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A MULTI-LEVEL SEISMIC FRAGILITY ASSESSMENT FRAMEWORK FOR EXISTING RC BUILDINGS

R. Gentile[(1)], C. Galasso[(2)]

[(1)] *Research Fellow, Department of Civil, Environmental and Geomatic Engineering & Institute for Risk and Disaster Reduction, University College of London, UK, r.gentile@ucl.ac.uk*
[(2)] *Associate Professor, Department of Civil, Environmental and Geomatic Engineering, University College of London, UK, and Scuola Universitaria Superiore (IUSS) Pavia, Italy, c.galasso@ucl.ac.uk*

Abstract

Fragility curves are commonly used in seismic performance assessment of structures. Various end users/stakeholders have different requirements on this matter: private owners likely need a detailed assessment focused on single buildings or small portfolios of buildings, while government agencies or (re)insurance companies might look at large portfolios requiring a lower refinement level and accepting higher uncertainties. In this paper, a multi-level framework for deriving seismic fragility is proposed to fulfil such needs, with particular reference to reinforced concrete (RC) frames. A refined approach based on non-linear time-history analysis is deemed to be appropriate for individual high-importance buildings (e.g. hospitals, schools) but it may result overcomplicated for assessing individual ordinary (e.g. residential) buildings. Instead, when dealing with large building portfolios, semi-empirical methods focused on "archetype" structural models are often the state-of-practice. These simplified approaches may provide a rapid-yet-accurate estimation of the seismic fragility, requiring a relatively small amount of input data. Such approaches often fail to capture specific deficiencies and/or failure mechanisms that might greatly affect the final assessment outcomes (e.g. shear failure in beam-column joints, in-plane and out-of-plane failure of the infill walls, among others). To overcome these shortcomings, the alternative response analysis methods in the proposed multi-level framework are characterised by: 1) a mechanics-based approach; 2) the explicit consideration of spectral shape in modelling seismic input/demands; and 3) the explicit consideration of record-to-record variability. Specifically, the proposed framework involves three levels for the seismic response analysis: 1) low refinement - non-linear static analysis (analytical SLaMA or numerical pushover), coupled with the capacity spectrum method; 2) medium refinement - non-linear time-history analysis of equivalent single degree of freedom (SDoF) systems calibrated based on either the SLaMA or the pushover curves; 3) high refinement - non-linear time-history analysis of full numerical models. In all cases, fragility curves are derived through a cloud-based approach employing unscaled real (i.e. recorded) ground motions. The proposed framework is applied to 14 four- or eight-storey RC frames showing different plastic mechanisms and distribution of infills. The results show that time-history analysis of SDoF systems is not substantially superior with respect to a non-linear static analysis coupled with the CSM. The estimated median fragility of the simplified methods generally falls within ±20% of the corresponding estimates from the full time-history analysis, excluding the uniformly-infilled frames, for which such error range increases up to ±40% due to the (preliminary) choice of a sub-optimal intensity measure. A number of limitations of the study, which are currently being addressed by the authors, are finally highlighted.

Keywords: seismic fragility; multi-level framework; SLaMA; pushover analysis; time-history analysis; error trends.

1. Introduction and motivation

Earthquake-induced direct and indirect losses tend to be high in highly populated, earthquake-prone areas, especially in countries where most of the existing buildings and infrastructure is designed according to pre-seismic codes. Therefore, there is a dire need to develop holistic strategies for mitigating and managing seismic risk, which firstly involve risk quantification. Fragility curves (i.e., likelihood of various damage levels as a function of a hazard intensity measure) are an important component of risk quantification exercises. Various end users/stakeholders have different needs on this matter: private owners likely need a detailed assessment focused on single buildings or small portfolios of buildings, while government agencies or (re)insurance companies might look at large portfolios requiring a lower refinement level and accepting higher uncertainties. In this paper, a multi-level framework for deriving seismic fragility is proposed to fulfil such needs. Particular focus is given to RC frames, which usually represent a high share for both residential and commercial occupancy among construction types built from 1960s onwards in several counties around the world (and especially in Europe). Apart from the fragility analysis, which is the focus of this paper, the proposed multi-level framework involves: 1) the exposure module, in which different amounts of input data can be compensated with sound assumptions based on simulated design; 2) the loss estimation module, spanning from the benchmark component-level approach proposed in the Federal Emergency Management Agency (FEMA) P-58 [1] guidelines and the Resilience-based Earthquake Design Initiative (REDi) rating [2] to a simplified building-level damage-to-cost approach; 3) multi-criteria decision making for optimal retrofit selection, combined with soft measures such as risk transfer (insurance) and/or tax rebates.

