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PROGRESSIVE COLLAPSE FRAGILITY MODELS OF RC FRAMED BUILDINGS BASED ON PUSHDOWN ANALYSIS

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Abstract. *Partial or total progressive collapse under abnormal loading conditions (e.g. deliberate terrorist attacks, uncontrolled gas releases, and vehicle or aircraft impacts) is one of the most vivid examples of low probability-high consequence (LPHC) event that may occur in the lifetime of a structure. Despite this, structural safety for extreme loads that may lead to disproportionate (or progressive) collapse has been probabilistically assessed and controlled in a few cases, thus neglecting uncertainties in loads and system capacity. As such, this paper moves from a deterministic to a probabilistic framework, proposing fragility models at multiple damage states for low-rise reinforced concrete (RC) framed bare structures which may be applied for progressive collapse risk assessment and management. Two building classes representative of structures designed for either gravity loads or earthquake resistance in accordance with current European rules were investigated. Monte Carlo (MC) simulation was used to generate random realizations of two-dimensional (2D) and three-dimensional (3D) structural models. Their fiber-based finite element (FE) representations were developed within an open source platform for nonlinear static pushdown analysis. The output consisted of fragility functions for each damage state of interest. Such fragility models were then compared to those derived through incremental dynamic analysis (IDA) in a previous study. IDA-based and pushdown-based capacities were additionally used to propose regression models for quick estimation of dynamic amplification factor (DAF) at a given displacement/drift target. The analysis results show a significant influence of both seismic design/detailing and secondary beams on robustness of the case-study building classes.*

1 INTRODUCTION

Accidental and man-made extreme events such as impacts, fires or explosions can induce abnormal loads on building structures, which in turn may suffer heavy damage and finally experience a partial or total building collapse. This depends on the fact that structural systems designed according to conventional approaches are not necessarily able to withstand extreme loads. To confirm this scenario, several residential, iconic and public buildings resulted in significant casualties and property loss as a consequence of a disproportionately high propagation of direct damage to their key components [1-5].

Following the early interest in blast- and progressive collapse-resistant building analysis and design, which was triggered by the 1968 partial collapse of Ronan Point tower in UK, the occurrence of further dramatic accidents and deliberate attacks either in urban or industrial environments has led homeland security to become a primary concern for public authorities and stakeholders, causing the protection of structures against extreme loads to have a larger and larger impact on economy and society. Several definitions of progressive collapse, the most known of which is based on the significant disproportion in size between the initial and final damage configurations, have appeared in the literature [6-12]. Acting on building exposure, the contribution of nonstructural protective measures such as barriers, sacrificial elements and limitation or control of public access have been reaffirmed to be crucial in order to increase structural safety and to mitigate progressive collapse-related risk in a cost-effective manner. Moving from passive to active strengthening strategies, extensive efforts have been more recently made in design and simulation [13-35] to propose approaches and rules for structural integrity and robustness against abnormal loads. Codified methodologies [13-16] have emerged in the last two decades providing design guidelines to inhibit cascade or domino effects as a result of a local failure, which progresses in time to a collapse encompassing the entire structure or essential parts of it. Focusing on performance of structures and structural components, the key role played by requirements such as robustness, integrity, continuity, redundancy and ductility has been recognized [17-19]. Besides direct and indirect design provisions prescribed in codes and standards [13-16], a huge amount of analytical studies on blast and progressive collapse phenomena have been done [11, 12, 17-35], thus quantifying the influence of modeling assumptions and analysis techniques. Solid or shell FE [25, 28, 33, 34], lumped-plasticity [18, 22-24, 26, 27] and spread-plasticity [11, 12, 17, 19, 25, 29-32, 35] approaches have been examined for progressive collapse analysis of different building typologies ranging from civilian to strategic and military facilities. In this respect, conventional or alternative pushdown procedures and implicit or explicit nonlinear dynamic analyses have been explored using general purpose codes or open platforms.

Despite the large number of deterministic investigations, probabilistic approaches have been applied to a lesser extent in this field and a few research works have appeared in the literature so far (see e.g. [1, 6, 36-41]), emphasizing the need for probabilistic risk assessment and management of disproportionate collapse under blast loads and/or sudden column loss scenarios. In light of this, a framework for fragility analysis of European RC framed buildings was implemented, integrating fiber-based FE modeling with Monte Carlo simulation for random realizations of structural prototypes and pushdown analysis techniques for progressive collapse assessment. Two 4-story building populations, each of which analyzed using two types of numerical representation (i.e. 2D models and 3D models), were studied in a probabilistic fashion in order to propose fragility models at multiple damage states. The former class was designed only for gravity loads according to Eurocode 2 (EC2) [42], and the latter was designed for earthquake resistance in compliance with Eurocode 8 (EC8) [43]. For each building class under consideration (i.e. EC2-compliant structures and EC8-compliant structures), fragility analysis was based on (i) the random generation of 2D and 3D fiber-based models through MC simulation, (ii) pushdown analysis of each structural model including dynamic effects in a simplified manner [15], and (iii) derivation of fragility functions for each structural model and damage state of interest. Considering design criteria and structural idealizations, four groups of fragility models were presented, thus quantifying the sensitivity of progressive collapse to design rules (i.e. seismic or non-seismic) and redistribution capacity of secondary beam systems. A comparison was then provided between such pushdown-based fragility models and those obtained by Brunesi et al. [40] using incremental-mass nonlinear dynamic analysis. Finally, the influence of dynamic effects on progressive collapse potential was estimated by taking advantage of IDA-based predictions [40] and pushdown capacity curves, which were determined without introducing any aprioristic amplification factor for specific structural portions. Pushdown and IDA estimates resulting from the application of the same uniform mass/load distribution over the entire structure were normalized in order to construct a set of DAF-vertical displacement/drift curves and to fit a corresponding regression model for each building class and structural representation under study, thus permitting a quick analytical assessment of DAF at a given displacement/drift design target.

2 FRAGILITY ANALYSIS: METHODOLOGY AND PROCEDURES

In a general framework for probabilistic risk analysis (PRA) using either hazard- or scenario-based approaches, the annual probability of disproportionate collapse can obtained through the conditional probabilities of two limit states: (1) local damage given an extreme event, and (2) global (structural) collapse given a local damage [6]. These two conditional probabilities can be in turn quantified using a multilevel analysis where uncertainties related to abnormal loading and structural system are modeled and propagated. The probability of damage to structural components and systems can be then obtained by means of reliability computations in which demand is convolved with capacity [44]. As such, PRA is a quantitative and rational tool for an effective risk-informed decision making process in case of LPHC events and fragility analysis can be considered as one of its fundamental components, which is indeed used in conventional risk management techniques to investigate and describe the physical vulnerability of a population of buildings or structural assemblies to specific damage scenarios. Although such concepts currently pertain to a well-established approach for disaster mitigation in earthquake engineering (see e.g. [45-50]), only a few applications have been recently presented for disproportionate collapse under blast loads and/or sudden column loss scenarios [36-41].

Several methodologies may be proposed to provide a mathematical formulation to vulnerability, either in terms of damage probability matrices or vulnerability curves, and there cannot be a unanimous consensus on a unique approach for derivation of vulnerability models to be implemented in PRA, due to a variety of structural modeling options, analysis techniques, and damage criteria. Even though any model with its level of sophistication presents peculiar advantages and drawbacks, the development of fragility models is recognized to be a key aspect for disaster mitigation actions, as those functions characterize the conditional probability of exceeding a certain damage level for a prescribed load intensity. The conditional probability may be defined as follows:

$$P_{ik} = P\left[D \geq d_i \,\middle|\, S = s_k\right] \tag{1}$$

where D is the damage measure (DM) used to describe structural or nonstructural damage, S is the scalar or vector-valued intensity measure (IM) used to describe the load intensity, and P_{ik} is the probability that the damage level d_i is reached or exceeded for a given IM level s_k.

