Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2023) 392-413

ECCOMAS
Proceedia

COMPDYN 2023
9th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Athens, Greece, 12-14 June 2023

SIMPLIFIED ANALYTICAL/MECHANICAL PROCEDURE FOR THE RESIDUAL CAPACITY ASSESSMENT OF EARTHQUAKE-DAMAGED REINFORCED CONCRETE FRAMES

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Abstract

The series of recent catastrophic earthquakes worldwide have further emphasized the evident complexity and difficulty related to the evaluation of the post-earthquake seismic residual capacity of buildings. In the aftermath of a major seismic event, a fast, yet effective, safety evaluation procedure for earthquake-damaged buildings is critical to speed up and support the definition of emergency planning strategies, as well as to provide useful intel to the stakeholders and aid the decision-making process to enhance community resilience.
Therefore, this paper aims to investigate the possible implementations of a procedure based on SLaMA (Simple Lateral Mechanism Analysis) methodology for the seismic assessment of damaged Reinforced Concrete (RC) frame buildings. The proposed procedure is based on the use of reduction coefficients for damaged structural members, in line with the FEMA 306 approach, and an update of the "hierarchy of strength" at the subassembly level by accounting for the earthquake-related damage. Results are compared against a numerical model in terms of a Capacity vs. Demand Safety Index" (IS-V or %New Building Standard, %NBS) and Expected Annual Losses (EAL). Moreover, the simplified procedure can be used to assess the feasibility and effects of a repair/retrofit solution.
Results show that the proposed analytical procedure is able to estimate with reasonable accuracy, considering its simplified nature, and the performance of the building when compared to numerical analyses. Finally, the effect of the use of low-damage exoskeletons based on the PRESSS low-damage technology has been evaluated via the application of the Displacement-Based Retrofit procedure.

Keywords: Seismic assessment; Seismic residual capacity; Post-earthquake; Reinforced concrete; Non-linear analysis

ISSN:2623-3347
Peer-review under responsibility of the organizing committee of COMPDYN 2023.
doi: 10.7712/120123.10409.20391

1 INTRODUCTION

Following a major seismic event, the assessment of the residual capacity of damaged buildings represents a critical step in the decision-making process guiding the recovery effort.

A fast, yet effective procedure, based on a mechanical/analytical approach rather than a numerical one, can considerably speed up the information-gathering process, enabling faster planning and intervention and leading to enhanced community resilience. Current first level of Tier 1 procedures rely on a "tagging" process based on a visual inspection (e.g., [1, 2]) which can lead to inconsistent results based on the judgement of who carries out the inspection. An easy-to-apply standardized simplified procedure has the ability to reduce inconsistency between results without relying on complex and time-consuming detailed numerical models by filling the gap between the initial evaluation process, based on tagging alone, and advanced ad-hoc evaluations on single buildings.

In fact, after the initial evaluation procedure based on a tagging process, there is the need to proceed with a Detailed Engineering Evaluation (DEE, Eg. [3]) which does not only rely on visual inspections but requires a thorough assessment. This is needed because even the lack of significant damage to a structure is usually not enough to expect a satisfactory behaviour in future events. Despite existing several regulated procedures for the initial evaluation and visual tagging for Reinforced Concrete (RC) buildings, there is a lack of a standardized DEE procedure that needs to be consistent, comprehensive, auditable and understandable, both by engineers and stakeholders.

In this paper an analytical/mechanical approach for RC frames is proposed and investigated, combining the well-established SLaMA procedure, adopted by internationally recognized guidelines for the seismic assessment of existing buildings ([4]), with the approach proposed by FEMA 306 to evaluate the residual capacity of buildings in terms of safety and losses. Furthermore, a modified SLaMA approach is here proposed to address some of the most relevant shortcomings that could hinder the reliability of the procedure for damaged frames. Finally, a conceptual strategy/solution for the retrofit of damaged frames is proposed making use of high-performing exoskeletons.

The paper is structured as follows: an overview of the existing methodology for seismic residual capacity assessment of the earthquake-damaged buildings is reported in section 2; in section 3, the proposed SLaMA-based methodology is presented; the effectiveness of the proposed procedure is investigated in section 4 through a parametric investigation and the results are discussed in section 5; finally, conclusions are given in section 6.

2 RESIDUAL CAPACITY RC STRUCTURES: LITERATURE REVIEW

In the last decades, the seismic residual capacity of earthquake-damaged buildings (i.e., their ability to withstand a subsequent seismic event such as an aftershock or a triggered earthquake) has been widely investigated. In the wake of the Canterbury 2010-2011 earthquake sequence, there was the sudden realization that there was a gap of knowledge regarding the evaluation of the performance of damaged buildings, let alone their repairability ([5]).

In addition to visual inspection-based methodologies for post-earthquake damage classifications and usability decisions (e.g., [1, 2]), mechanical-based procedures for a more detailed evaluation of the effects of earthquake-related damage on the seismic performance of buildings have been developed. In this research line, the Federal Emergency Management Agency (FEMA) 306 report [6] and the Japan Building Disaster Prevention Association (JBDPA) guidelines ([7]; overview in English available in [8]) arguably represent the most relevant documents at the international level. Both documents adopt a non-linear static approach based on the use of modification factors to assess the seismic response of damaged structural members. More specifically, in the approach adopted by the FEMA 306 report, the effects of earthquake-related damage to structural members are simulated by capacity reduction factors in terms of stiffness (λ_K), strength (λ_Q), and ductility (λ_D) for their plastic hinges' response (Figure 1). These λ-factors are provided as a function of the component typology (e.g., isolated wall), the behaviour mode (e.g., flexural, shear, flexural/shear), and the observed post-earthquake damage (e.g., slight, moderate, severe, extreme). To support the engineers and professionals in the practical application of the procedure, the report also provides a "Component Damage Classification Guide" consisting of tables and charts with a description (including sketches of the typical crack patterns) of each level of damage. Some provisions on the possible repair/restoration interventions are also provided for each post-earthquake damage level. It is worth mentioning that the report only focuses on RC and masonry wall structures, while only some considerations for infill panels and their seismic interaction with the surrounding RC frame are given.

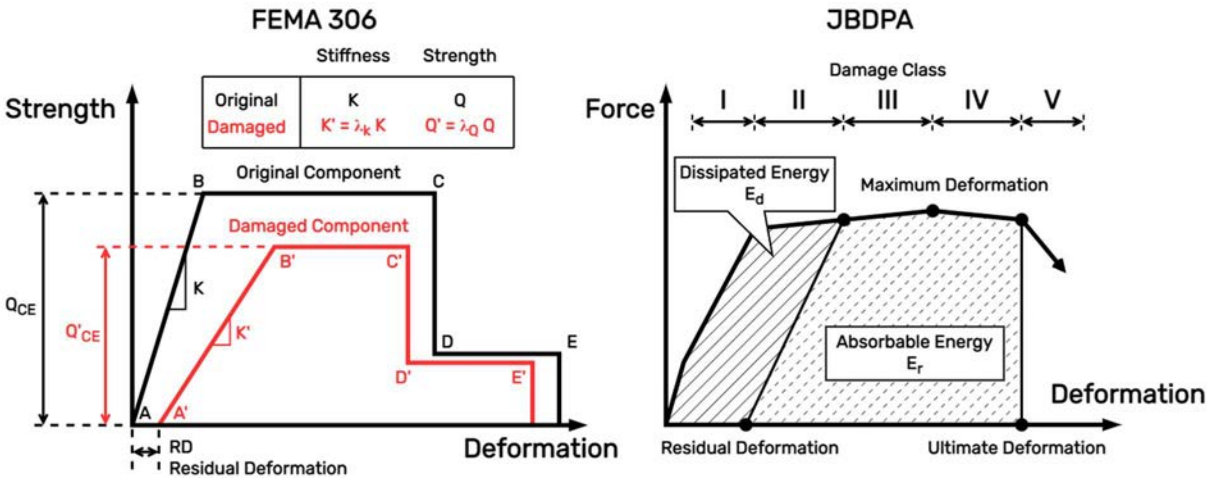

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Figure 1: FEMA 306 methodology for the capacity of a damaged component (Modified after FEMA 306, left) and JBDPA approach (Modified after Maeda et al. [8], right).

