statistics and probability distributions for each RV, several thousands of structural models for each building class were randomly generated via Monte Carlo simulation. Simulated design, numerical modeling and damage analysis of every structural model were carried out to assess the potential of progressive collapse. Two design rules, that is, according to EC2 (2004) and EC8 (2004), and two structural representations (i.e. 2D and 3D models) were considered, resulting in four types of structural and fragility models. While the

definition of both column loss scenario and reference intensity measure (IM) is required to perform nonlinear analysis, the selection of criteria for the identification of damage states is a crucial aspect for structural performance assessment. After that demand was compared with capacity, the distribution of buildings in each damage state was used to derive the statistical parameters – i.e. mean (μ), standard deviation (σ) and coefficient of determination (R^2) – of each lognormal fragility function, fitting cumulative fragility points by means of nonlinear regression analysis.
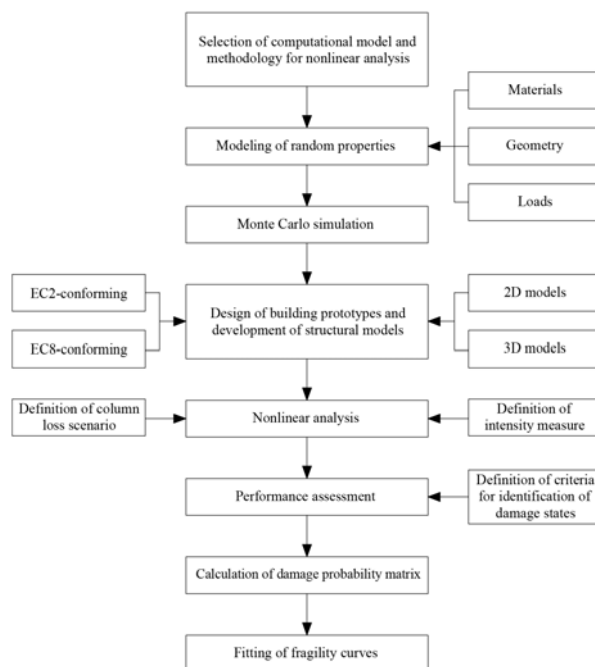

Figure 1: Flowchart of progressive collapse fragility analysis.

More details on capacity modeling and fragility analysis are provided in the following sections.

2. FRAGILITY ANALYSIS

The analytical procedure used for derivation of progressive collapse fragility models was separately applied to each building class and capacity model selected in this study. After that performance criteria associated with increasing levels of damage were defined, different

thresholds for key performance indicators (KPIs) were assumed, depending on the building class under analysis. Based on the selected damage level, either the vertical beam drift (global level) or the concrete/steel strain (local level) was assumed as KPI. The residual structure, i.e. the structure without a column, was randomly generated and its nonlinear performance was evaluated through incremental dynamic analysis (IDA). Hence, the attainment of KPI thresholds was recorded during IDA, allowing the failure probability to be estimated as discussed below.

2.1. Case-study building classes and uncertainty modeling

The following populations of low-rise, RC framed buildings were considered:

- EC2-conforming building class (i.e. gravity-load designed buildings);
- EC8-conforming building class (i.e. earthquake-resistant buildings).

Two types of structural models were randomly generated and analyzed in a probabilistic fashion, namely 2D and 3D models, for each building class under study. As a result, a total amount of four fragility models corresponding to as many combinations in terms of design code regulations and structural modeling strategies were developed.

Uncertainty modeling focused on the structural geometry, material properties and loads. According to past studies (see e.g. Borzi et al., 2008), the span length in the longitudinal and transverse directions, here denoted as L_x and L_y, were assumed as RVs with uniform distribution in the range [4.0 m, 6.0 m]. On the other hand, the interstory height was considered as a deterministic parameter equal to 3.0 m, as it does not significantly influence the progressive collapse resistance. Material uncertainties were taken into account by randomly selecting steel yield strengths of 380 MPa and 450 MPa and cubic concrete strengths of 25 MPa, 30 MPa and 35 MPa, each of them assumed to have the same probability of occurrence (Borzi et al., 2008). Noteworthy is that the above values were

considered as nominal strengths, which were then multiplied by a normally distributed (dimensionless) RV with mean equal to 1.1 in case of reinforcing steel and 1.5 in case of concrete, and coefficient of variation (CoV) equal to 10%. A normal distribution was assigned to the dead load with mean of 3.0 kN/m^2 and CoV = 17%, whereas the live load was considered as a deterministic parameter equal to 2.0 kN/m^2 because it has negligible impact on the overall downward load for progressive collapse analysis (DoD, 2013).