Therefore, vulnerability to progressive collapse may be regarded as a measure of how prone a building is to damage for a given severity of the selected column removal scenario and the corresponding fragility models at multiple damage states may be regarded as a graphical representation of vulnerability to this type of mechanism, providing estimates of the proportion of a building class falling within discrete damage bands from a specified IM.

In light of the aforementioned considerations, this study presents a probabilistic mechanics-based assessment procedure for fragility analysis of structures subjected to disproportionate collapse. Analytical fragility curves were derived in case of gravity-load designed [42] and earthquake resistant [43] RC framed buildings. As discussed in the following, concepts of methodologies developed for earthquake engineering applications [45-50] and recently applied in this field [39, 40] were integrated with specific analysis techniques for progressive collapse simulation. A flowchart presenting the proposed routine and their prevailing components is shown in Fig. 1, while the rationale behind this framework is summarized hereafter.

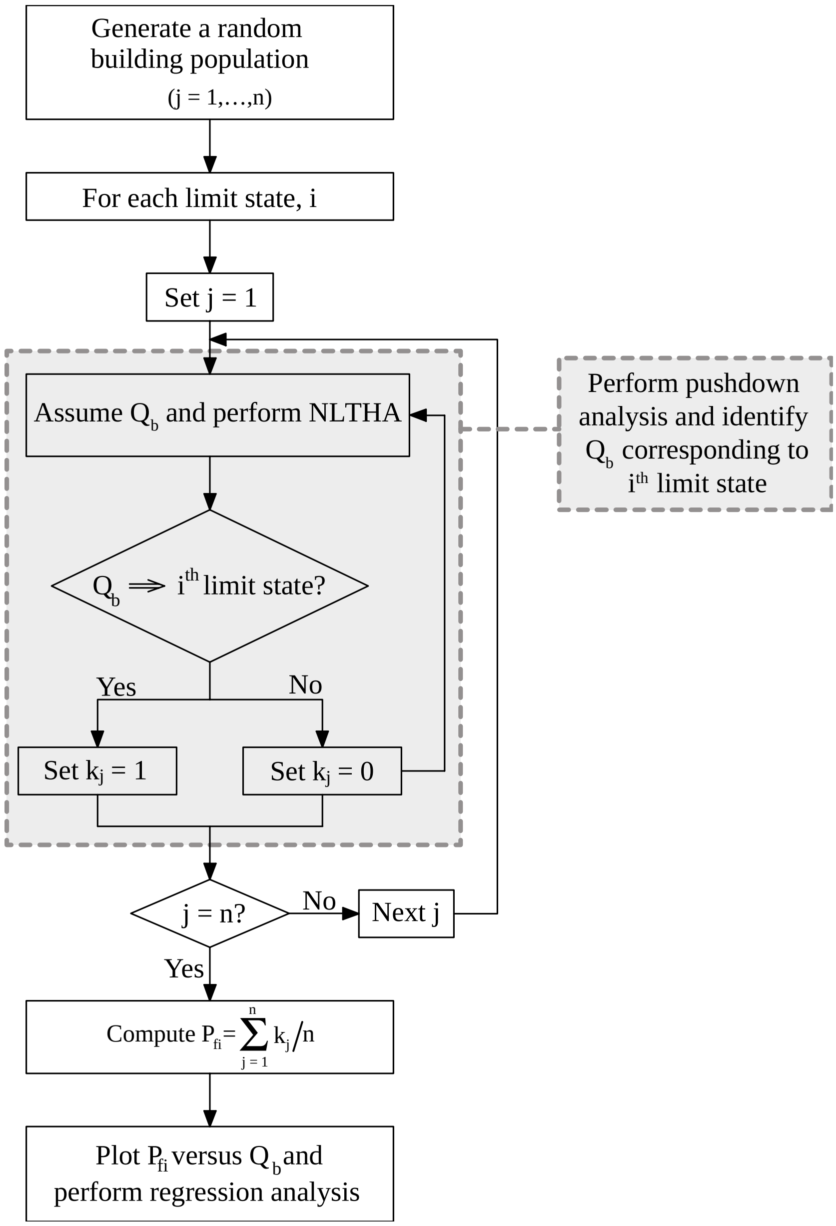

Figure 1: Flowchart for derivation of progressive collapse fragility models.

According to Brunesi et al. [40], fragility was estimated by means of a MC simulation consisting of the following three steps:

1) Generation of the building population which implied modeling of random variables (RVs) in compliance with statistics and probability distribution functions for material properties, geometrical parameters and design loads.

2) Damage analysis of each random realization which implied identifying simulated design criteria and numerical modeling strategies to predict progressive collapse potential and assess whether a prescribed amount of damage at a given IM level is reached or not.

3) Analytical derivation of progressive collapse fragility which implied adopting procedures for nonlinear regression analysis in order to fit cumulative fragility points, after that capacity and demand were convolved.

In this latter step, the distribution of buildings in each damage state of interest 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 discrete points from damage probability matrices through the least squares technique. To evaluate progressive collapse fragility, several thousands of structural prototypes for each building class were randomly generated on the basis of statistics and probability distributions for each RV, as specified in Section 3, and their capacity was estimated by means of classical fiber force-based [51] FE approaches commonly adopted in earthquake [46, 48, 50, 52] and progressive collapse analysis [11, 12, 17, 19, 25, 29-32, 35, 40]. To this aim, pushdown simulation techniques [11, 17, 18, 23, 24, 31] rather than IDA procedures [19, 26, 40] were implemented and the downward load on beams (Q_b) was assumed as reference IM. Three criteria, either at local/sectional or global/structural levels, were selected for the identification of limit states corresponding to slight damage (LS1), significant damage (LS2) and complete damage (LS3), thus allowing the evaluation of structural performance. Based on capacity/demand convolution, fragility models for EC2-compliant [42] and EC8-compliant [43] buildings analyzed using either 2D or 3D prototypes were finally computed, allowing a two-way comparison between functions obtained (i) by different design solutions for the same type of structural representation (i.e. EC2-conforming versus EC8-conforming), and (ii) by different modeling types for the same design rules (i.e. 2D versus 3D).

3 UNCERTAINTY MODELING AND COMPUTATIONAL STRATEGY

Pushdown analysis was implemented in a fragility analysis framework to provide the probability of exceedance of different damage states induced by column loss events. In this study, two design rules (i.e. according to EC2 [42] and EC8 [43]) and two structural idealizations (i.e. 2D and 3D models) were considered, resulting in four types of structural models each of them characterized by different kinds of uncertainty. Thousands of samples of RC framed structures were randomly generated for each of the two building populations (i.e. gravity-load and earthquake resistant prototypes). Uncertainties in geometry, material properties and loads were modeled according to Brunesi et al. [40], as a further aim of this paper is to compare IDA-based and pushdown-based fragility models, thus quantifying the mismatch between the two analysis methods. As discussed in Section 6, corrective coefficients were proposed for the statistical parameters (i.e. μ and σ) of each lognormal fragility function derived from the latter approach in direct relation to those computed through nonlinear dynamic simulations. As such, every simulation realization analyzed herein was assumed to be equal to the corresponding random idealization studied in the comparative research [40]. A summary of selected RVs with their peculiar probabilistic features was provided in the following, while more comprehensive details regarding the statistical background of MC sampling can be found in [40]. A similar consideration can be drawn in terms of FE idealization of each case-study of the two building classes, as specified in the upcoming discussion.