Differently from the FEMA 306 report, the JBDPA guideline adopts a single capacity reduction factor η for damaged structural members, defined as the ratio of residual energy dissipation capacity to the original one. These η-factors have been derived from the results of past experiments and numerical investigations and are provided as a function of member typology (e.g., beam, column, wall), the behaviour mode (e.g., ductile, brittle) and the "damage class". The latter is identified based on the observed damage; the guideline defines 5 damage classes, from "I" to "V", where class "V" represents the more severe damage level (Figure 1). The seismic residual capacity of the structure is finally identified through an index-based approach. More details on the procedure can be found in Maeda et al. [8].

A conceptually similar approach to the one described in the JBDPA guideline has been also adopted to develop the post-earthquake damage assessment algorithm for the Turkish Catastrophe Insurance Pool (TCIP). The last revised version of this assessment procedure (i.e., TCIP-DAM-2020, [9], Figure 2) allows for the evaluation of the Building Damage Category (BDC) through a two-stage procedure, involving an exterior and interior assessment phase. More specifically, the former (exterior assessment) aims to collect and classify any significant damage to the building detachable from the outside, e.g. global/partial collapse, significant residual displacement and/or rigid rotations. If no substantial damage is observed, the interior assessment is performed and the BDC is identified according to standardized damage state thresholds for the vertical and horizontal structural members. Six different BDCs are defined, namely: (i) Undamaged Building; (ii) Slightly Damaged Building; (iii) Moderately Damaged Building; (iv) Heavily Damaged Building; (v) Building to be Urgently Demolished (Demolish As Soon As Possible); and (vi) Collapsed Building. More details on the procedure are given in Ilki et al. [9].

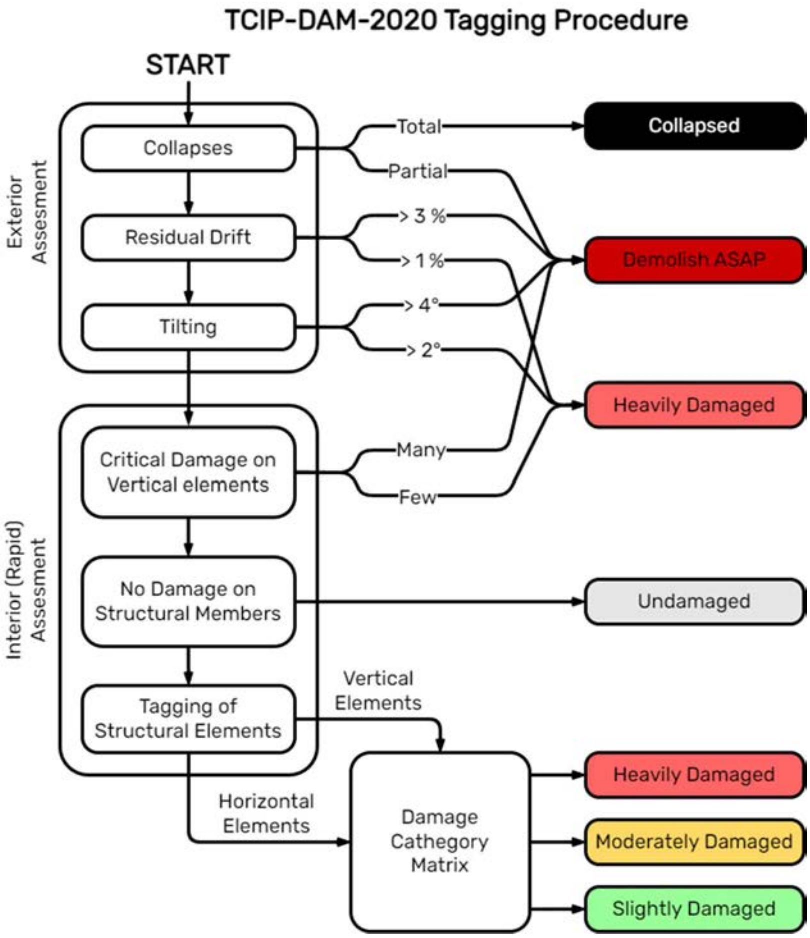

Figure 2: A simplified overview of tagging procedure proposed by TCIP for earthquake-damaged buildings. (ASAP * As Soon As Possible).

In line with the approaches described in these documents, several analytical, numerical, and experimental investigations have been carried out to better understand the seismic residual capacity of earthquake-damaged buildings. Di Ludovico et al. [10] derived λ-factors for not-seismic-code-compliant RC columns typical of the Mediterranean region. To implement the study, a database of cyclic experimental results was considered, involving columns with both smooth and deformed bars. The authors proposed suitable regression-based formulations to obtain the λ-factors as a function of the rotation ductility demand. More recently, Marder et al. [11] tested 17 nominally identical modern RC beams, considering different loading protocols and constraints. The experimental results have been considered by the same authors ([12]) to derive a lower-bound formulation for residual stiffness of moderately damaged (i.e., until cover concrete spalling) modern RC beams. Rossi et al. [13] adopted a loss-assessment framework to evaluate the effects of post-earthquake damage and repair intervention on RC walls. The seismic response of different RC walls was investigated through a refined Finite Element Method (FEM) model. The same modelling strategy has been adopted by Ceccarelli et al. [14]

to derive capacity reduction factors in terms of stiffness and strength for RC walls with different failure modes (i.e., flexural, shear, and flexural/shear). These reduction factors have been used to investigate the influence of earthquake-related damage on the economic seismic losses of a case-study wall structure. Following the FEMA 306 methodology, Polese et al. [15] proposed a framework to derive damage-dependent vulnerability curves for RC frame structures through a non-linear static approach. A conceptually similar pushover-based procedure has been recently introduced by Pedone et al. [16] in order to perform state-dependent fragility analyses of RC frame structures. The proposed methodology allows one to directly account for the record-to-record variability in the fragility estimation. Cuevas and Pampanin [17] proposed a displacement-based framework for the assessment of the residual capacity of RC frame structure considering mainshock-aftershocks sequences. The same authors ([18]) experimentally investigated the residual capacity and the effects of repair intervention on three beam-column joints subassemblies extracted from a 22-storey RC building demolished after the 2010-2011 Canterbury earthquake sequence. Moreover, the effects of strain ageing on the residual fatigue life of steel reinforcement have been also investigated (e.g., [19]).

3 PROPOSED ANALYTICAL METHODOLOGY

The proposed methodology relies on the well-established SLaMA analytical/mechanical simplified procedure ([4, 20]) by modifying the element capacity following the FEMA 306 [6] approach as proposed by Pampanin [21]. The proposed framework is independent of damage states and can be performed with observed damage using reference charts such as the ones contained in the FEMA 306 itself.

3.1 Application to SLaMA

The Simple Lateral Mechanism Analysis (SLaMA) is an analytical/mechanical procedure for the assessment of Reinforced Concrete (RC) and UnReinforced Masonry (URM) existing buildings. This approach enables a rapid assessment of the safety and losses of existing not seismic-code-compliant buildings, without relying on complex and time-consuming numerical simulations on finite element models. This methodology yields good accuracy in the results considering the simplified nature of the procedure, as demonstrated by numerous comparisons with detailed numerical models (e.g., [22-25]).

The SLaMA method is composed of five major steps (Figure 3):

1. Collection of the relevant building data such as geometry, material proprieties and reinforcement details.
2. Evaluation of element-level capacities in terms of moment-rotation and shear-moment interaction.
3. Assessment of sub-assembly performance through the evaluation of the hierarchy of strengths.
4. Definition of the expected global capacity in terms of inelastic failure mechanism and force-displacement capacity curve; safety and loss assessment.
5. Evaluation of the seismic performance of the structure through a capacity vs demand comparison in an Acceleration-Displacement Response Spectrum (ADRS) domain.

The rapidity of the procedure makes this approach suitable for time-sensitive applications such as screening of damaged buildings in the aftermath of a major seismic event. Thus, it is herein suggested to combine the SLaMA method with the application of reduction coefficients for stiffness (λ_K), strength (λ_Q), and ductility (λ_D), following the FEMA 306 approach, to account for initial earthquake-related damage to structural components.

In the proposed SLaMA-based methodology, the reduction coefficients are applied after the sub-assembly performance of the undamaged structure is evaluated (Figure 3), in order to reduce the number of steps and calculations needed to carry out the procedure. In fact, in the original procedure, the sub-assembly performance curve inherits the characteristics of the weakest element as determined in the hierarchy of strength analysis. The capacity reduction coefficients (or λ-factors) used for each subassembly are therefore selected based on the weak element of the sub-assembly such as column, beam, or joint panel. More specifically, in this paper, the coefficients proposed by [10] are adopted for beams and columns, while an assumption of secant reloading is made for joint panels due to the lack of data in the current state-of-the-art. Clearly, different criteria for the definition of the λ-factors can be also adopted (for instance, if more relevant research outcomes become available in this field in the future) without affecting the effectiveness of the proposed methodology.