2.2. Simulated design and nonlinear capacity modeling

The buildings under study are 4-story, 4 × 4-bay RC framed structures, which are composed of five primary frames in the *x*-direction. In the perpendicular direction of the building plan, primary frames were connected each other by one-way RC joist slabs and continuous, cast-in-situ secondary beams. Partition walls, infill walls and floor slabs were modeled as loads, so their contribution to the progressive collapse capacity was not considered. That was a conservative assumption because those elements may improve structural response when properly detailed, contributing to the redistribution capacity of the entire building. In addition, all framed systems were assumed to be rigidly fixed to the ground.

The design compressive strength of concrete and design tensile strength of reinforcing steel were defined as $f_{cd} = 0.85f_{ck}/1.5$ and $f_{yd} = f_{yk}/1.15$, respectively, where f_{ck} is the characteristic (cylindrical) compressive strength of concrete and f_{yk} is the characteristic yield strength of steel. The mean compressive strength of concrete and mean yield strength of steel were assumed to be $f_{cm} = f_{ck} + 8$ and $f_{ym} = 1.1f_{yk}$, respectively. The simulated design process was performed by assuming cross section dimensions of beams and columns to be constant and equal over all case-study structures. In detail, column and beam sections were assumed to be 400 × 400 mm^2 and 300 × 500 mm^2 in size, respectively. By contrast, the amount of steel reinforcement was derived from a direct application of EC2 (2004) and EC8

(2004) provisions. The simulated design for earthquake resistance was undertaken by assuming a construction site with medium–high seismicity level, type C ground, and medium ductility class. Hence, peak ground acceleration at bedrock was set to 0.30g for life safety limit state, where g is the gravitational acceleration. Seismic design was based on response spectrum analysis of 3D structural models, incorporating accidental eccentricity of the center of mass.

Figure 2 shows isometric and plan views of a representative building model, providing also key details on modeling scheme and column removal strategy for progressive collapse analysis.
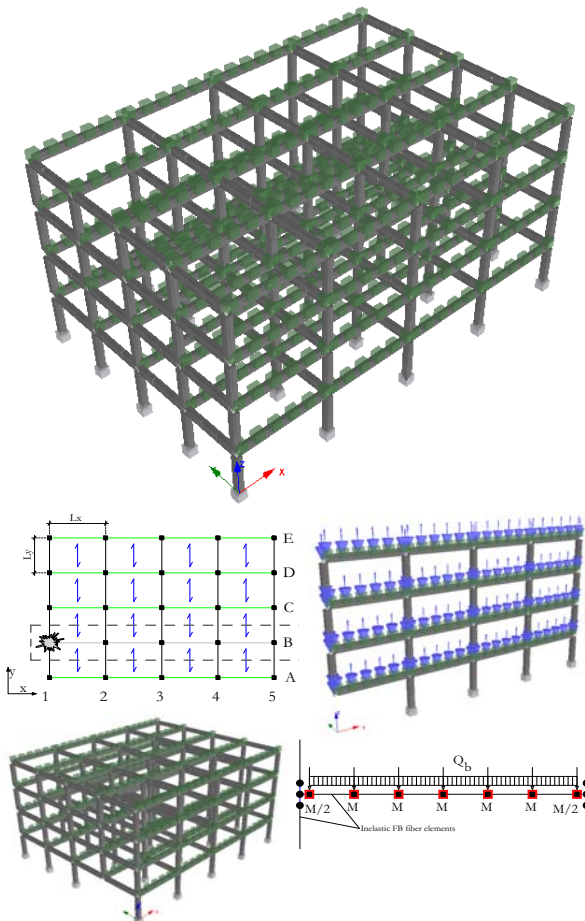

Figure 2: Isometric and plan views of a case-study building model.

Nonlinear capacity modeling of structures representative of the selected building classes followed a spread plasticity approach in view of

incremental-mass nonlinear dynamic analysis. In particular, the finite element code SeismoStruct rel. 6 (2016) was used to develop capacity models and to carry out IDA, explicitly including material and geometric nonlinearities. In addition to 3D models, internal 2D frames were extracted and analyzed after removal of an external ground column. Potential large displacements/rotations and P-Delta effects were taken into account using a total corotational transformation. The spreading of inelasticity over the member length and cross section was reproduced through a direct integration of the uniaxial material response of individual fibers, which provided sectional stresses and strains at key positions of the frame member. Each inelastic beam-column element had 5 integration points and each cross section was discretized in 400 fibers to accurately represent its stress/strain state during dynamic loading and to capture damage localization in critical beam sections. A simple bilinear constitutive model with isotropic strain hardening was assigned to reinforcing steel. The uniaxial uniform confinement model proposed by Mander et al. (1988) was used to simulate the cyclic behavior of concrete, explicitly accounting for tension softening.