As shown in Fig. 2, the structures under study were 4-story, 4×4-bay RC framed buildings, which had a lateral force-resisting system (LFRS) composed of five primary frames connected each other by one-way RC joist slabs and continuous cast-in-situ secondary beams. Following typical layouts and building practice, the span lengths in both longitudinal and transverse directions, namely L_x and L_y, were assumed as RVs with uniform distribution in the range [4.0 m, 6.0 m]. A similar criterion was applied in past studies for random sampling associated with seismic vulnerability analysis of RC structures (see e.g. [45, 46, 48, 50]). The center-to-center plan dimensions were equal at any floor, but varied in length (*x*-direction) and width (*y*-direction) according to different random combinations of column spacing in the two directions. As progressive collapse resistance was found to be almost insensitive to changes in the interstory height [19], the latter was considered as a deterministic parameter equal to 3.0 m. Material uncertainties were taken into account by randomly selecting steel yielding 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 [48]. Those 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 with coefficient of variation (CoV) equal to 10%. Finally, a normal distribution was assigned to dead load (DL) with mean of 3.0 kN/m^2 and CoV = 17%, whereas live load (LL) was considered as a deterministic parameter equal to 2.0 kN/m^2 because the case-study buildings were assumed to be employed for housing. In order to permit a representation of the sample in the low probability region of the distribution, the Latin Hypercube algorithm was adopted for MC sampling of RVs with normal distribution [40, 48].

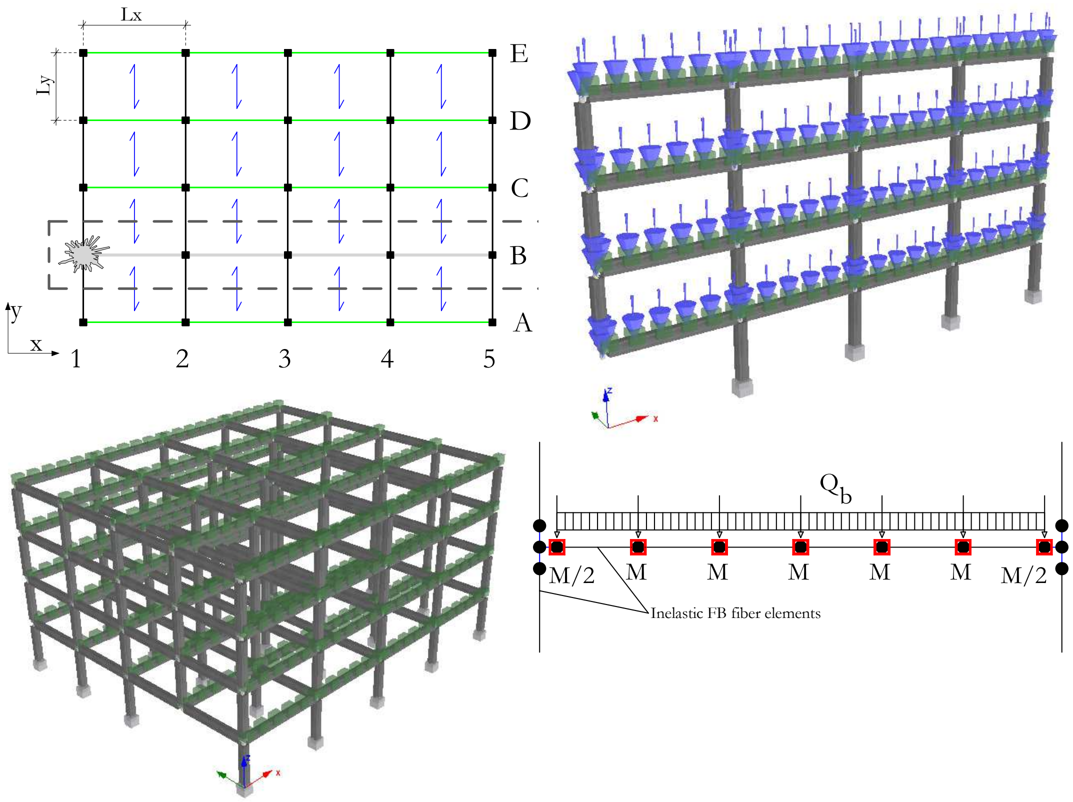

Figure 2: Example of a representative structural prototype.

Fig. 2 shows isometric and plan views of a representative building model used for damage analysis through distributed-plasticity approach and pushdown simulation techniques. While high-definition solid FE models based on classical principles of nonlinear fracture mechanics [53-55] represent an attractive solution for interpreting the evolution of crack patterns and damage mechanisms of RC structures at a local level [25, 33, 34], fiber-based idealizations were shown to be a transparent and viable approach to accurately characterize the inelastic response of such building typology in a computationally efficient manner [32]. To this aim, the open FE platform SeismoStruct [56] was adopted to prepare 2D and 3D models of EC2-compliant and EC8-compliant structures and to play out the series of pushdown analyses explicitly including material and geometric nonlinearities. Potential large displacements/rotations and P-Delta effects were taken into account using a total corotational transformation [52] and 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. 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 incremental monotonic loading. A one-to-six correspondence between structural members and model elements was assumed to reduce numerical instability and accommodate the deformed shape during progressive collapse analysis (Fig. 3). A simple bilinear constitutive rule with isotropic strain hardening was assigned to reinforcing steel, whereas the uniaxial uniform confinement model proposed by Mander et al. [57] was assumed to simulate the inelastic behavior of concrete, explicitly accounting for tension softening [40].