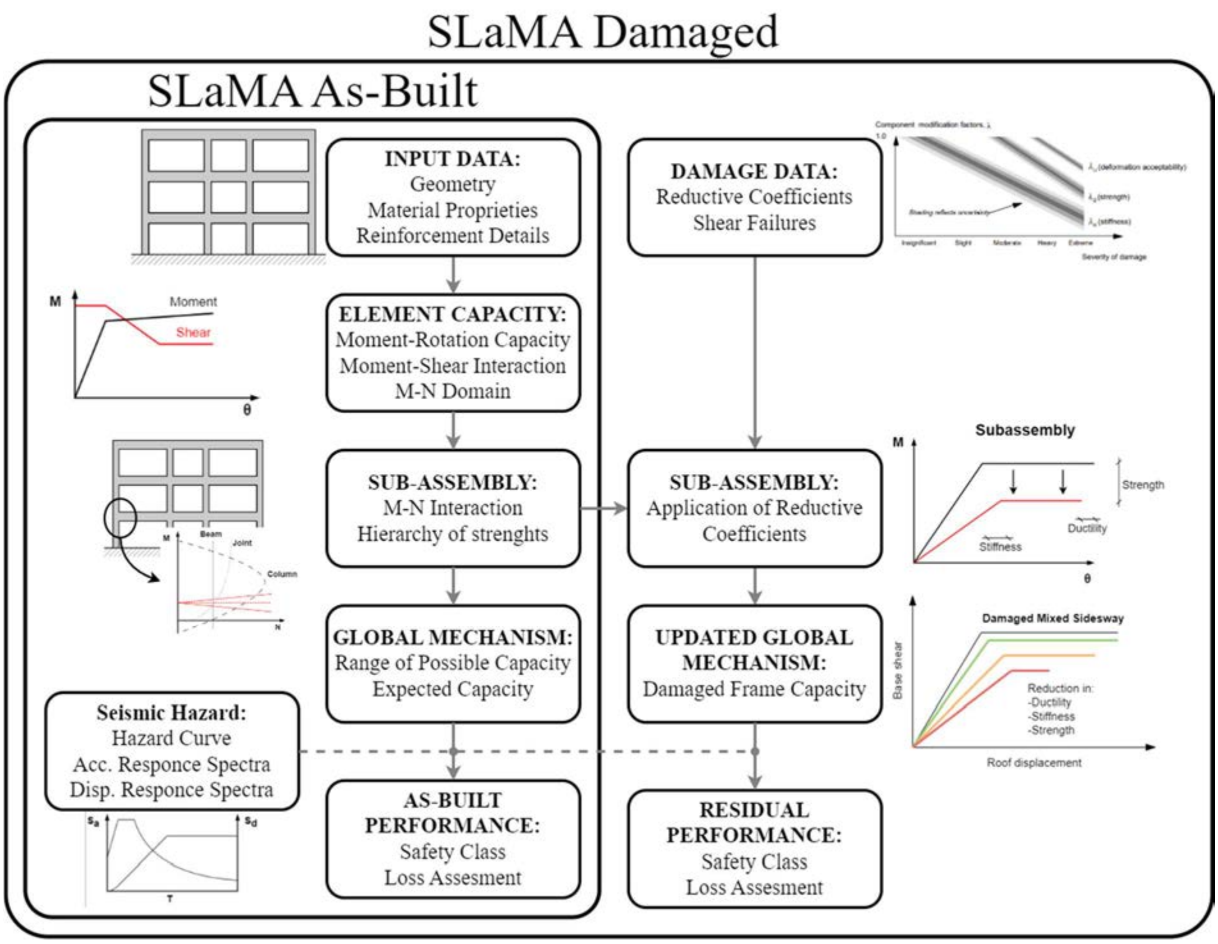

Figure 3: Outline of the original procedure and the proposed one.

It is worth reminding that the SLaMA method can lead to a possible overestimation of the elastic stiffness, based on the assumption of equivalent elasto-plastic system with fully developed inelastic mechanism. When dealing with damaged frames, the main expected effect on the seismic response is instead a stiffness reduction (as also highlighted by the FEMA 306 report), potentially leading to a loss of serviceability and a reduction of the expected damping. Hence, correctly capturing the yielding point of the structure is key for a reliable safety and performance assessment of damaged buildings.

For this reason, a refinement of the SLaMA procedure is then proposed to overcome some of the SLaMA procedure shortcomings.

3.2 Refined SLaMA Approach

To gain a better estimation of the elastic stiffness of the frame, a refinement of the SLaMA procedure is here proposed. From a technical point of view, the possible overestimation of the stiffness by the SLaMA methodology is related to the assumption that the base shear at yielding displacement is the same as the ultimate displacement. This concept has been highlighted in past research work in the literature and some possible refinements have been already introduced (e.g., [25,26]).

In this work, a more in-depth analysis is carried out at a sub-assembly level to investigate the expected stiffness of the frame and the level of participation of the various sub-assemblies of the structure in the global mechanism. The procedure is still based on the assumption of a linear displacement profile and is designed to require minimal additional calculation, due to the analytical nature of the approach.

The first step involves the computation of the sub-assembly stiffness starting from the moment-rotation relationships of the structural elements. The stiffnesses of the elements that make up the sub-assembly are combined in a simplified rheological model considering beams and columns in parallel to each other and in series with the joint panel (Figure 4). The equivalent stiffness is then used to redefine the yielding rotation considering the moment of the weakest element (Eq. 1).

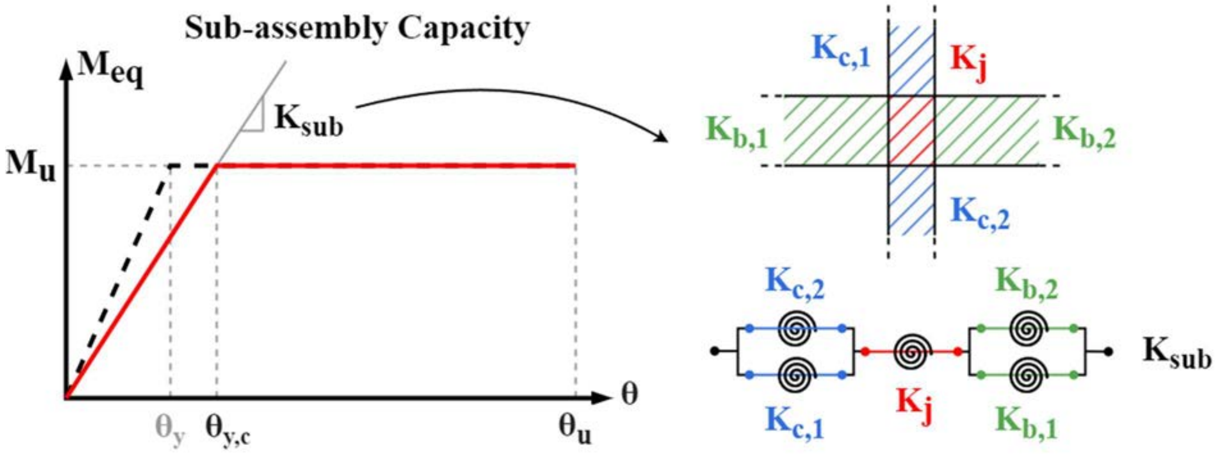

Figure 4: Assessment of the sub-assembly performance considering elements' stiffnesses.

The sub-assembly stiffness can be computed as follow:

$$K_{sub,beam,ed} = \frac{1}{n_{beam}}\left(\frac{1}{K_{b,1}+K_{b,2}} + \frac{1}{K_{c,1}+K_{c,2}} + \frac{1}{n_{col}K_j}\right)^{-1} \tag{1}$$

Where $n_{beam}$and n_{col} are the number of beams and columns in the sub-assembly, respectively. In this formulation, the stiffness of the joint (K_j) is computed using the equivalent lower column moment capacity of the joint panel.

After computing the new yielding rotation for each sub-assembly considering the elements' contributions, the critical sub-assembly is identified as the one with the lowest yielding rotation.

Therefore, the moment of each sub-assembly is computed under the assumption that every sub-assembly and every column undergo the same rotation defined as the yielding of the critical sub-assembly. Afterwards, the overturning moment is obtained using the previously computed moments. Base columns' contributions to the overturning moment are also scaled using the same imposed drift. Finally, the base shear is derived together with the yielding displacement (Figure 5) considering the effective height of the building. The ultimate base shear and displacement are obtained using the classical SLaMA approach.