2.3. Damage states definition and derivation of fragility functions

To compare demand with capacity under progressive collapse condition, the following three criteria were considered – either at structural (i.e. vertical drifts) or sectional (i.e. concrete and steel strains) levels – for limit states definition: slight (LS1), significant (LS2) and extensive (LS3) damage. LS1 refers to a situation in which the building can be immediately used after an event with minor repair or strengthening only, so it can be regarded as a sort of serviceability limit state under extreme load conditions (important for resilience). Conversely, LS3 can be considered as a performance level beyond which the structure is no longer able to sustain any further increment of gravity loads nor the gravity loads for which it was designed, requiring significant repair. LS2 can be referred to as an intermediate performance

level beyond which the building becomes unsafe for its occupants, because nonlinearity sources are activated. Therefore, it may be identified as a sort of life safety limit state beyond which the structure should be evacuated and moderately repaired in critical portions of beams.

Furthermore, to evaluate whether an element of the skeletal frame reaches or exceeds a limit condition, strains and drifts experienced in the critical portion of the structure where the progressive collapse mechanism is imposed to occur were compared with conventionally identified limit capacities:

- LS1 was defined by steel and concrete strains for both EC2- and EC8-conforming building classes. For each randomly generated structure, the yield strain of steel bars was calculated as the ratio between yield strength and Young's modulus, whereas concrete strain at peak strength was computed according to Mander et al. (1988).
- LS2 was supposed to occur when the vertical drift, obtained as the ratio between the peak displacement above the removed column and the beam span length, exceeds a deterministic threshold. The vertical drift threshold was set to 0.5% and 1.0% in the case of EC2-conforming and EC8-conforming building classes, respectively.
- LS3 was characterized in terms of ultimate steel strain and ultimate concrete strain. For each structure, the ultimate concrete strain was calculated according to different material properties and reinforcement arrangement, whereas the ultimate strain of steel rebar was set to 4% and 6% in the case of EC2-conforming and EC8-conforming building classes, respectively.

As shown in Figure 3, where the routine for derivation of fragility models is presented, the downward load on beams (in progressive collapse combination), which is here denoted as Q_b, was assumed to be the IM.
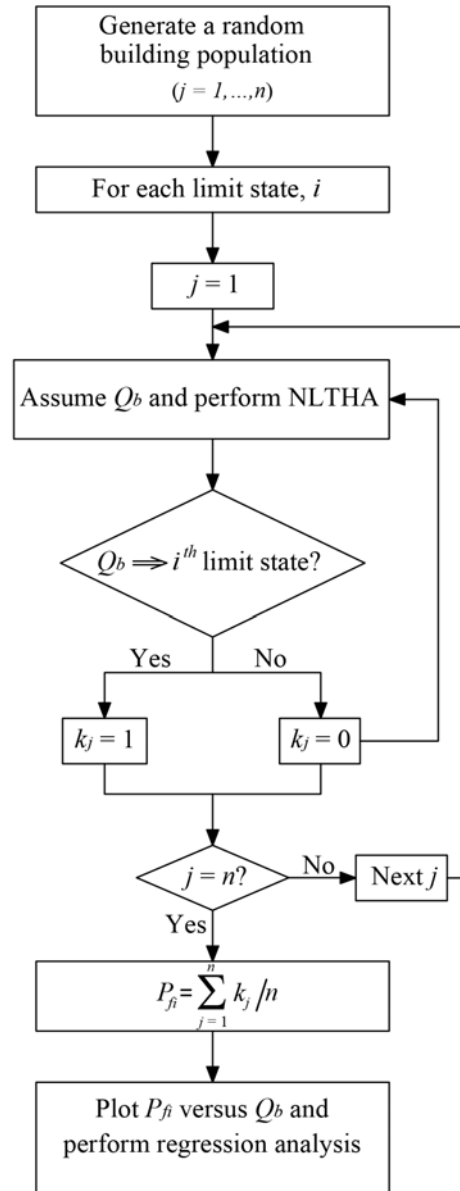

Figure 3: Routine for derivation of fragility curves.