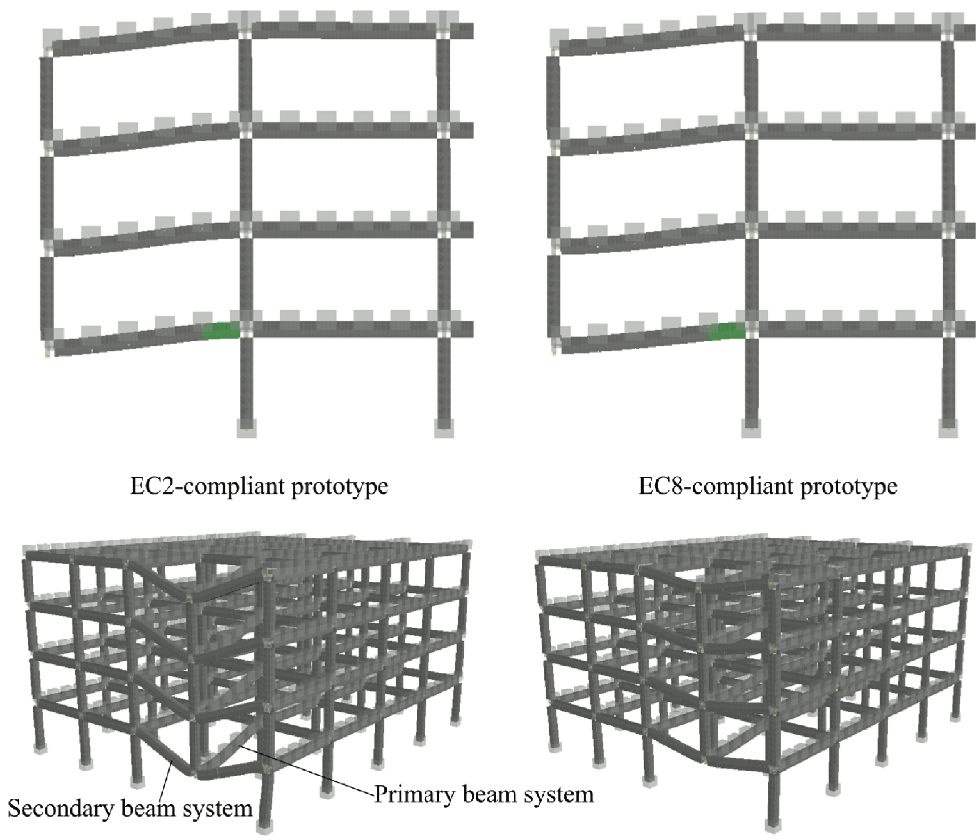

Figure 3: Deformed shapes at ultimate conditions of EC2-compliant and EC8-compliant buildings.

In order to characterize the nonlinear response of random populations of case-study buildings, two analysis types can be used in both direct and indirect progressive collapse-resistant assessment procedures: nonlinear static (or pushdown) analysis, and nonlinear dynamic analysis. The former is a relatively simple approach and provides a load-displacement capacity curve that, similarly to seismic analysis, permits one identifying whether a structure has adequate capacity to withstand extreme loads or not, in a static fashion. Material and geometric nonlinearities such as large displacements/rotations (i.e. beam-catenary action), second order effects, coupled axial-flexural inelastic behavior and plastic hinge formation (i.e. stiffness/strength degradation and ductility exploitation) can be included in the analysis using a nonlinear static method. Nonetheless, the main disadvantage of a static simulation is the inability to take into account the dynamic behavior of a structure in case that one or more bearing elements are instantaneously removed from the frame. This loading condition may cause highly dynamic effects which in turn may lead a statically safe structure to become dynamically unsafe due to the fact that time-dependent overloads produced by column loss may induce a progressive fracture of other members before a new equilibrium state is achieved [19, 22, 29, 34, 40]. As a result of this cascade effect, a different dissipative mechanism and related propagation of damage through the cross section depth may be predicted to occur by different analysis methods. If pushdown analysis is based on the application of a monotonically increased downward load or load distribution, during dynamic analysis the structure is damaged under cyclic reversals, as it oscillates upward and downward in such a way that more closely reflects the physical nature of progressive collapse.

In light of those considerations, procedures were proposed to equivalently include dynamic effects in a static fashion whether pushdown techniques are adopted for progressive collapse assessment [14, 15, 26]. In particular, the new Unified Facility Criteria (UFC) released in 2009 [15] prescribed the application of a non-uniform downward load distribution to the structure, explicitly identifying structural portions characterized by different levels of dynamic amplification (i.e. adjacent spans and areas away from the removed column). As such, this code-compliant approach was integrated within the probabilistic simulations performed to carry out fragility analysis, thus accounting for the dynamic nature of progressive collapse phenomena in a simplified and computationally efficient manner. As discussed in [40], fragility analysis needs an iterative procedure in case of dynamic analysis because a step function with monotonically increasing magnitude is required for an aprioristic factorization of Q_b that is able to simulate the different dynamic loading conditions corresponding to attainment of each limit state for each structure realization (Fig. 1). On the other hand, a single incremental analysis can be more rapidly performed in a static fashion and directly provides a set of displacement-controlled Q_b values for each building prototype which allows one evaluating multiple damage states at once.

The effectiveness of this equivalent nonlinear static procedure [15] was evaluated in comparison with IDA estimates [40] and a further set of standard pushdown analyses [14] was carried out assuming a uniform downward load distribution consistent with that selected for vertical mass when performing dynamic simulations. Therefore, pushdown capacity curves obtained by this latter approach were normalized with respect to IDA envelopes in order to compute a series of specific force-based DAFs for the two populations of EC2-conforming and EC8-conforming case-study buildings. Regression analyses were then performed using predictions for both 2D and 3D structural models and simplified equations were proposed to allow DAF to be estimated as a function of vertical displacement/beam drift. Furthermore, sensitivity of progressive collapse potential to simulation techniques with different levels of sophistication (i.e. IDA [40] versus pushdown [15]) was probabilistically assessed in terms of fragility functions.

The prevailing trends emerged from pushdown analysis with and without consideration of equivalent dynamic effects [15], namely P1 and P2, are discussed in the following.

4 PUSHDOWN ANALYSIS OF EC2-CONFORMING STRUCTURES

In the first stage of this research, progressive collapse capacity of EC2-compliant buildings was predicted by means of 2D and 3D structural models in combination with pushdown analysis techniques including or not dynamic effects in a static fashion (i.e. P1 and P2). This allowed the authors to carry out a two-way comparison between response predictions obtained (i) by different types of structural idealization for the same type of analysis (i.e. 2D versus 3D models), and (ii) by different analysis types for the same structural representation (i.e. P1 versus P2). Thus, the influence of dynamic effects and secondary beam systems on robustness of EC2-conforming case-study prototypes was quantified. The conclusions drawn from capacity curves presented in this section were then compared to those derived for earthquake resistant building class examined in Section 5 and finally extended in terms of fragility models, as discussed in Section 6.

4.1 Two-dimensional versus three-dimensional models: EC2-compliant buildings

To assess and compare the progressive collapse potentials of gravity-load designed structures in terms of analysis method and structural modeling, an internal 2D frame was first extracted from each 3D model and then analyzed by removing the leftmost ground column. That scenario was considered as it is the most demanding single column-removal condition in case of framed buildings [19]. As shown in Fig. 4, response estimates for the 2D models were then collected and compared to those provided by nonlinear static analysis of 3D models, in order to quantify the effects of secondary beam systems in the redistribution of gravity loads from a removed column. This sensitivity was initially investigated considering the results of standard pushdown analysis (i.e. P2) and the trends observed were then compared to those obtained by pushdown assuming a non-uniform load distribution (i.e. P1), thus exploring the influence of dynamic effects and corresponding equivalent load pattern on the resisting/redistribution mechanisms enforced in both primary and secondary frames.

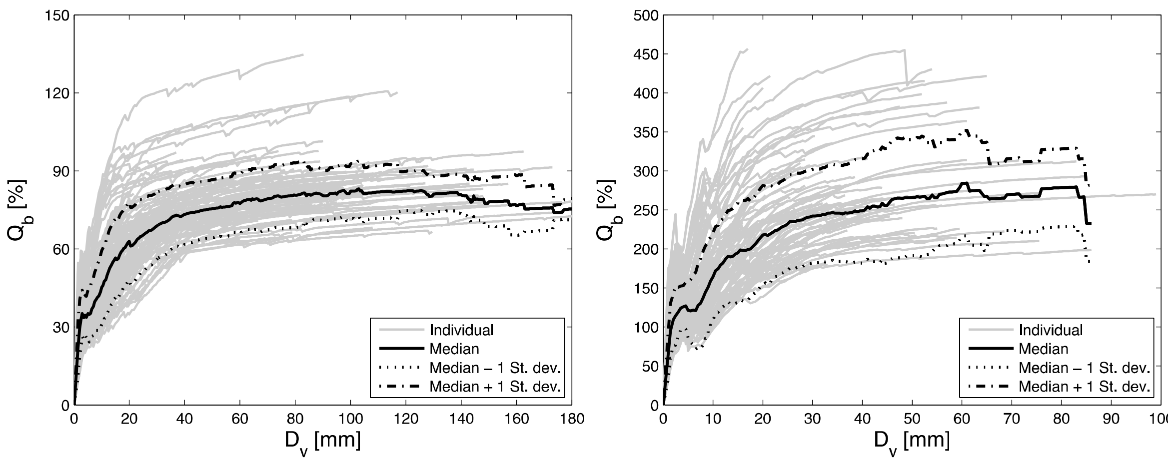

Figure 4: Pushdown (P2) curves of EC2-conforming buildings using 2D models (left) and 3D models (right).