The use of an effective subassembly stiffness hinders the simplicity that characterizes the application of the reduction factors at the subassembly level as described in the previous paragraph. Anyway, is possible to derive analytical formulas to update the sub-assembly stiffnesses without having to recompute all the mechanisms.

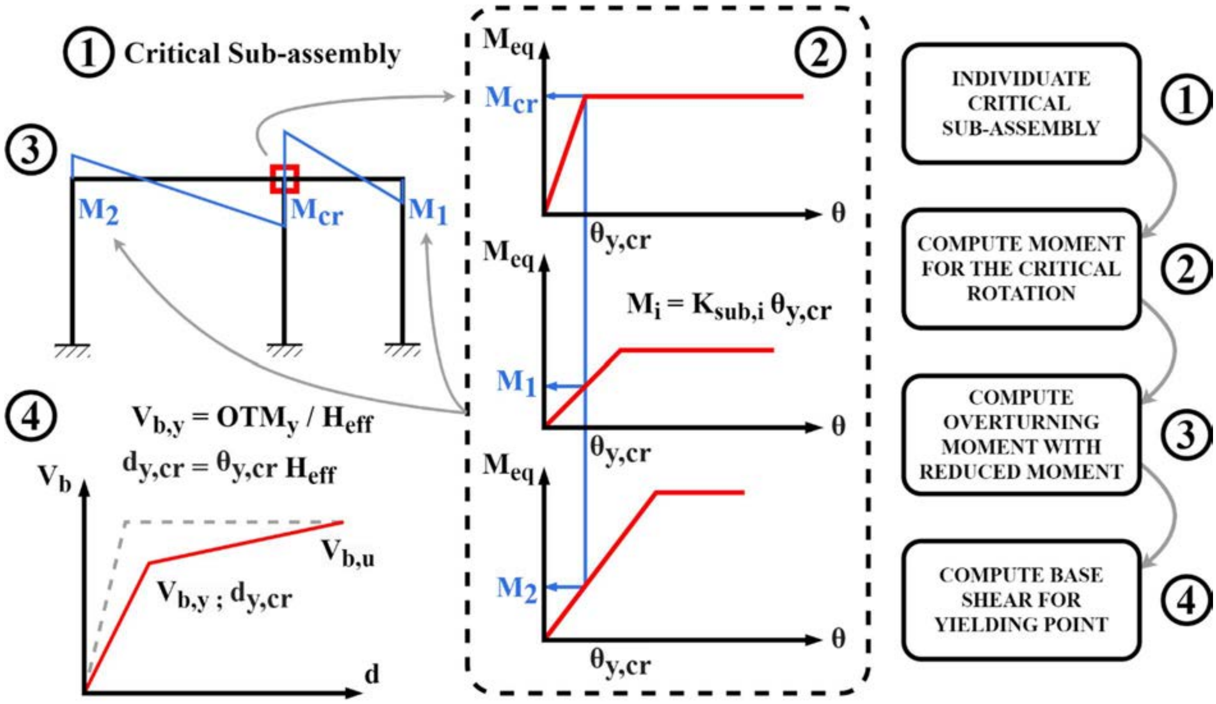

Figure 5: Definition of the yielding mechanism.

The updated stiffnesses of the sub-assembly can be computed as in Eq. (2), (3). Specifically:

- If the damaged element is either a column or a beam:

$$K_{sub,dmg} = \left(\frac{1}{K_{sub,old}} + \frac{n_{beam}}{K_{c,b}}\frac{1-\lambda_k}{\lambda_k}\right)^{-1} \tag{2}$$

- If the damaged element is the joint panel:

$$K_{sub,dmg} = \left(\frac{1}{K_{sub,old}} + \frac{n_{beam}}{n_{col}K_j}\frac{1-\lambda_k}{\lambda_k}\right)^{-1} \tag{3}$$

Where:

- $K_{sub,old}$ is the equivalent stiffness of the undamaged sub-assembly.
- λ_k is the stiffness reduction factor for the damaged element.
- K_j is the stiffness of the undamaged joint panel.
- $K_{c,b}$ is the stiffness of either the undamaged column or beam.

The reduction coefficients for strength and ductility are applied as described in FEMA306 [6]. The ductility achieved by the element can be easily obtained by removing from the imposed drift the elastic contribution of the stronger elements.

3.3 Performance Evaluation

The seismic performance of the structure is then evaluated according to the procedure described in Italian seismic risk classification guidelines ([27, 28]). This methodology relies on the assessment of two performance indexes: one related to the safety and one assessing the expected losses.

The safety index IS-V (Index of Safety for Life-Saving limit state, equivalent to %New Building Standard, %NBS, adopted by the NZSEE guidelines [4]) is defined as the ratio between the seismic capacity and demand at the Life Safety Limit State (LSLS) and is computed through a capacity/demand comparison within an ADRS domain (Figure 6a).

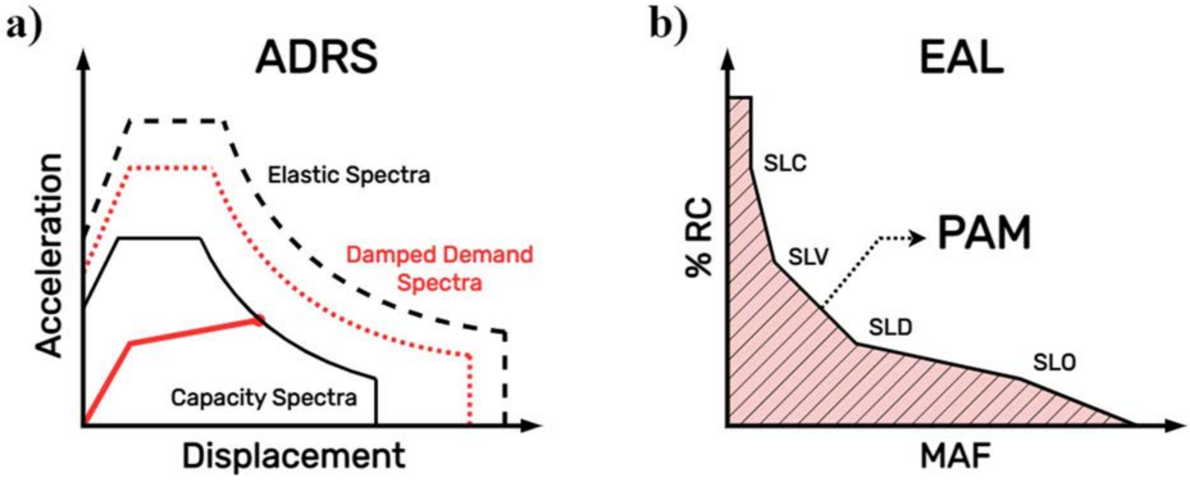

Figure 6: Methodologies for a) safety performance evaluation and b) expected annual loss (EAL or PAM).

The evaluation of the losses is carried out by assessing the EAL index (Expected Annual Loss, or PAM, "Perdita Annua Media" in [29]). The Italian guideline provides a simplified approach to computing the EAL, where fixed percentages of Reconstruction Cost (%RC) are associated with the achievement of specific Limit States (LSs) (Figure 6b). These %RC values have been calibrated using the extensive database on the repair cost of damaged buildings following the 2009 L'Aquila earthquake ("White Book" [30-32]). The procedure requires assessing the performance of the structure at different LSs (at least at the Ultimate and Serviceability Limit States, respectively SLV and SLD in Italian Building Code [29]) in terms of Mean Annual Frequency (MAF) of exceedance.

4 CASE STUDY BUILDING

To implement the study, six case-study RC frame structures are selected, characterized by either a different number of stories and/or bays. The selected frames are representative of archetype pre-1970s building in Italy, i.e. designed for gravity load only according to old pre-seismic-codes provision. Therefore, the typical structural weaknesses of buildings designed in that period are expected, e.g. a lack of "capacity design" principles (strong beam/weak column), inadequate amount of transversal reinforcement for shear capacity and confinement, poor construction details, and low quality of construction materials. In this investigation, it is assumed that the case-study buildings are located in Udine, Italy (soil type C) (Table 1).