Therefore, a multiplier of Q_b was identified on each IDA curve for each damage state, so a damage probability matrix (DPM) was assembled. The DPM included the fractions of sampled structures in each damage state, for a set of increasing Q_b levels. The cumulative fraction of buildings in each damage state was then computed, summing up the percentages of frames pertaining to each of them. Lastly, a lognormal cumulative distribution function was fitted to fragility points through regression analysis, thus

providing the probability of exceeding each damage state in a continuous fashion.

Fragility functions for the selected building classes and structural models are shown in Figure 4 and Figure 5. The parameters and the coefficient of determination of fragility functions are outlined in Table 1, Table 2 and Table 3.

A good match with fragility points from incremental-mass nonlinear dynamic analyses can be observed for each limit state (i.e. LS1, LS2 and LS3), design approach (i.e. EC2- and EC8-conforming structures) and structural idealization (i.e. 2D and 3D models), with R^2 ranging from 0.839 to 0.999.
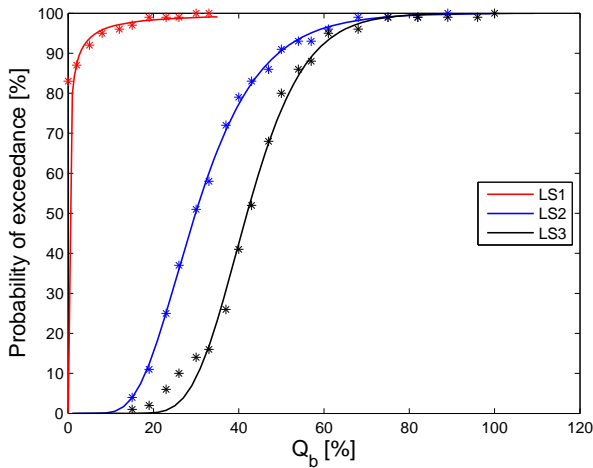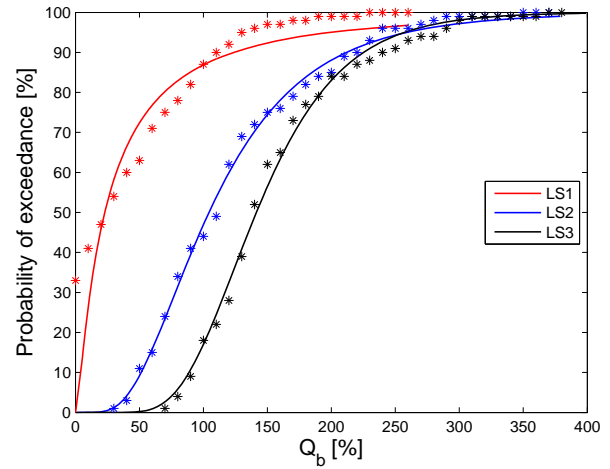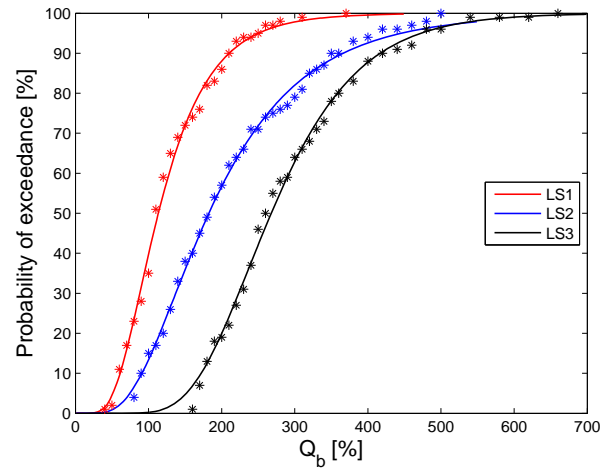

(a)
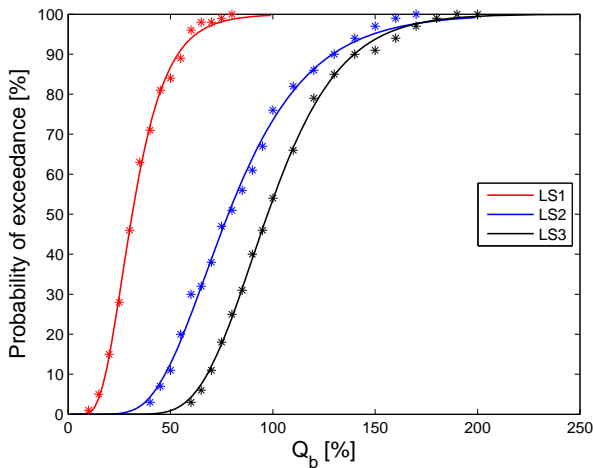
(b)

Figure 4: Fragility curves of 2D models: (a) EC2-conforming and (b) EC8-conforming building class.