It is widely accepted in the research community [3] that reduced-order models of considered buildings are desirable for portfolio-level vulnerability assessments. However, no consensus is yet established for selecting the adequate number of sample buildings needed to characterise an asset class, including the within-building and building-to-building variabilities. Although it is reasonable to assume that building-to-building variability is dominant over the within-building variability, its proper representation requires a substantial knowledge of the building stock. This is seldom the case, and simulated design approaches are often adopted to reasonably "guess" the missing information. It is finally acknowledged that the (essentially inevitable) use of reduced-order models and/or simplified analysis methods may consistently bias the results [3], as well as providing a wrong representation of the uncertainties involved in the analysis (e.g. record-to-record variability). Therefore, a detailed knowledge of such a bias is very much needed for an informed development of a risk model. An objective of this paper is to analyse such bias in the context of the proposed multi-level framework for the seismic fragility analysis of RC residential buildings. Although the proposed alternative analysis methods involve both non-linear static and dynamic analysis, some (at least arguably) indispensable properties are shared among the considered approaches: 1) mechanics-based approach; 2) explicit consideration of spectral shape in modelling seismic input/demands; and 3) explicit consideration of record-to-record variability.

Non-linear time-history analysis (NLTHA) arguably represents the most advanced procedure for seismic fragility analysis; yet, it requires a detailed characterisation/modelling of both the building and the hazard at the site, high computational time/effort, and very specific skills for its implementation/results interpretation. Such a detailed approach is deemed to be appropriate for individual high-importance buildings (e.g. hospitals, schools) but may result overcomplicated for assessing individual ordinary (e.g. residential) buildings. Moreover, this level of detail is not always feasible, or even necessary, for instance, in applications dealing with large building portfolios. In the latter case, particularly-simplified empirical methods are the often the state-of-practice. Indeed, these applications focus on "archetype" structural models, often coupled with limited exposure information, which are not compatible with the level of detail and computing required by NLTHA. These simplified approaches may provide a rapid estimation of the seismic fragility, requiring a relatively small amount of input data. However, current approaches do not capture specific deficiencies and/or failure mechanisms that might greatly affect the final assessment outcomes (e.g. plan asymmetry-driven torsional effects, shear failure in beam-column joints, in-plane and out-of-plane failure of the infill walls, among others). Therefore, with regard to the fragility analysis, the proposed framework involves the following analysis types (from low to high analysis refinement): 1) the Simple Lateral Mechanism Analysis (SLaMA) [4–8], combined with the capacity spectrum method (CSM) [9]; 2) numerical pushover analysis, combined with the CSM; 3) NLTHA of inelastic single-degree-of-freedom (SDoF) systems, based on the SLaMA curve; 4) NLTHA of SDoF systems, based on the pushover curve; 5) NLTHA of a highly-refined full-scale numerical model. In all cases, fragility curves are derived through a cloud-based approach employing unscaled real (i.e. recorded) ground motions.

The proposed framework is demonstrated for 14 RC frame buildings, characterised by different height levels (four or eight storeys), plastic mechanisms (Beam-Sway, Mixed-Sway and Column-Sway), configuration of the infill panels (bare frame, uniformly-infilled frame, pilotis frame). Considering the results from case 5) as a benchmark, critical discussion of the error trends is provided, along with guidance to select the analysis method most consistent with the chosen trade-off between accuracy and simplicity.

2. Methodology

2.1. Seismic response analysis

Regardless of the selected refinement level within the proposed framework, the seismic response of the analysed structure(s) is represented by a cloud of points in the engineering demand parameter (EDP) vs intensity measure (IM) space. Maximum inter-story drift is the selected EDP; it is a convenient proxy highly correlated with (non)structural damage and repair costs. For all the case studies, the selected IM is defined as the geometric mean ($AvgSA$) of the pseudo-spectral acceleration in the range $T_{1,min} - 1.5T_{1,max}$, which are respectively the minimum and maximum first-mode periods of the case studies. This ensures increased efficiency and (relative) sufficiency in estimating a given EDP by means of a scalar IM [10,11]. For the application in this work, a set made of 150 unscaled natural (i.e., recorded) ground motions is selected from the SIMBAD database, "Selected Input Motions for displacement-Based Assessment and Design" [12]. As in [13], the 3-component 467 records in the database are ranked according to their PGA values (by using the geometric mean of the two horizontal components) and then keeping the component with the largest PGA value. The first 150 records are arbitrarily selected; hazard-consistent site-specific record selection is outside the scope of the study, especially considering the cloud-based approach for fragility/vulnerability derivation. According to the proposed framework, the resulting EDPs for this suite of ground motions can be computed using methods with increasing refinement (Table 1).

Table 1 – Adopted seismic response analysis methods.