LS	a_g	F_0	T_c*	Soil Cat.
SLD (SLS)	0.074	2.474	0.262	C
SLV (ULS)	0.208	2.446	0.332	

Note: $\boldsymbol{a_g}$=PGA on bedrock soil type; $\boldsymbol{F_0}$=dynamic amplification factor; $\boldsymbol{T_c^*}$=corner period

Table 1: Parameters of the site's seismic hazard according to the Italian building code [34].

The structural skeleton consists of 3 moment-resistant frames in the longitudinal direction and two 2-bay frames in the transversal direction. The parametric investigation is carried out considering the longitudinal central frame, assuming the same interstorey height of 3.0m and a different number of stories (3, 4, and 5); moreover, a different number of bays are considered (3 and 5). Figure 7 shows the global dimensions of the case-study structure and the geometrical detail of the analysed frames.

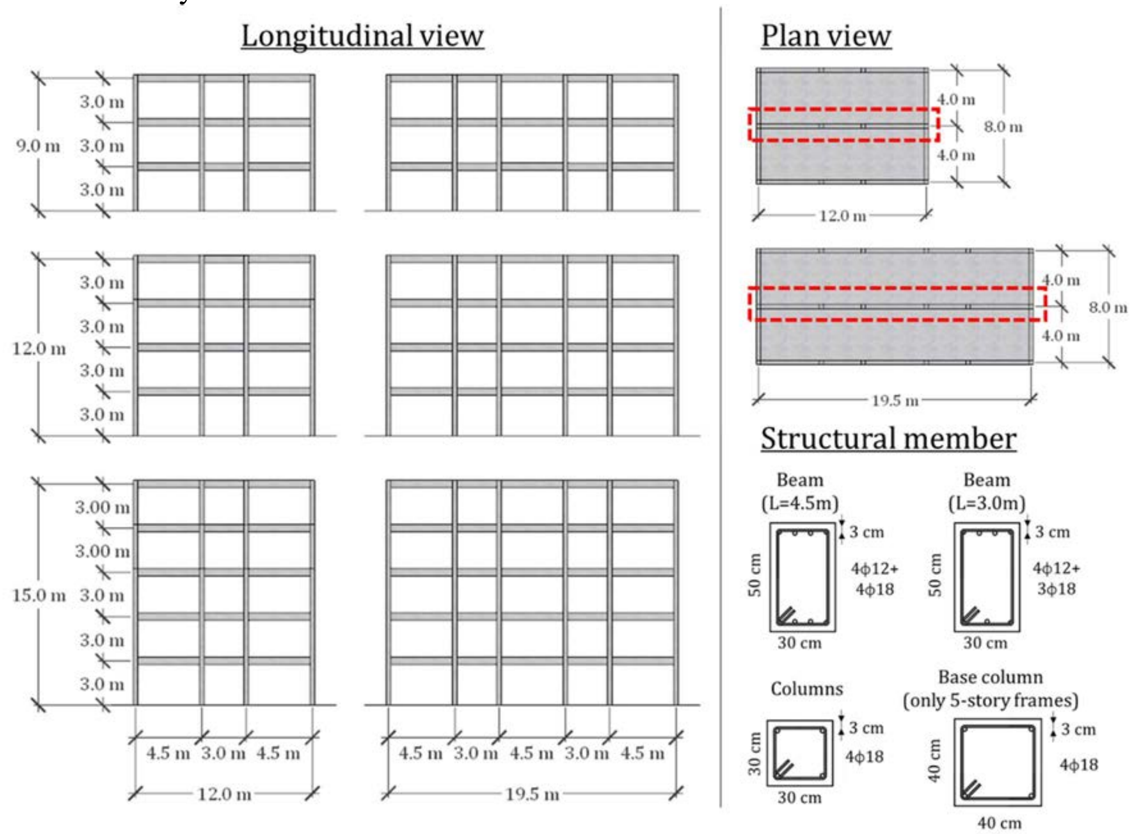

Figure 7: Plan view (upper right) of the case-study frame structures and geometry of the analysed frames (left); geometrical and reinforcement details of the RC structural members (lower right).

Geometrical and reinforcement details of the structural members are selected similarly to past experimental investigations available in the literature ([33]) and are reported in Figure 7 (lower right). For the sake of simplicity, the same column cross-section is considered for all the stories, except for the base columns of the 5-storey frames (both 3 and 5-bays), i.e. 40x40cm. Transversal reinforcement is ϕ6/15 and ϕ8/15 for beams and columns, respectively. Beams' longitudinal reinforcement consists of plain round bars with hooked ends. Yet, no stirrups are provided in the joint panel. Concerning material mechanical properties, the mean concrete cylindrical strength is $f_c' = 14.41\ MPa$, while the mean steel yield stress is equal to $f_y' = 340.51\ MPa$.

The masses considered are reported in Table 2.

FLOOR PLAN	INTERSTOREY MASS	ROOF MASS
3 BAYS	66.8 ton	44.58 ton
5 BAYS	108.6 ton	72.5 ton

Table 2: Masses of the case-study buildings relative to a single frame.

4.1 Modeling Approach

Numerical pushover analyses are carried out by implementing a two-dimensional (2D) lumped plasticity model in the structural software Ruaumoko ([34]). For the sake of simplicity, floor diaphragms are assumed rigid in their plane and the soil-structure interaction is neglected (i.e., fixed base foundation connection nodes are assumed). The structural RC members are modelled by Giberson elements, i.e. mono-dimensional elastic elements with plastic hinges at the end sections. Beam plastic hinges are defined through bi-linear moment-curvature relationships and plastic hinge lengths are evaluated according to [35]. For columns' plastic hinges, an axial load-moment interaction diagram is implemented. Shear failure mechanisms and flexural/shear interaction are also evaluated, according to [4]. Moreover, the shear capacity of joint panels is also accounted for in the numerical simulation by defining additional nonlinear rotational springs in the panel zones, as suggested in Pampanin et al. [36]). More specifically, equivalent column moment versus drift relationships (derived following the NZSEE [4] provisions) characterize the nonlinear behaviour of the rotational springs. Furthermore, the influence of the axial load on the joint capacity is also considered by implementing an axial load-moment interaction diagram. Finally, linear strength degradation is considered for all RC members and beam-column joints. Specifically, it is assumed that the moment capacity is equal to zero when a deformation equal to twice the ultimate capacity of the member is achieved (as in [37]). The force profile used for the non-linear analyses is defined as proportional to the buildings' masses.

The moment-curvature relationship is computed using a stress-block constitutive model for the unconfined concrete and an elastoplastic stress-strain model for the reinforcement, as defined by the Italian building code [29]. For sake of consistency, the same assumptions are made when considering the capacity of the element during the analytical procedures.

5 RESULTS AND DISCUSSION

In this section, the results of the non-linear static analyses of both the proposed analytical procedures and the numerical simulations are presented and compared to each other. Firstly, the curves obtained from the numerical models are compared with the analytical one in terms of base shear and displacement at the effective height, and then the obtained values of the safety (IS-V) and loss (PAM) indexes are compared. The comparisons are carried out for both the undamaged structure and the damaged one, considering three alternative initial damage states.

5.1 Non-Linear Static Analyses

Non-linear static analyses (either SLaMA or numerical) are performed considering the structure in its undamaged and damaged configurations. To implement the study, four damage states, namely DS1 to DS4, are chosen as representative of slight, moderate, extensive, and complete damage. Building-level DS thresholds are herein considered, defined as a function of the yielding (Δ_y) and ultimate (Δ_u) displacement of the equivalent Single-Degree-of-Freedom (SDoF) curve, following the methodology proposed by [38] (Figure **8**). This methodology is similar to the one proposed by Lagomarsino & Giovinazzi [39] but ensures that damage points are equally spaced in terms of displacement. In the case of numerical non-linear static analyses, the results in terms of storey displacement are extrapolated and then turned into an equivalent SDoF curve using the displacement profile obtained at the ultimate displacement and using the formulation proposed by Priestley et al. [35]. The latter is defined as the onset of ultimate deformation inside the first structural element and is also considered as the performance point for the safety assessment. Meanwhile, the yielding displacement is defined as the deformation that leads to yielding rotation in the first structural element of the frame.