Fig. 4 presents the set of downward load-vertical displacement curves predicted for 2D and 3D models, revealing a maximum progressive collapse capacity of about $1.35Q_b$ in case of the most resistant 2D structure. Peaks of up to $0.83Q_b$ can be observed if the median is considered, and capacities approximately lying within a ± 12% fork were shown when referring to median ± 1 standard deviation. The set of numerical predictions obtained by 3D models confirmed the significant influence of secondary framing beams on progressive collapse potential of the system, as they provide a rationally-controlled alternative load path for the unbalanced demand induced by a column loss scenario. Given that one-way slabs were assumed according to construction practice for such buildings, secondary beams were supposed to resist only their self-weight so they were designed to comply with minimum reinforcement requirements by current European standards [42] and, hence, they can be considered as a latent resource of stiffness and strength. The redundancy provided by secondary beams or "secondary-beam effect" visibly improved the robustness of case-study prototypes, being their contribution effective in controlling the overall structural response. As a result, capacities approximately 3 times higher than those shown for 2D models were predicted in this case considering either maximum or median resistance of the EC2-compliant buildings. Slightly more scattered estimates were determined as a wider fork of up to ± 24% resulted when computing median ± 1 standard deviation. As discussed by Brunesi et al. [40], predictions suggested that these secondary members may be actively used to interact with the primary frame systems, resulting in a stiffer, stronger and more stable response under column loss-induced overloads. Due to their visible participation in primary mechanism, vertical displacement peaks more than halved can be shown when 3D and 2D models are compared (Fig. 4).

4.2 Uniform versus non-uniform load distributions: EC2-compliant buildings

In this subsection, the sensitivity of progressive collapse resistance to analysis method was investigated, focusing on gravity-load designed 2D and 3D structural models. In detail, Fig. 5 collects the series of downward load-vertical displacement capacity curves determined for the EC2-conforming population of structures by means of pushdown analysis based on the application of code-compliant [15] non-uniform load distributions able to account for different levels of dynamic amplification in specific portions of the structure. Peculiar behavioral aspects of obtained response (i.e. P1) were examined. In addition, a comparison was provided with those discussed in subsection 4.1 for conventional pushdown analysis approach (i.e. P2).

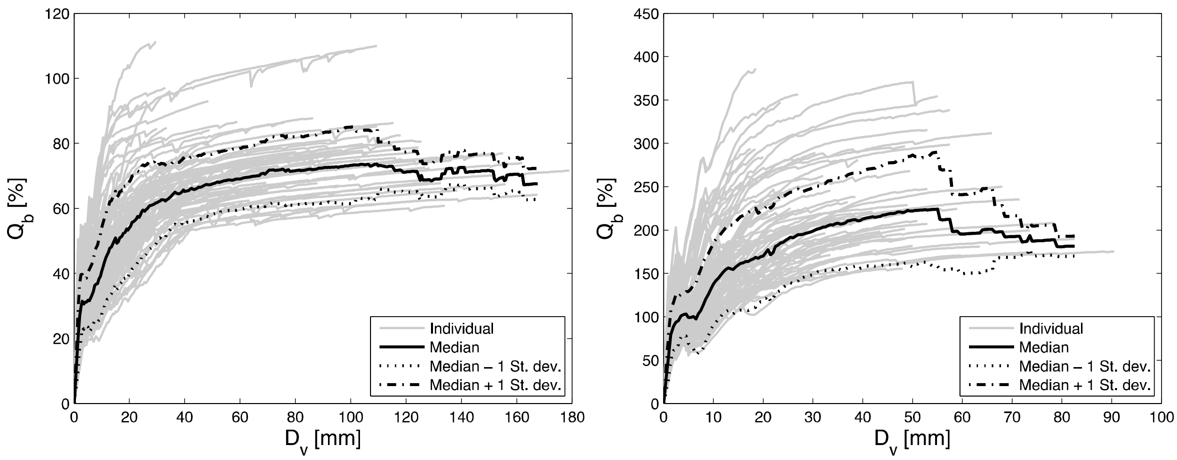

Figure 5: Pushdown (P1) curves of EC2-compliant buildings using 2D models (left) and 3D models (right).

Such an approach explicitly includes dynamic effects in a nonlinear static analysis using an equivalent/global procedure that provides a constant amount of overload dependent on plastic and yielding rotational capacity to be applied to specific structural portions more sensitive to a sudden column removal than others (adjacent spans versus areas away from the removed column). Therefore, predictions more conservative than those resulting from P2 were achieved in this case either for 2D or 3D models. Peaks of about $1.11Q_b$ and $3.86Q_b$ were predicted for the most resistant 2D and 3D structure, respectively. Such an outcome confirms the considerable contribution provided by secondary frame elements for gravity load redistribution as a consequence of a sudden column-removal condition. As previously mentioned, 3D models resulted in capacities approximately 3 times larger than those determined in case of 2D models. Compared to P2, estimates roughly 10%-20% smaller can be observed as a consequence of equivalent dynamic effects. By contrast, similar considerations can be drawn in terms of median and standard deviation.

4.3 Regression models for DAF estimation: EC2-compliant buildings

DAF can be considered as a mechanical measure that permits one to quantify the inaccuracy of pushdown analysis in comparison with IDA for different design targets beyond the elastic range of the structure. As such, DAF was obtained as the ratio between static and dynamic capacities in terms of vertical peak resistance of the 2D or 3D structural model at a given vertical displacement. Fig. 6 presents the series of force-based DAF versus vertical displacement curves of the population of gravity-load designed structures.

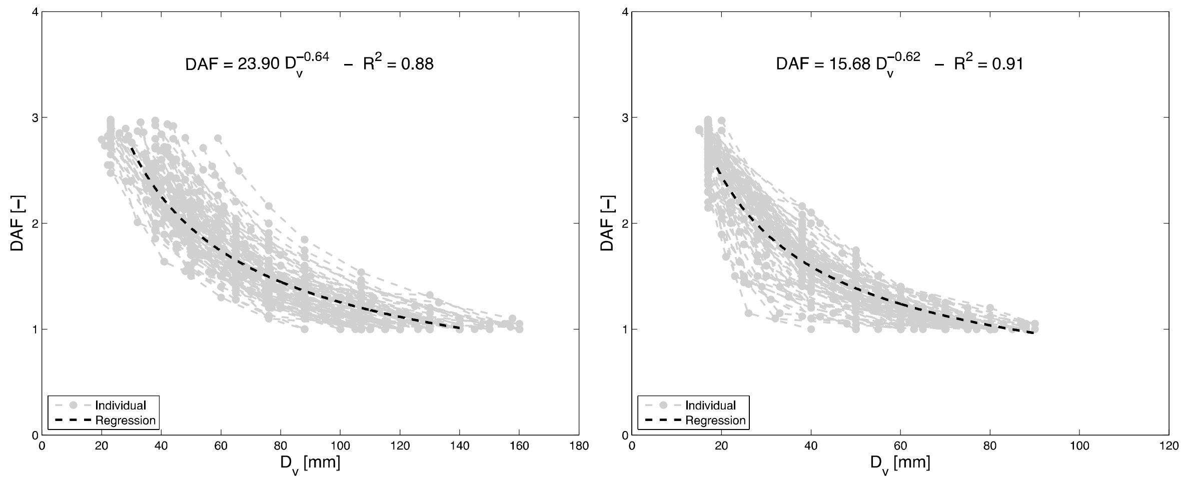

Figure 6: DAF estimates for EC2-conforming building class using 2D models (left) and 3D models (right).