Method	Refinement	Description
CSM-SLaMA	Low	Analytical capacity curve (SLaMA); Capacity Spectrum Method
CSM-PO	Low	Numerical capacity curve (pushover); Capacity Spectrum Method
TH-SDoF-SLaMA	Medium	SDoF time-history; SLaMA-based capacity curve;
TH-SDoF-PO	Medium	SDoF time-history; pushover-based capacity curve;
TH-FULL	High/Benchmark	Full-model time-history analysis

Non-linear time-history analyses (*TH-FULL*), representing the benchmark refinement level, are carried out for refined 2D numerical models defined using the finite element software Ruaumoko [14]. P-Delta effects are considered in the analyses. The modelling strategy (Fig. 1) is based on an experimentally-validated lumped plasticity approach which allows to predict flexural, bar slip and shear failure of RC beams and columns, along with strength degradation and possible shear failure in the joint panels. The modified Takeda hysteresis [15] is adopted for beams and columns, with the columns having a thinner loop. Beam-column joints are consistent with the Modified Sina model [15], which allows to consider a pinching behaviour. Fully fixed boundary conditions are considered at the base and floor diaphragms are modelled as rigid in plane. The selected EDP is directly extracted from the analysis results. It is worth mentioning that a non-linear static analysis (SLaMA or pushover) is always needed, since this allows to calibrate structure-specific damage state (DS) drift thresholds for the fragility analysis (regardless of the refinement level).

In the low-refinement level, the CSM is applied using the suite of real records described above. The formulations provided in [16] are used to calculate the effective mass and the equivalent viscous damping (EVD), which is considered equally for both bare and infilled frames. There are two options to derive the force-displacement curve of the structure, needed as input for the CSM. The first possibility is to conduct a numerical pushover analysis adopting the same numerical model described above (*CSM-PO*). The final capacity curve is expressed in terms of the displacement calculated at the effective height [16], to represent an equivalent SDoF system. Alternatively, SLaMA can be used to calculate the capacity curve (*CSM-SLaMA*). This analytical tool allows to derive both the expected plastic mechanism and the capacity curve of RC frame, wall and dual-system buildings by using a "by-hand" procedure (i.e., using an electronic spreadsheet). This allows to identify potential structural weaknesses in the lateral resisting mechanism and allows to test the reliability of numerical computer models in capturing the most probable behaviour of a structure. Each beam and column in the system is characterised considering many possible failure mechanisms (i.e., flexure, bar buckling, lap-splice failure, shear), considering that the weakest link will govern the overall structural behaviour. It is worth mentioning that, using a real spectrum (i.e., derived from actual ground-motion records) for the CSM, as opposed to a smooth code-based spectrum, may lead to multiple performance points. To the authors' knowledge, the literature provides no clear guidance on the selection among those solutions, and further investigation is needed. As a tentative approach, in such occasions the performance point with the smallest displacement is arbitrarily selected among those. After the performance point displacement is calculated for each ground motion, a displacement shape is adopted to calculate the corresponding inter-storey drift. The displacement shapes provided in [5] are adopted in the CSM-SLaMA case, while the displacement profile is extracted from the pushover analysis in the CSM-PO.

The medium refinement level in the framework involves a set of time-history analyses on a calibrated SDoF model of the structure. The backbone response of the SDoF is consistent with the non-linear static response of the structure, which can be derived both according to SLaMA (*TH-SDoF-SLaMA*) or a pushover analysis (*TH-SDoF-PO*). Clearly, appropriately-calibrated hysteresis rules should be used for such analysis. The selection of the hysteresis rule (and parameters) should be at least dependent on the expected plastic mechanism of the analysed structure (calculated based on SLaMA or pushover analysis). However, it may be argued that the estimation of peak-response EDPs (such as the maximum inter-storey drift; herein selected) may be relatively insensitive to the hysteresis parameters. For the application in this paper, the modified Takeda hysteresis rule is adopted. The maximum inter-storey drift for each ground motion is calculated based on the registered maximum displacement and the displacement shapes described above.