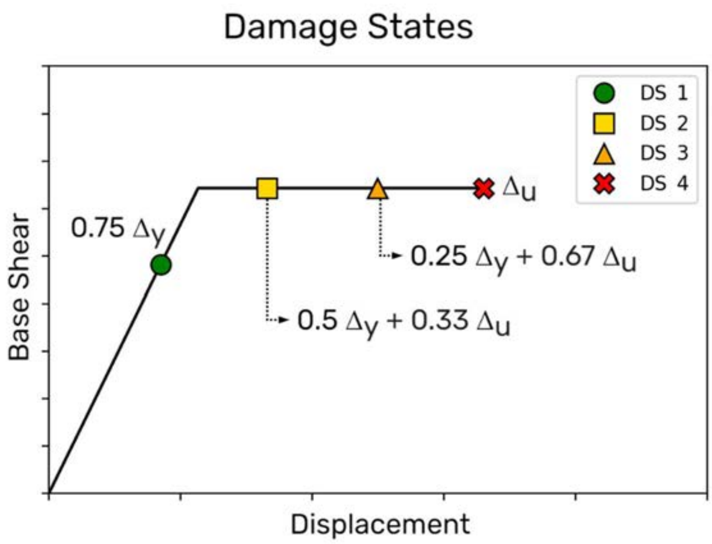

Figure 8: Definition of damage states on the global pushover curve.

For all the above-mentioned cases, the numerical force-displacement curve of the undamaged frame is plotted against the analytical prediction computed using both the classical SLaMA procedure and the proposed modified one (Figure 9).

The comparison between the numerical and analytical approaches shows a relatively good agreement in the results in terms of ultimate strength and deformation capacity for all the methodologies. Nevertheless, looking at the results shown in Figure 9, it can be noted how effective the proposed refined SLaMA methodology is in estimating the elastic stiffness of the numerical curve, especially for the 3-storey and 4-storey buildings, if compared to the classical SLaMA method. However, a significant discrepancy is observed in the 5-storey case studies, mainly due to the presence of stronger column sections on the first floor. Therefore, the deformed shape of the frames deviates from the assumption of a linear displacement profile, and the building is not able to develop a global mechanism as predicted by the analytical procedure.

Despite the criticalities that arise in higher and less regular buildings, the modified approach represents a great improvement over the regular SLaMA procedure requiring minimal additional computation.

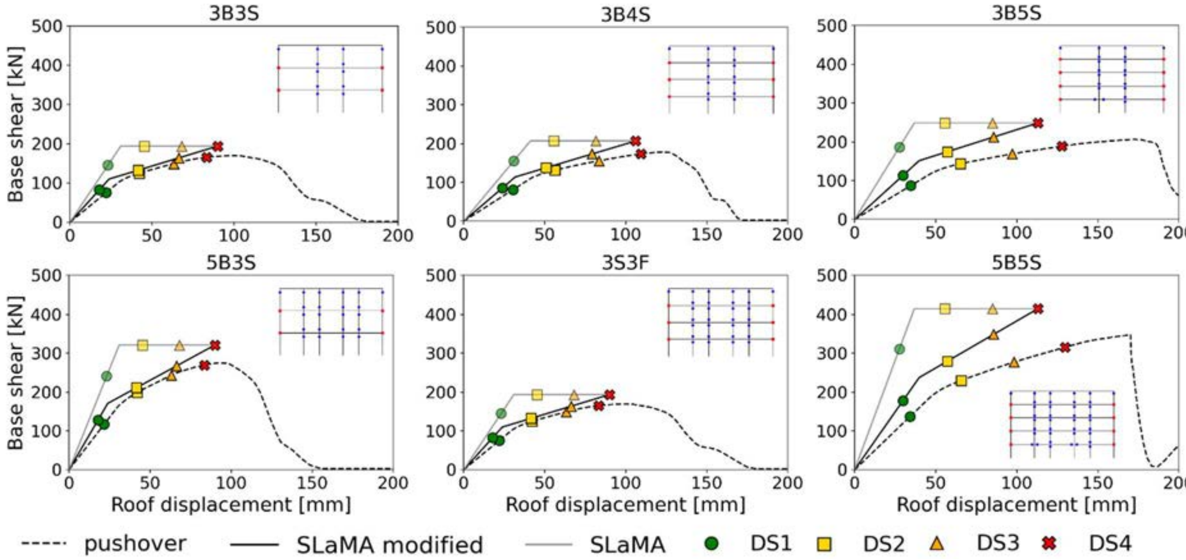

Figure 9: Comparison between force-displacement curves obtained by SLaMA, SLaMA modified and numerical model.

Following the methodologies for the evaluation of damaged frames using the FEMA 306 approach [6], the damaged capacity curves of the frame were computed for the DS1, DS2, and DS3 damage levels (Figure 10). Initial DS4 is not considered since it represents a complete damage scenario.

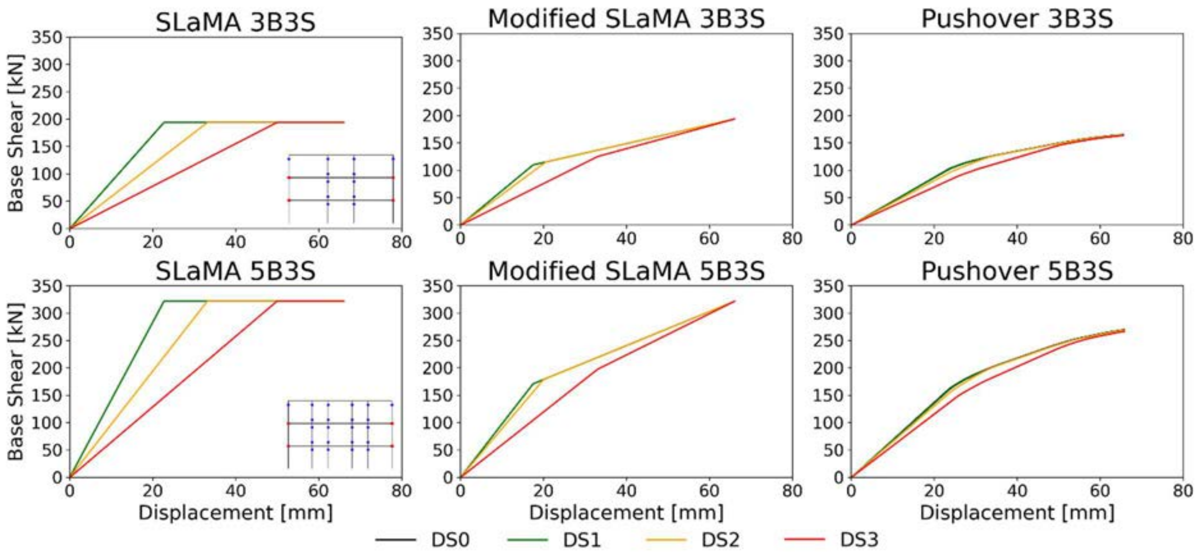

Figure 10: Comparison of the capacity curves obtained using SLaMA, SLaMA modified, and numerical models following the FEMA 306 approach.

The results show how the DS1 does not influence the capacity curve in any of the approaches; this is due to the low value of displacement associated with the damage state.

For the initial DS2, the classical SLaMA procedure exhibits a great reduction in stiffness which is not representative of the damaged curve as shown by the numerical results. Yet, the SLaMA modified approach, for this DS, in some cases can lead to a yielding point which is above the original curve. This issue sometimes arises also when bi-linearizing the damaged numerical curves using an equal-area approach. For these reasons, the decision was made to limit the base shear of the damaged curve to the intersection with the plastic branch of the original curve.

Finally, the DS3 leads for both analytical cases to a greater reduction in stiffness than the numerical approach. However, the stiffness of the curve obtained following the modified SLaMA approach is alike the one computed through the numerical model.

The SLaMA capacity curves tend to approximate a secant-reloading approach similar to the procedure proposed by [40]: this is because the stiffness of the SLaMA curve is strongly dominated by the weakest element, in this case, the external joints, which follows a secant-reloading damage model. This does not happen using the modified SLaMA approach because it can grasp how the reduced stiffness of one element is mitigated by all the surrounding undamaged elements which maintain their original capacity.

5.2 PAM ISV

The %IS-V safety index and PAM losses are finally computed for all the case studies and all the considered initial DSs.

The %IS-V is computed following a capacity-spectrum method approach for both the numerical and analytical capacity curves. For the modified SLaMA approach and the numerical curve, the procedure described by the Italian Building Code ([29]) was used to compute the damping because it considers the hardening behaviour of the capacity curve. For the SLaMA approach, the damping was computed following the NZSEE guidelines [4] because the Italian approach leads to a great overestimation of the performance.

The results (Figure 11) reveal how the value of the safety index computed using the modified approach is not only closer to the one computed using the capacity curves from the numerical models but also manifests a consistent decrease in the safety index relative to the numerical approach. Meanwhile, the classical SLaMA approach shows a greater reduction of the safety index for more severe initial damage states tight to the larger reduction in ductility than the ones obtained by the other analytical and numerical approaches.