To get a reliable estimate of this quantity, standard pushdown analysis (P2) was used as it is based on the application of a uniform downward load distribution, according to that assumed for nonlinear dynamic simulations [40]. The set of static-to-dynamic capacity ratios of 2D and 3D models collected in Fig. 6 were then used to conduct a regression analysis, which is aimed at providing specific analytical expressions for a quick assessment of DAF at a given vertical displacement/drift target. Accurate predictions were observed for both 2D and 3D models, as the coefficient of determination (R^2) was equal to 0.88 and 0.91, respectively.

5 PUSHDOWN ANALYSIS OF EC8-CONFORMING STRUCTURES

The procedures assumed to perform progressive collapse analysis of EC2-compliant building class were then applied to the population of EC8-compliant structures. As before, numerical simulations were carried out considering either 2D or 3D models. Pushdown analysis with and without consideration of dynamic effects was performed on prototype buildings having the same overall geometry in accordance with MC random sampling of RVs but different detailing as they were separately designed in compliance with EC2 [42] and EC8 [43]. In particular, the population EC8-conforming structures was designed for a medium-high seismicity, assuming the PGA at bedrock to be 0.30g for life safety limit state. A type C ground was selected to perform design in medium ductility class by response spectrum analysis on 3D models, incorporating accidental eccentricity of the center of mass. Further and more comprehensive details regarding simulated design criteria and related sectional properties/reinforcement layouts may be found in Brunesi et al. [40], while the prevailing numerical observations resulting from the series of pushdown analyses carried out are summarized hereafter, in direct relation to those derived for EC2-conforming case-study buildings.

5.1 Two-dimensional versus three-dimensional models: EC8-compliant buildings

The set of pushdown capacity curves obtained by 2D and 3D models are collected in Fig. 7, and reflected a visible benefit from seismic design/detailing rules. Peak force capacities 50%-70% larger than those observed in case of gravity-load designed structures were predicted for earthquake-resistant framed buildings.

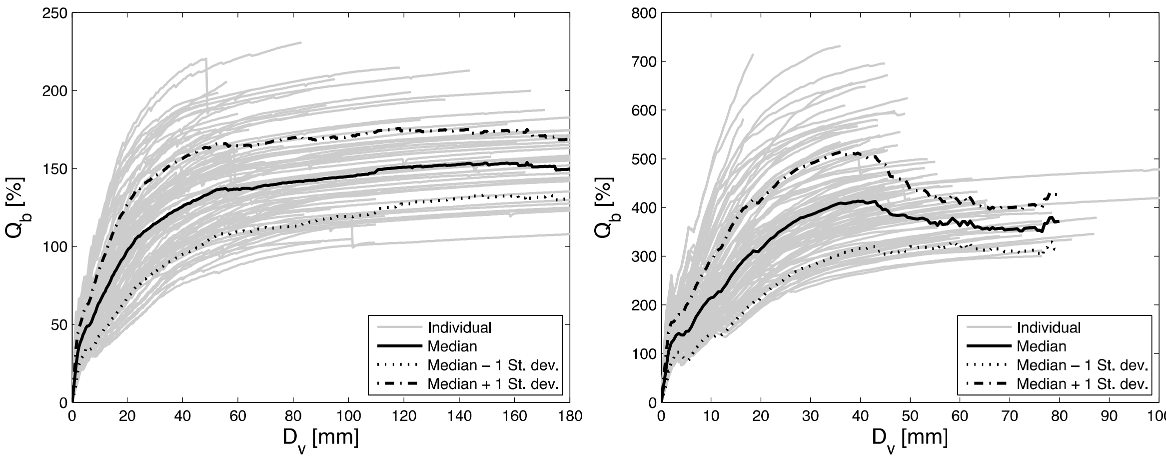

Figure 7: Pushdown (P2) curves of EC8-compliant buildings using 2D models (left) and 3D models (right).

Such an outcome is mostly due to more stable and robust flexural hinging mechanisms and arch effects/catenary actions developed in the critical portions of EC8-compliant structures, as a result of minimum seismic requirements and symmetrical arrangement of longitudinal reinforcement [43]. Compared to EC2-conforming buildings, their collapse modes were characterized by different levels of resistance, robustness, rationale and control. The EC8-conforming prototypes can displace further in the nonlinear regime of their response, or alternatively carry higher loads for a given vertical displacement. Maxima of about $2.31Q_b$ and $7.31Q_b$ were determined using 2D and 3D models, respectively. Hence, the trends emerged by comparing the responses of 2D and 3D gravity-load resistant framed structures were confirmed and corroborated even further by the series of pushdown curves shown in Fig. 7.

5.2 Uniform versus non-uniform load distributions: EC8-compliant buildings

Dynamic effects were included in progressive collapse analysis according to the equivalent procedure proposed in UFC [15], thus permitting one to evaluate their influence on robustness of EC8-conforming building class. In Fig. 8, the downward load-vertical displacement curves predicted by means of 2D models were compared to those obtained using a 3D structural idealization. The peak progressive collapse resistance was found to be equal to $2.09Q_b$ and $6.77Q_b$ in case that the strongest 2D and 3D prototypes are considered, respectively. Compared to Fig. 7, Fig. 8 reveals 10%-15% lower capacities, thus reaffirming the key role played by this code-compliant procedure in controlling the unsafety level of a standard pushdown approach. When considering the median of EC8-compliant buildings, the maximum load carrying capacity was approximately $1.39Q_b$ in case of 2D models and a ± 15% fork was observed whether referring to median ± 1 standard deviation. On the other hand, a more than doubled peak resistance resulted from the median of 3D models, in combination with a ± 10% wider fork for median ± 1 standard deviation.

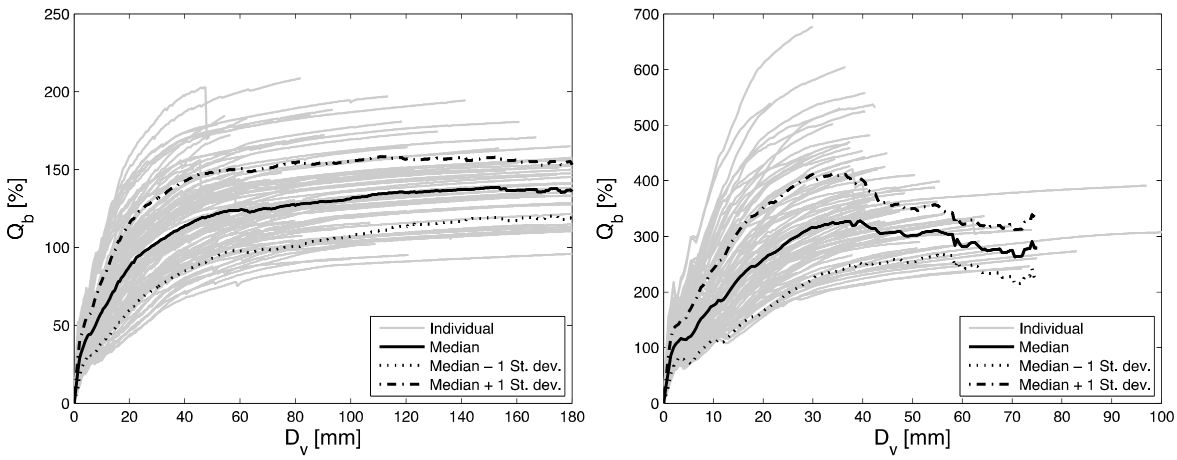

Figure 8: Pushdown (P1) curves of EC8-conforming buildings using 2D models (left) and 3D models (right).