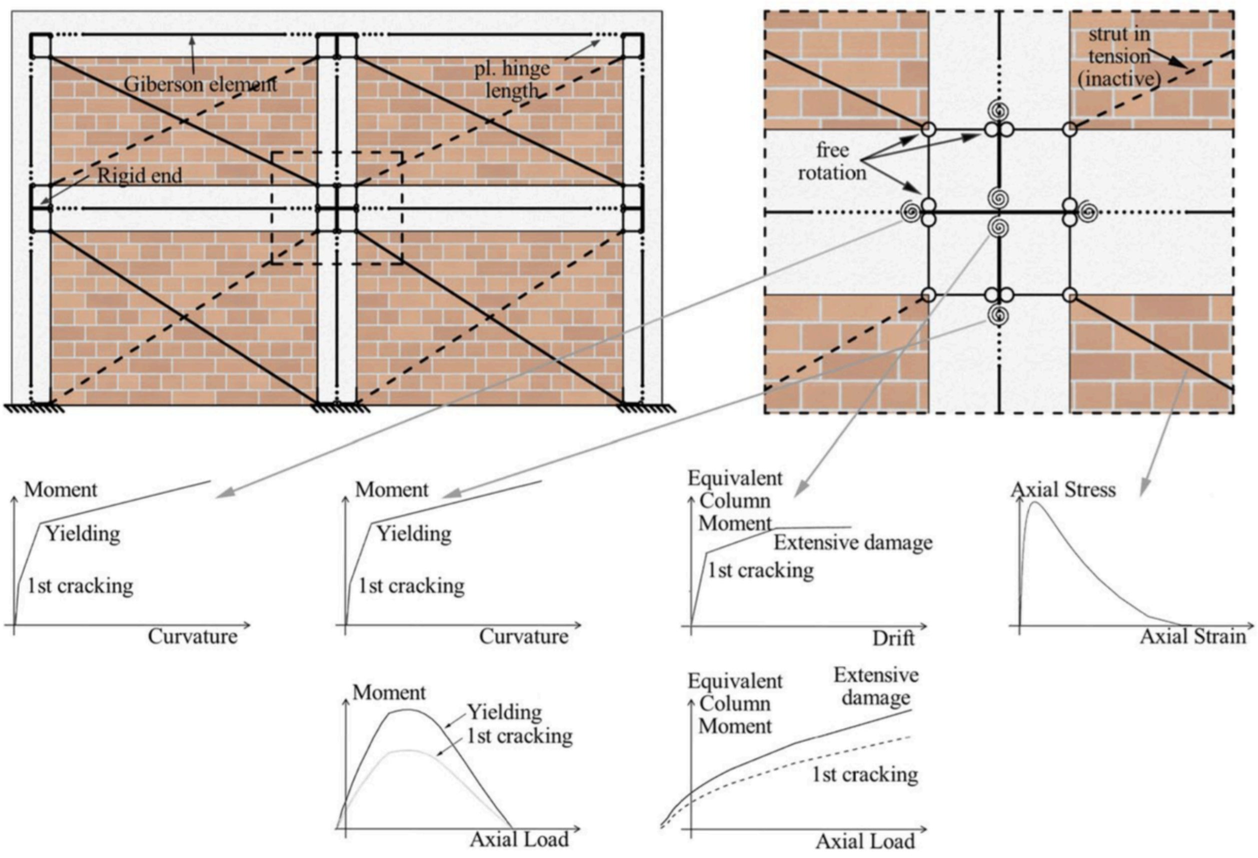

Fig. 1 – Numerical modelling strategy [6].

2.2. Fragility estimation

For this study, building-level fragility relationships are calculated for four DSs: slight, moderate, extensive and complete damage. Those DSs are defined according to HAZUS, HAZard United States [17], and quantified using the non-linear analyses results. The cloud of points resulting from the analyses is partitioned in two subsets: the "collapse (C)" cases, corresponding to ground motions leading to dynamic instability of the analysis or exceedance of a conventional 10% drift threshold; and the "non-collapse (NoC)" cases, corresponding to ground motions not leading to collapse. Eq. 1 describes the derivation of the fragility functions, where $P(EDP \geq EDP_{DS}|IM, NoC)$ is the conditional probability that the EDP threshold is exceeded given that collapse does not occur, and $P(C|IM)$ is the probability of collapse. It is implicitly assumed that the EDP threshold (EDP_{DS}) is exceeded for collapse cases, i.e. $P(EDP \geq EDP_{DS}|IM, C) = 1$.

$$F_{DS}(IM) = P(EDP \geq EDP_{DS}|IM) =$$

$$P(EDP \geq EDP_{DS}|IM, NoC)(1 - P(C|IM)) + P(C|IM) \quad (1)$$

The linear least square method is applied on the "NoC" EDP-IM pairs to derive the commonly-used power-law probabilistic seismic demand model $EDP = aIM^b$, where a and b are the parameters of the regression. This allows to define a lognormal cumulative distribution function (CDF) representing $P(EDP \geq EDP_{DS}|IM, NoC)$ for a given DS. The probability of collapse $P(C|IM)$ is fitted with a logistic regression, which is appropriate for cases in which the response variable is binary ("collapsed" or "non-collapsed"). As in [18], the final result is converted into a lognormal CDF, defined by a median and a logarithmic standard deviation (or simply dispersion).