Moreover, the modified analytical performance tends to slightly overestimate the safety index for the 3-storey buildings and underestimate it for the 5-storey ones: even if the procedure yields a larger base shear value in the case of higher buildings, the approach does also return a lower displacement averaging out the inaccuracies.

To compute the PAM index, it is fundamental to address the issue regarding the choice of serviceability limit state (SLD in the Italian Building Code) performance point on a damaged capacity curve. In fact, using the newly defined yielding point for damaged capacity curves as the SLD performance point could lead to a better performance than the undamaged one (as highlighted in [14]). For this reason, the displacement at which the SLD performance is evaluated always corresponds to the displacement that leads the undamaged structure to the onset of the serviceability limit state (as in [41]).

The serviceability performance point has been defined as the yielding point for the analytical approaches (SLaMA and SLaMA modified) and as the point on the capacity curve corresponding to the yielding of the first element in the numerical model.

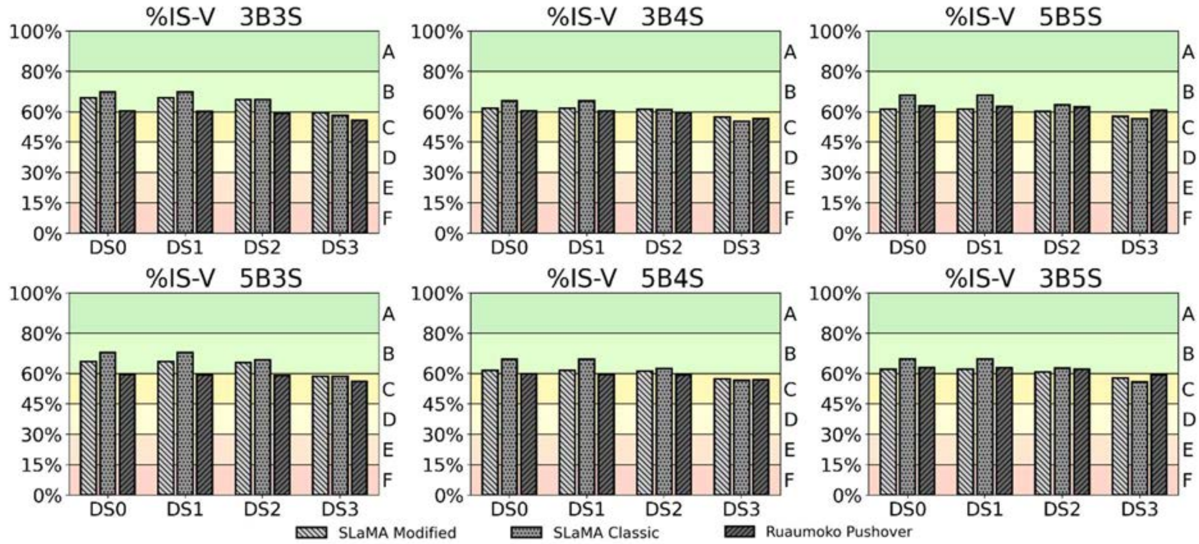

Figure 11: Values of %IS-V safety index for all the case studies and all the damage states, shown with performance classes defined by [27], higher is better (Note: 3B3S=3-bay 3-storey frame).

The results (Figure 12) show how the EAL index computed using the classical SLaMA approach leads to significantly lower values than the other approaches. This is due to a general overestimation of the seismic performance of the frames for lower, yet more frequent, earthquake intensities. Meanwhile, the modified approach is way more effective in grasping the performance of the case-study buildings in comparison with the numerical model.

The classical SLaMA method tends to largely overestimate the relative increase of the EAL with increasing levels of initial damage: this is related to the greatly reducing stiffness of the capacity curve as shown in Figure 3. However, the results displayed by the modified analytical approach, even if they show a lower discrepancy, do not exhibit a consistent increase with the numerical models for higher damage levels on 5-storey case studies.

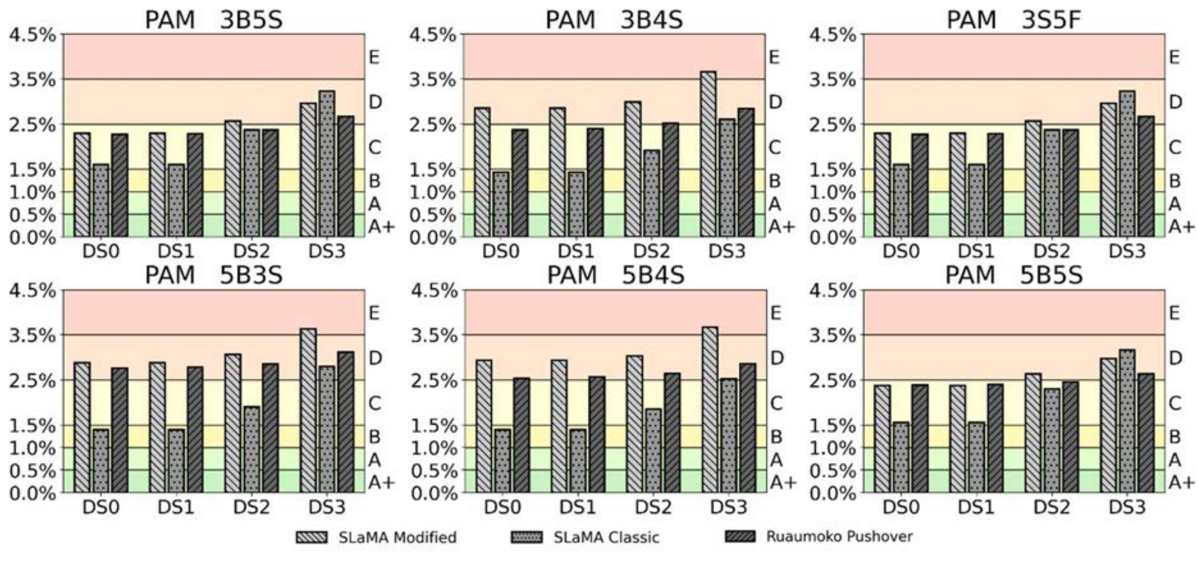

Figure 12: Values of PAM loss index for all the case studies and all the damage states, shown with performance classes defined by [27], lower is better.

6 REPAIR/RETROFIT/RECOVERING STRATEGY

Following the assessment of the residual capacity of damaged buildings, the decision between repair/retrofit or demolition must be performed. Considering that usually, the damaged buildings reveal an already poor performance in their un-damaged state, a simple repair could result not feasible or cost-effective. Moreover, the repair of the damaged structural components of RC frames, even for new up-to-date code-compliant buildings, yields poor performance because is usually not possible to recover the original stiffness of the element ([18]). For this reason, in recent earthquakes, in many cases, the repair of buildings was deemed not cost-effective, leading to the complete demolition and reconstruction of modern buildings such as the PwC Building following the 2010-2011 Canterbury earthquake sequence ([42]).

However, the demolition and reconstruction process is neither cheap nor fast, since it can lead to great expenses related to downtime, especially for commercial real estate, and/or to a larger CO2 output related to the disposal of the old building and the construction of the new one.

A possible alternative solution to this issue can be the implementation of high-performance precast exoskeletons. This technology of external systems would be able to not only restore the lost capacity of the building in terms of strength/stiffness but could also improve the seismic performance of the building to IS-V=100%. Assuming that all the damaged elements can withstand supplemental cycles and the residual ductility as estimated by FEMA 306 [6] approach, this solution can bring the capacity of the retrofitted structure to building-code requirements. Clearly, this approach would be only feasible for relatively low damage states because the uncertainties related to higher damage levels make the response not reliable enough.

The proposed retrofit strategy is achieved with the use of external frames that make use of the PRESSS (Precast Seismic Structural System) technology ([43-45]) (Figure 13). Developed in the 1990s, the PRESSS structural system is a solution for frames and walls that relies on coupling post-tension unbonded tendons with supplemental damping devices to provide energy dissipation while ensuring a small residual deformation after a major earthquake.