The comparison between pushdown capacity curves of EC2-compliant and EC8-compliant random realizations (i.e. Fig. 5 versus Fig. 8) revealed trends similar to those evidenced when comparing the results of standard pushdown simulations (i.e. Fig. 4 versus Fig. 7). The significant benefit from seismic design/detailing criteria, which was highlighted for either 2D models or 3D models, can be therefore reaffirmed.

5.3 Regression models for DAF estimation: EC8-compliant buildings

Conventional pushdown predictions (i.e. P2) and IDA estimates [40] were used to quantify the discrepancy between the two simulation techniques at different levels of inelastic demand for column loss-induced overloads. As done in case of EC2-conforming prototypes, an exponentially decaying model was fitted to the series of static-to-dynamic capacity ratios obtained by 2D and 3D models (see Fig. 9), thus proposing simplified equations for quick analytical assessment of DAF for earthquake resistant buildings.

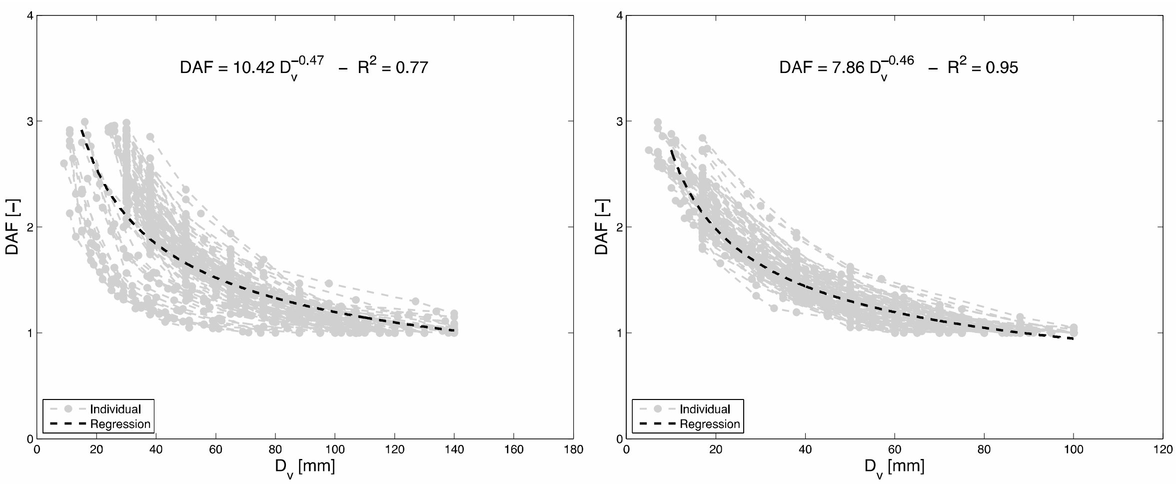

Figure 9: DAF estimates for EC8-compliant building class using 2D models (left) and 3D models (right).

The scatter is very low in case of 3D structural models, as the resulting regression function is characterized by a coefficient of determination equal to 0.95. Conversely, regression analysis provided a less accurate fit using capacities from 2D models (i.e. $R^2 = 0.77$), particularly in the small displacement range, that is when structural performance was characterized by minor or moderate damage.

6 DERIVATION OF FRAGILITY MODELS

To convolve capacity and demand under progressive collapse scenario, the following three criteria were introduced – either at global/structural (i.e. vertical drifts) or local/sectional (i.e. concrete and steel strains) levels – for limit states definition: slight (LS1), significant (LS2) and complete (LS3) damage. As discussed in [40], slight damage limit condition 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. Conversely, complete damage can be considered as an ultimate limit state beyond which the structure is close to collapse, being no longer able to deform and to sustain any further load increment nor the gravity loads for which it has been designed. Significant damage can be regarded as an intermediate case beyond which the building becomes unsafe for its occupants, being the majority of its source of nonlinearity yet fully in use. Therefore, LS2 may be identified as a sort of life safety limit state.

To determine whether a structural element reaches a limit condition, strains and drifts experienced in the critical portion of the skeletal frame where the progressive collapse mechanism was forced to occur were compared with conventionally identified limit capacities [40]:

- LS1 was defined by steel and concrete strains for both EC8- and EC2-compliant building classes. For each randomly generated structure, the yielding strain of steel bars was computed as the ratio between yielding strength and Young's modulus, while concrete strain at peak strength was determined in accordance with Mander et al. [57].
- LS2 was supposed to occur when the vertical drift, obtained as the ratio between the peak displacement above the column removed and the beam span length, exceeded a deterministic threshold. In detail, that drift was assumed to be equal to 0.5% and 1.0% in 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 randomly generated structure, the ultimate concrete strain was calculated according to different material properties and reinforcement arrangement, whereas the buckling/fracture strain of steel bars was set to 4% and 6% in the case of EC2-conforming and EC8-conforming building classes, respectively.

Therefore, a multiplier of Q_b was identified on each pushdown (P1) curve for each damage state, in order to assemble a damage probability matrix. The latter contained 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 according to MC simulation. Finally, 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 each structure typology are shown in Figs. 10 and 11, while their parameters, in terms of mean, standard deviation and coefficient of determination, are collected in Table 1. A good match with fragility points from pushdown analysis was observed for each limit state (i.e. LS1, LS2 and LS3), design approach (i.e. EC2- and EC8-compliant structures) and structural idealization (i.e. 2D and 3D models), being R^2 in the range 0.972-0.999.

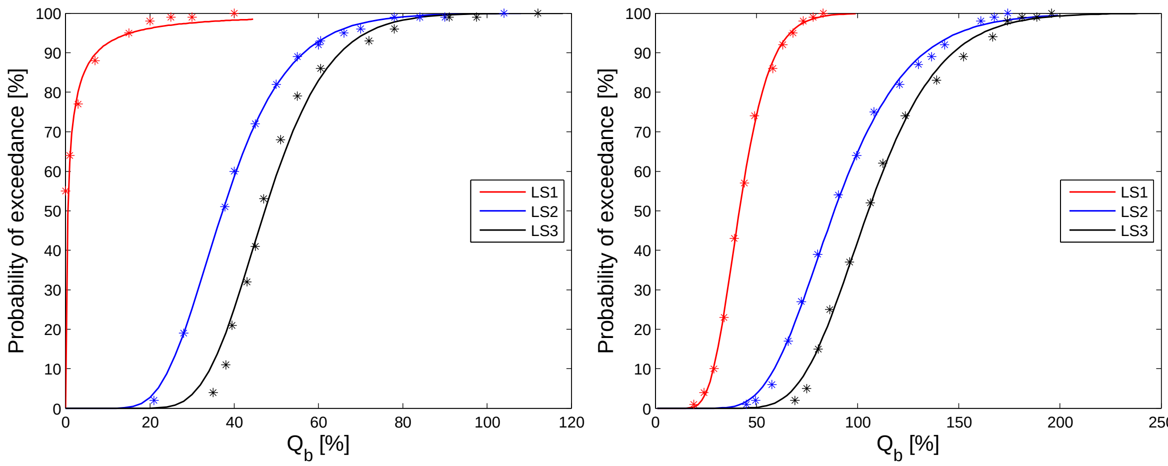

Figure 10: Fragility models of EC2-compliant (left) and EC8-compliant (right) building classes – 2D prototypes.