2.3. Description of the case study frames

All the refinement levels of the proposed framework are demonstrated for 14 case studies representing the longitudinal frames of the RC buildings shown in Fig. 2. The transverse frames are not assessed in this paper, and their influence on the lateral capacity of the longitudinal frames is reasonably neglected. The case studies have four bays and either four or eight storeys. For each geometrical configuration, three different solutions are adopted for the detailing of the RC members, leading to three different expected plastic mechanisms: Beam-Sway (all beams and the base columns yield), Mixed-Sway (combination of joint shear failures with beam and/or column flexure, shear or lap-splice failures) and Column-Sway (soft storey mechanism at ground storey). For each plastic mechanism configuration, both a bare and a uniformly-infilled configuration is considered. Finally, a pilotis configuration (infills missing at the ground floor) is also considered for the Column-Sway cases. The reader is referred to [6] for details on the design of the case studies, the member detailing of each RC member, the adopted material models, the load analysis and mass properties.

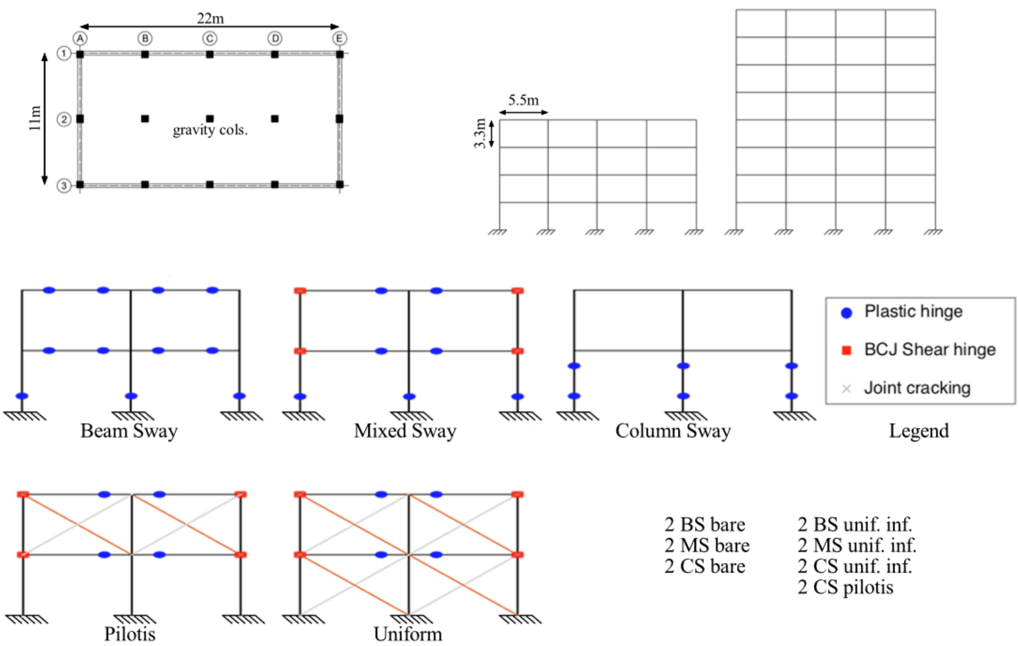

Fig. 2 – Analysed case studies; modified after (Gentile et al., 2019c).

3. Results and discussion

The results of the non-linear static analyses are discussed first, since those represent the first step of the framework regardless of the chosen refinement level. As an example, Fig. 3a shows the comparison of the pushover and the SLaMA curves for the Mixed-Sway, 4-storey bare frame, together with the plastic mechanism developed at DS3 (which is considered as ultimate limit state, consistently with Eurocode 8 [19]). As discussed in detail in [5,6], there is a particularly good match between the SLaMA and pushover curves at DS3 both in terms of displacement and base shear for all the four-storey case studies, while a slightly larger error is observed for the eight-storey ones. As expected, a greater error is measured for the initial stiffness and, most importantly, SLaMA considerably over-estimates the peak base shear capacity of the uniformly-infilled frames. All the SLaMA curve (normally ending at DS3) are extended up to the "P-Delta" point, where the second-order overturning moment equates the first-order one. Consistently with Eurocode 8 [19], the DS4 displacement is equal to four-thirds of the DS3 one.
Fig. 3b shows the assumed SDoF capacity curves, normalised by the effective weight. The pushover-based SDoF curve is obtained by a multi-linear fit of the pushover curve, calculated consistently with the provisions by the applied technology council [20]. The same procedure is used for the SLaMA-based SDoF curves of the uniformly-infilled frames, to appropriately consider the post-peak strength degradation. On the other hand, the residual strength of the SLaMA-based bare frames has been tentatively assumed to be 50% for Beam- and Mixed-Sway cases. 75% strength degradation is assumed for the Column-Sway frames, due to the pronounced softening behaviour of the first-storey columns which starts soon after yielding. Finally, a linear behaviour is assumed for the within-cycle degradation, which starts at DS4 and ends at twice such displacement. The tentative assumption for strength degradation in SLaMA leads to great discrepancies with the pushover curve for Beam- and Mixed-Sway bare frames (this issue is not evident for Column-Sway cases). As shown in Fig. 3b, this mismatch develops only for very large displacements, and it is unlikely to jeopardise the results.