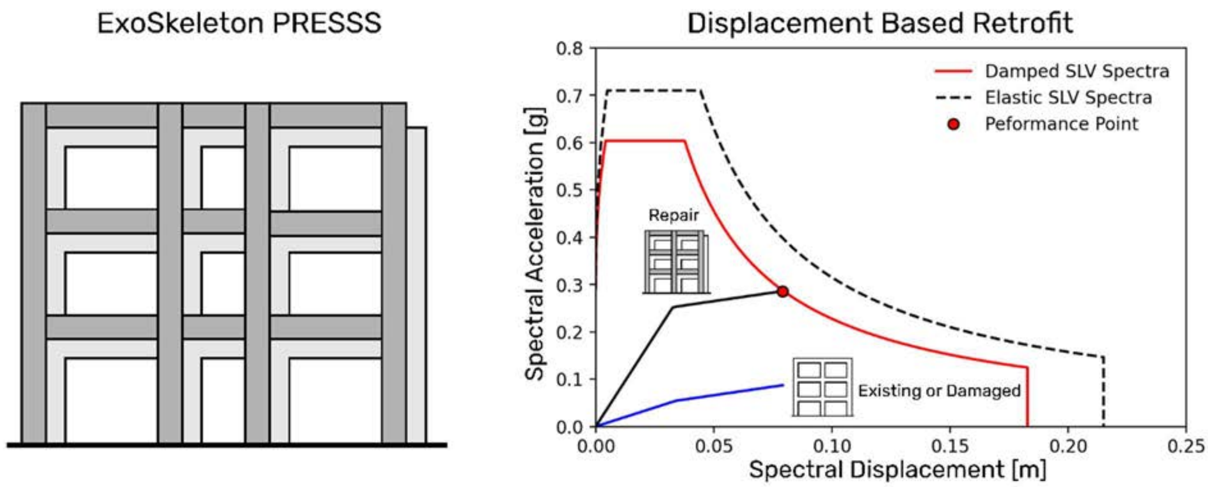

Figure 13: Overview of the repair technique and Displacement Based Retrofit design curve.

In this paper, for illustrative application, only the design phase is reported in terms of base shears of the proposed solution computed using the Displacement Based Retrofit (DBR) ([46-48]) procedure. The DBR is an adaptation of the Direct Displacement-Based Design for new buildings [35,49].

The procedure is performed as follows:

- Selection of the target displacement, in this case corresponding to the onset of DS4, the same one considered for the calculation of %IS-V index.
- Definition of the equivalent SDoF based on the inelastic mode displacement profile of the exoskeleton-retrofitted structure.
- Evaluation of damping considering the weighted average on base shears between the frame's original damping and the exoskeleton damping.
- Computation of the effective period from the design displacement and the damped displacement response spectra.
- Computation of the base shear demand of the exoskeleton-building system using the effective stiffness, obtained by the effective period, and design displacement.
- Definition of the exoskeleton demand by removing from the design base shear the residual capacity of the damaged building.

The target displacement is chosen as the ultimate displacement of the existing frame, and an IS-V=100% index is set as the performance target of the retrofitted solution to make the building code-compliant (Figure 13).

The proposed solution is herein applied only to the 5-storey case studies using the residual capacity curves obtained through the proposed modified SLaMA-based procedure for all the investigated damage states.

The exoskeleton is designed to achieve a 1.75 re-centring ratio (usually referred to as λ and defined as the ratio of the moment contribution of post-tensioned members and mild steel dissipaters; [50]) and is composed of 50x30cm Reinforced Concrete Precast beams and columns. The results are discussed in terms base shear of the exoskeleton solution and of the loss index computed on the design capacity curve.

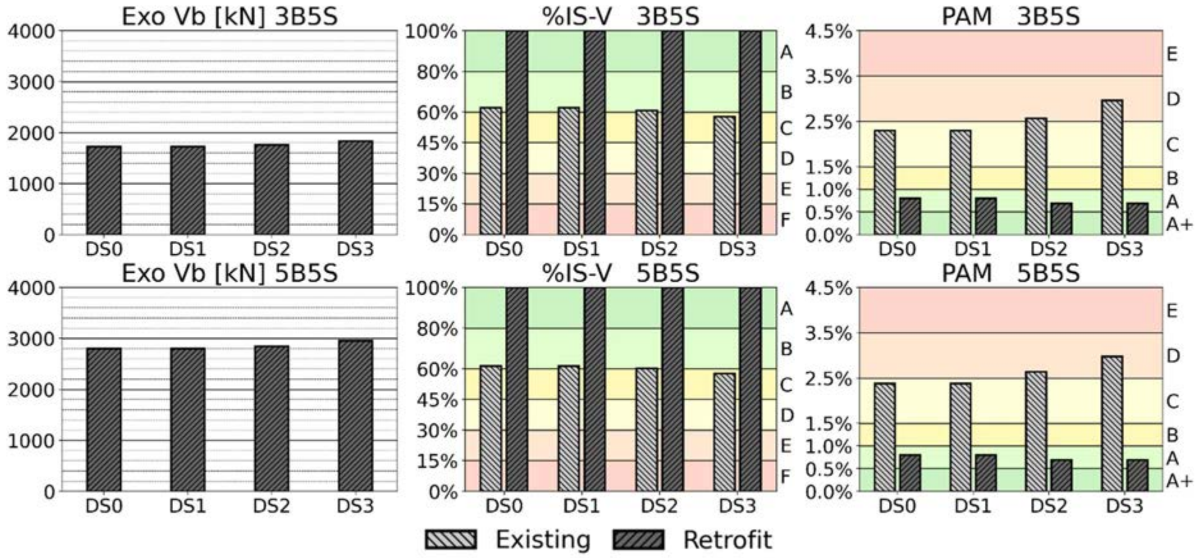

Figure 14: Results of the DBR procedure for the 5-storey case studies considering all the different damage states expressed as base shear demand for the exoskeleton solution for the whole building (left), %IS-V safety index (centre), and PAM loss index (right).

The results (Figure **14**) show how, according to the design capacity curve, the solution is able to increase the performance in terms of losses as well as safety, bringing the PAM index below 1%. Moreover, results exhibit how more severe initial damage states lead to a small increase in the demand for the retrofit solution, which is around 5% between the undamaged building and the DS4. In fact, the loss of safety of the damaged building is related to the reduction of ductility which leads to a reduction of damping, worsening the demand. With the addition of the exoskeleton, the structure is able to recover a good level of damping reducing the original demand and then reducing the differences in demand between damage levels.

7 CONCLUSIONS

In this paper, the effectiveness of a proposed analytical approach combining the SLaMA procedure and FEMA 306 methodology [6] has been investigated and a modification has been proposed to address the shortcomings of the original approach. The results were compared with non-linear static pushovers to evaluate the reliability of the proposed procedure in terms of safety index and expected annual losses. Finally, a repair/retrofit solution has been proposed to address the issue of the repair of damaged RC elements.

A wide range of case studies has been analysed following the FEMA 306 approach using the SLaMA simplified analytical approach to assess the performance of the damaged structure in terms of safety and losses for three different levels of damage. The results were compared against a numerical model developed in Ruaumoko following the FEMA 306 approach. Results have shown how the original SLaMA methodology is unable to predict accurately the performance of the damaged structure because is unable to predict accurately the elastic stiffness of the RC frame. For this reason, a modification to the SLaMA approach has been proposed to better estimate the yielding stiffness of the structure.

The proposed modification regards the sub-assembly evaluation step and is designed to better estimate the stiffness of the subassemblies that compose the frame. This step, together with an alternative methodology to better grasp the different levels of engagements in the resisting mechanism of the various elements of the frame at lower drifts, makes the evaluation of the yielding point of the capacity curve much more accurate. This procedure can yield better results with minimal additional computation to keep the procedure user-friendly.

Implementing such procedure, the results show a satisfying degree of accuracy and precision as the newly developed methodology is not only able to better estimate the capacity of the undamaged frame but also exhibits a higher degree of consistency regarding the reduction of the performances related to damage when compared against the numerical model's results.

Furthermore, a repair/retrofit strategy and solution making use of high-performing precast RC external exoskeletons have been proposed. The solution is conceptually able to not only restore the original capacity but also bring the performance to building-code regulations. Furthermore, the solution is able to drastically reduce seismic-related losses. Anyway, this proposed solution needs to be further investigated through non-linear static and dynamic analyses to prove its effectiveness.

To further evaluate the reliability of the procedure and the feasibility of the proposed solution, a probabilistic approach that takes into account the uncertainties related to the damage level of the building is worth to be investigated.

ACKNOWLEDGEMENTS

The authors want to acknowledge the financial support of the Italian Ministry of University and Research (MUR) for funding of the Doctoral Scholarship of Michele Matteoni, Livio Pedone and Simone D'Amore. The authors also acknowledge the support of the DPC-ReLUIS 2019-2021 research project.

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