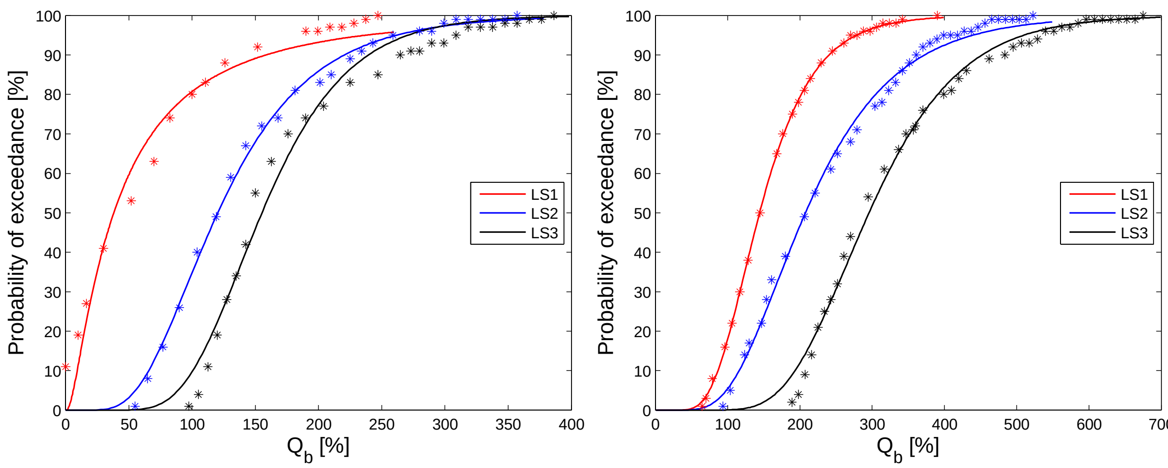

Figure 11: Fragility models of EC2-compliant (left) and EC8-compliant (right) building classes – 3D prototypes.

		LS1			LS2			LS3		
Class	**Model**	μ	σ	R^2	μ	σ	R^2	μ	σ	R^2
EC2-conf.	2D	-5.243	2.053	0.985	-0.987	0.325	0.999	-0.749	0.240	0.985
	3D	-0.964	1.112	0.972	0.186	0.472	0.997	0.440	0.338	0.982
EC8-conf.	2D	-0.880	0.288	0.999	-0.122	0.319	0.998	0.051	0.261	0.996
	3D	0.367	0.399	0.999	0.731	0.456	0.995	1.084	0.332	0.994

Table 1: Pushdown analysis (P1) – μ, σ and R^2 of fragility functions in terms of Q_b.

A very low scatter can be observed in case of EC8-conforming structures, as 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-compliant building class is concerned, an identical accuracy was obtained for LS2 and LS3, considering both 2D and 3D models, while the goodness of fit for LS1 was slightly lower as R^2 equals 0.985 and 0.972 for 2D and 3D models, respectively. Nonetheless, that limit state (i.e. slight damage condition) is less crucial than others 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 optimal design target is the life safety or collapse prevention limit state in light of classical performance-based design principles. Moreover, the fragility models proposed in this study reaffirm the key role played by secondary framing beams and seismic design/detailing rules for a cost-effective progressive collapse vulnerability mitigation of RC structures. In fact, a symmetrical reinforcement configuration is effective for the creation of a rationally-controlled primary resisting mechanism, whereas redundancy added by secondary beam systems (designed to satisfy minimum reinforcement requirements prescribed in current code regulations) is crucial to ensure alternative load paths under abnormal loading conditions. In this respect, it can be emphasized that the additional robustness resources provided by secondary frame systems can be predicted at the design stage, for instance through a correlation between the maximum vertical displacement of the 3D model normalized to that of the equivalent 2D model ($D_{max,3D}/D_{max,2D}$) and the maximum demand-to-capacity ratio of beams [19].

Finally, the set of fragility models at multiple damage states derived and commented in this research were compared to the corresponding set of IDA-based functions proposed by Brunesi et al. [40]. Table 2 presents ratios of mean ($\chi_\mu = \mu_{IDA}/\mu_{P1}$) and standard deviation ($\chi_\sigma = \sigma_{IDA}/\sigma_{P1}$) of IDA-based and pushdown-based fragility functions for each damage state, design approach and structural representation of interest.

		LS1		**LS2**		**LS3**	
Class	**Model**	χ_μ	χ_σ	χ_μ	χ_σ	χ_μ	χ_σ
EC2-conf.	2D	1.249	1.132	1.224	1.205	1.158	1.038
	3D	1.528	1.182	0.232	1.169	0.789	1.073
EC8-conf.	2D	1.321	1.459	2.015	1.225	-0.511	1.075
	3D	0.344	1.208	0.822	1.192	0.909	1.040

Table 2: IDA vs. P1 – ratios of mean ($\chi_\mu = \mu_{IDA}/\mu_{P1}$) and standard deviation ($\chi_\sigma = \sigma_{IDA}/\sigma_{P1}$) of fragility functions.

Pushdown analysis was therefore confirmed to be unconservative for vulnerability assessment of RC framed structures subjected to progressive collapse, as detailed from a deterministic standpoint in past studies [19, 40]. In light of this, ad hoc countermeasures are likely to be implemented in current regulations for progressive collapse scenario loss modeling.

7 CONCLUSIONS

In this work, a modeling procedure for large displacement inelastic pushdown analysis was integrated in a probabilistic framework for the derivation of fragility functions for RC framed structures under extreme loads resulting from a threat-independent sudden column loss. Two low-rise RC building classes were studied, namely gravity-load designed buildings and earthquake resistant buildings. The case-study structures were investigated using two analysis methods, i.e. pushdown analysis with and without consideration of equivalent dynamic effects, and two modeling strategies, i.e. 2D and 3D fiber-based models. The progressive collapse fragility was evaluated for both case-study building classes, considering both structural idealizations as well as uncertainties related to geometry of structural elements, material properties and gravity loads. Classical performance-based assessment principles assumed in earthquake engineering were extended to define demand and capacity at local and global structural levels. A set of fragility models were then derived, allowing the following conclusions to be drawn:

- The fragility models proposed in this study are optimally fitted to discrete fragility estimates provided by pushdown analysis combined with MC simulation, particularly for life safety and total 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 considered in probabilistic progressive collapse assessment.
- Considerable benefits from seismic design and secondary beams on the actual robustness level of RC bare building structures are confirmed and quantified by the fragility models proposed herein.
- Incremental-mass nonlinear dynamic analysis is suggested to be implemented in progressive collapse-resistant building analysis and design, as pushdown analysis was proven to underestimate the fragility level associated with this type of structures in case of progressive collapse.

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