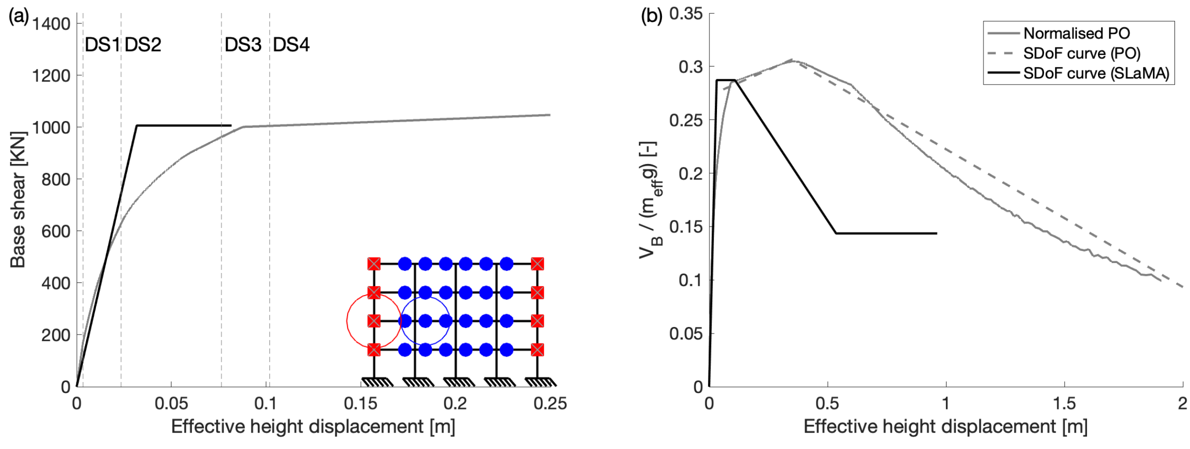

Fig. 3 – Mixed-Sway, 4-storey bare frame: a) pushover curve, SLaMA curve and plastic mechanism at DS3; b) assumed curves for SDoF representation.

The participating mass of the first vibration mode is approximately equal to 85% for the Beam-Sway and Mixed-Sway, four-storey bare and uniformly-infilled frames (80% for the eight-storey ones), while it is greater than 95% for the Column-Sway cases studies. The analysed structures are therefore first-mode dominated, which justifies using simplified response analysis methods. As shown in Fig. 4, this reflects in a particularly good match in the clouds across all refinement levels of the framework, as well as in the fragility curves. It is worth mentioning, however, that the fitting of the probabilistic seismic demand model (and therefore the fragility curve) is herein affected by the distribution of IMs of the ground motion record suite. In this case, the low-IM ground motions outnumber the high-IM ones, and thus the overall fitting is much more affected by the former group. For this reason, the power factor of the fitting is constrained to be greater than one, to avoid any

unsound downwards concavity in the EDP-IM plane. The above issue requires further investigation and it can be overcome by either using a record suite with a uniform distribution of IM or assigning weights in the fitting of the power law, possibly proportional to the IM itself.
Given the high strength and the particularly-stable response behaviour up to high displacements, no collapse is registered for this particular case study. However, this is not the case for the Column-Sway frames, which show considerably less strength. Moreover, the first-storey columns of these case studies show a pronounced softening behaviour, which causes a softening response of the whole first storey and, in turn, of the entire structure. In the time-history analyses (medium and high refinement level), this causes instability phenomena due to P-Delta effects, which are clearly more evident for the eight-storey frames.