(a)

(b)

Figure 5: Fragility curves of 3D models: (a) EC2-conforming and (b) EC8-conforming building class.

As can be inferred from Table 1, Table 2 and Table 3, the scatter is very low in case of EC8-conforming buildings, given that their fragility models are characterized by a coefficient of determination close to unity ($0.995 < R^2 < 0.999$) for both 2D and 3D models. As far as the EC2-conforming building class is concerned, an identical accuracy is observed for limit states LS2 and LS3, considering both 2D and 3D models, while the goodness of fit for limit state LS1 was slightly lower as R^2 equals 0.955 and 0.839 for 2D and 3D models, respectively. Nevertheless, unless a critical structure is considered, LS1 is less crucial than other limit states for progressive collapse applications, in which (i) moderate

damage is likely to occur as a consequence of redistribution of vertical loads from a removed column and (ii) an optimum design target is life safety limit state or the collapse prevention limit state, in light of performance-based design principles.

Table 1: μ, σ and R^2 of fragility functions for limit state LS1, in terms of Q_b.

Building class	Model	LS1		
		μ	σ	R^2
EC2-conforming	2D	-6.548	2.323	0.955
	3D	-1.473	1.314	0.839
EC8-conforming	2D	-1.162	0.420	0.998
	3D	0.126	0.482	0.995

Table 2: μ, σ and R^2 of fragility functions for limit state LS2, in terms of Q_b.

Building class	Model	LS2		
		μ	σ	R^2
EC2-conforming	2D	-1.208	0.391	0.999
	3D	0.043	0.552	0.996
EC8-conforming	2D	-0.246	0.391	0.996
	3D	0.601	0.543	0.997

Table 3: μ, σ and R^2 of fragility functions for limit state LS3, in terms of Q_b.

Building class	Model	LS3		
		μ	σ	R^2
EC2-conforming	2D	-0.868	0.249	0.995
	3D	0.347	0.3631	0.995
EC8-conforming	2D	-0.026	0.281	0.999
	3D	0.985	0.345	0.996

Moreover, it is noteworthy that the fragility models proposed in this paper affirm the key role played by secondary framing beams and seismic design/detailing criteria, which can be embraced and implemented for a cost-effective mitigation of progressive collapse vulnerability of RC framed buildings. In fact, the above results lay evident that (i) a symmetrical reinforcement configuration is effective for developing a rationally controlled

resisting mechanism, whereas (ii) the redundancy added by the secondary beam systems is crucial to ensure alternative load paths under extreme load conditions due to sudden column losses. In this respect, additional robustness resources provided by secondary frame systems may be predicted at the design stage, for instance through a correlation between the maximum vertical displacement of the 3D structural model, normalized to that of the 2D model, and the maximum demand-to-capacity ratio in the beams (Brunesi and Nascimbene, 2014).

3. CONCLUSIONS

In this study, a procedure for large displacement inelastic dynamic simulation of RC framed buildings subjected to threat-independent sudden column loss was integrated in a probabilistic framework for fragility analysis of this structural typology, leading to the derivation of fragility functions. Two low-rise RC building classes were investigated, namely gravity-load-resistant RC buildings designed in compliance with EC2 and earthquake-resistant RC buildings designed in compliance with EC8. The performance of the selected structures was evaluated through incremental-mass nonlinear time-history analysis, using two modeling strategies (i.e. 2D and 3D fiber-based models). A set of fragility models was thus derived accordingly, allowing the following conclusions to be drawn:

- The fragility models proposed in this study are optimally fitted to discrete fragility estimates provided by IDA combined with Monte Carlo simulation, particularly for life safety and collapse prevention limit states, which are crucial for progressive collapse assessment.
- Different fragility levels are associated with multiple damage states, depending on the building class and modeling strategy used for probabilistic progressive collapse resistance assessment.
- Significant benefits from both seismic design and secondary beams on the actual robustness level of RC buildings are confirmed and

quantified by the fragility functions proposed herein.

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