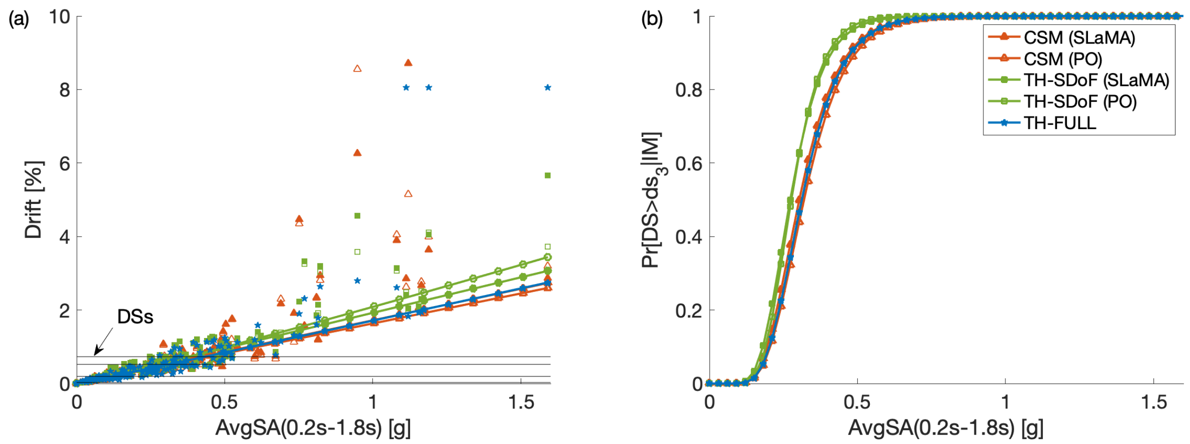

Fig. 4 – Mixed-Sway, 4-storey bare frame: a) seismic response analysis; b) DS3 fragility curves.

The discussion of the overall results for the fragility parameters focuses mainly on the life-safety damage state, or DS3, unless stated otherwise. As a measure of accuracy of the simplified methods, Fig. 5 shows the "F-ratios": the ratio of the fragility parameters with respect to the TH-FULL. Considering the bare and pilotis frames, the estimated median fragility of the low- and medium-refinement methods falls within ±20% of TH-FULL, showing the high accuracy of such analysis methods, despite their inherent simplification. The above error range increases up to ±40% for the uniformly-infilled frames. However, such trend may be linked to the choosing the same IM for all the case studies, both bare and infilled, despite their large difference in the first-mode period of vibration. Indeed, such trends should be re-considered (and possibly modified) after defining avgSA consistently with the (considerably-lower) first-mode period of the infilled frames. Similar trends are seen for the other DSs, with the relative errors being slightly higher for DS1. However, this is likely affected by the very low nature of the DS1 fragility (averaging 0.04g for TH-FULL), and it is not deemed to represent a shift from the main error trend. The error for the dispersion of the fragility is more uniform, bounded in the range ±30% for all DSs and case studies, with the exception of the Column-Sway ones showing a slightly higher error. Such result is likely connected with the estimation collapse, based on both numerical instability of the numerical solver (when appropriate) and a conventional drift threshold. Indeed, a numerical instability is more likely for the TH-FULL method rather than the TH-SDoF one. However, this is more-clearly connected with a physical instability (P-Delta driven) for the TH-SDoF, while also triggered by local numerical convergence issues for the TH-FULL. Therefore, the issue of defining collapse cases (possibly refining the conventional drift threshold) still requires some investigation, and possibly further tests using different procedures, e.g. a limit based on second-order effects or energy balance [21].
An interesting aspect of the results is that the TH-SDoF method is not substantially superior with respect to the analytical CSM. Both methods, for a given characterisation of the capacity curve (SLaMA or pushover), show a similar bias with respect to the TH-FULL for bare and pilotis frames. Greater differences are seen for uniformly-infilled frames with Beam-Sway or Column-Sway plastic mechanisms. However, this implies a higher performance of the TH-SDoF method only for four-storey cases (CSM provides better results for the

eight-storey frames). Such result may be related to the chosen EVD model in the CSM, which may be inappropriate for infilled frames. Surprisingly, for Column-Sway, uniformly-infilled frames the TH-SDoF method is at least as biased as the CSM. Clearly, the above trends should be re-considered using a more appropriate IM for infilled frames, as discussed above.
The comparison between the SLaMA- and pushover-based methods, for a given response analysis approach (CSM or TH-SDoF), shows that for bare and pilotis frames the SLaMA-vs-pushover error on the capacity curve has little-to-no effect on the estimation of fragility median. On the other hand, higher errors are shown for the infilled frames, caused by the propagation of the SLaMA-vs-pushover discrepancy for the peak base shear in the capacity curve, already reported in [6]. It is worth mentioning, however, that such error propagation is only significant for Column-Sway, uniformly-infilled frames, causing shifts in the fragility median as high as 40%. Negligible effects are observed for the estimation of the fragility dispersion. This propagation effect is particularly similar across all DSs, and is overall not deemed to be significant.

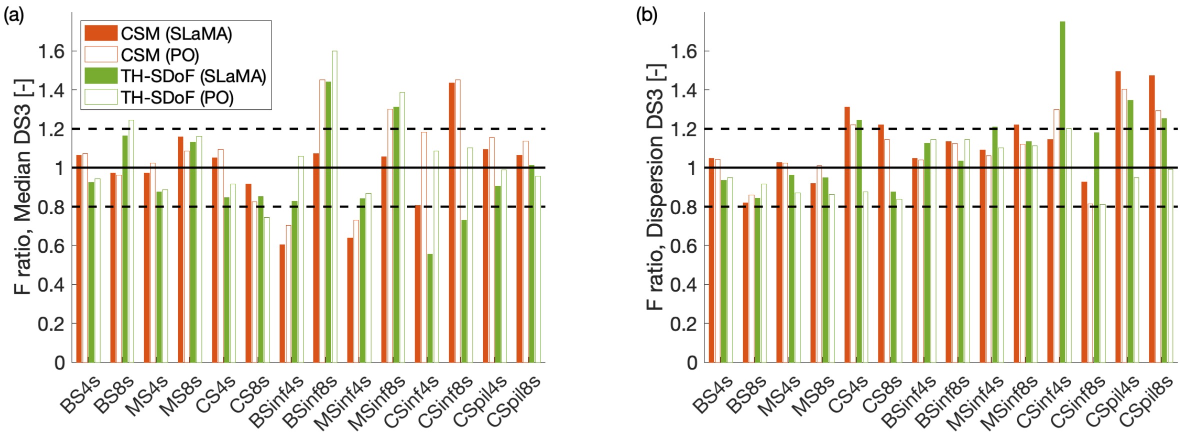

Fig. 5 – Fragility parameters of the low- and medium-refinement methods as a ratio of the TH-FULL: a) DS3 median; b) DS3 dispersion. [BS: Beam-Sway; MS: Mixed-Sway; CS: Column-Sway; 4s: four storeys; 8s: eight storeys; inf: uniformly infilled; pil: pilotis]

4. Conclusions and limitations/future work

This paper proposed a multi-level framework for deriving seismic fragility to fulfil the needs of different end users/stakeholders with regard to risk quantification. Private owners likely need a detailed analysis consisting of single buildings or small portfolios of buildings while government agencies or (re)insurance companies might look at large portfolios requiring a lower refinement level and accepting higher uncertainties. Particular focus is given to RC frames, which usually represent a high share for both residential and commercial occupancy among construction types built from 1960s onwards.
The framework involves three different levels of refinement for the seismic response analysis: 1) low refinement - non-linear static analysis (analytical SLaMA or numerical pushover), coupled with the CSM; 2) medium refinement - non-linear time-history analysis of SDoF systems calibrated based on either the SLaMA or the pushover curve; 3) high refinement - non-linear time-history analysis of full numerical models. The proposed framework is demonstrated for 14 RC frame buildings, characterised by different height levels (four or eight storeys), plastic mechanisms (Beam-Sway, Mixed-Sway and Column-Sway), configuration of the infill panels (bare frame, uniformly-infilled frame, pilotis frame). The proposed framework is applied to 14 four- or eight-storey RC frames showing different plastic mechanisms and distribution of the infills. Although still preliminary, the results can be summarised as follows:

- For the bare and pilotis frames, the estimated median fragility of the low- and medium-refinement methods falls within ±20% of the corresponding estimates from the full time-history analysis. The above error range increases up to ±40% for the uniformly-infilled frames. This result is valid for al DSs, with a slight increase for DS1.

- The error for the dispersion of the fragility is more uniform, bounded in the range ±30% for all DSs and case studies, with the exception of the Column-Sway ones showing a slightly-higher error;
- The time-history analysis of SDoF systems is not substantially superior with respect to a non-linear static analysis coupled with the CSM, regardless of the adopted characterisation of the capacity curve (SLaMA or pushover);
- For a given response analysis approach (CSM or SDoF time-history analysis), for bare and pilotis frames the SLaMA-vs-pushover error on the capacity curve has little-to-no effect on the estimation of fragility median. Higher errors are shown for the infilled frames, caused by the propagation of the SLaMA-vs-pushover discrepancy for the peak base shear in the capacity curve. Negligible effects are observed for the estimation of the fragility dispersion. This propagation effect is particularly similar across all DSs, and is overall not deemed to be significant.
- These preliminary results are affected by a number of limitations, and more effort is currently on-going by the authors to overcome those. In order of (expected) importance, future improvements involve: 1) adoption of different (optimal) IMs for the various case studies; 2) refined calibration of the drift threshold for collapse; 3) identification of a clear rule to select among different possible solutions of the CSM for the same ground motion spectrum; 4) ad-hoc calibration of the hysteresis type and parameters for the SDoF time-history analyses; 5) refined calibration of strength degradation for the SLaMA curve of bare frames.

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