RISK ASSESSMENT OF EC8-COMPLIANT STEEL BUILDINGS

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Abstract

The main objective of this paper is to understand and characterize the level of expected earthquake-induced economic losses of steel buildings designed to Eurocode 8. This is attained through the use of a comprehensive population of case-study buildings that considers different lateral load resisting systems, plan configurations and building heights. Different locations of low and high seismicity, reflecting the majority of the European seismic reality, are considered. The results indicate that EC8-compliant steel moment-resisting frame (MRF) buildings and concentrically-braced frame (CBF) buildings generally exhibit good seismic performance in terms of mean annual frequency of reaching collapse. Although all buildings showed good performance against collapse, when compared to the requirements present in literature, clear indications on a lower performance of chevron-braced CBFs are observed. Furthermore, low expected annual losses (EALs) are recorded across the population of steel buildings considered, with a predominant contribution of losses coming from structural and non-structural damage. Based on the findings reported in this paper, different insights towards the incorporation of loss performance objectives in the European seismic design code are provided.

Keywords: steel moment and braced frames, Eurocode 8, seismic losses, FEMA P-58

1. Introduction

Performance-Based Earthquake Engineering (PBEE) [1][2] aims to design structures with predictable seismic performance whilst accommodating multiple performance objectives. PBEE has experienced a continuous evolution since its inception, being presently incorporated across several existing seismic design and assessment guidelines. In the case of Part 1 of Eurocode 8 (EC8-1) [3], for example, the key performance objectives for the seismic design of new buildings are the: i) protection of human life, by avoiding the development of unstable collapse mechanisms; and ii) limitation of drift-sensitive non-structural damage. Recent earthquakes, however, have shown the limitations of such an "essentially-life-saving" approach in modern societies, in which earthquake-induced direct (e.g. structural and non-structural damage, deaths) and indirect (e.g. business disruption) losses might be socio-economically strenuous [4][5].

Nowadays, it is possible to express seismic performance through broader metrics of greater meaning to building stakeholders (e.g. collapse risk, expected annual losses or EAL, business downtime). This approach, which can be implemented following the FEMA P-58 [6] framework, potentially facilitates the process of decision making (e.g. lifetime-analysis-based definition of optimum design/retrofit solutions, regional/national prioritized fund allocation for seismic risk reduction, stipulation of insurance premiums). Recently, this philosophy has inspired the development of the EAL-based Italian rating scheme for seismic risk classification of existing buildings [7] and its use for seismic design purposes could also be fruitful. In particular, economic-loss-related performance objectives could potentially address the issues raised above. This shift can initially be supported by a clear understanding of the expected loss levels of current code-compliant buildings, in order to identify latent areas in need of improvement. Such baseline could, for example, support new or more stringent performance objectives.

This research study aims to address the limitations in the available literature by characterizing the expected levels of direct economic losses of EC8-compliant steel buildings. Particular attention is paid to a realistic quantification of repair-related losses in the European context, using FEMA P-58, in a broad and consistent manner. In order to do so, a large population of synthetic building archetypes is defined, incorporating different lateral load resisting systems (LLRS), regular plan configurations and building heights. Different regions of low and high seismicity in Portugal, whose seismic hazard reflects the majority of the European seismic reality, are adopted. State-of-the-art modelling procedures are employed to simulate the buildings' seismic response and characterize their performance using several metrics. The results obtained from this study should enable the identification of ways to improve the current code-based European seismic design framework, towards a more explicit consideration of economic-loss-related performance objectives.

2. Earthquake-induced loss assessment framework

In this study, seismic economic losses were computed following the building-specific story-based loss estimation methodology proposed by Ramirez et al. [8], considering the effect of demolition-related losses [9], as expressed in Equation (1).

$$E[L_T|IM]=E[L_T|NC\cap R,IM]\cdot\{1-P(D|NC,IM)\}\cdot\{1-P(C|IM)\}+E[L_T|NC\cap D]\cdot P(D|NC,IM)\cdot\{1-P(C|IM)\}+E[L_T|C]\cdot P(C|IM) \tag{1}$$

Loss assessments were herein conducted by subjecting a given structure to increasing intensity levels, using Incremental Dynamic Analysis (IDA), employing a hunt-and-fill algorithm [10]. The ground motion record suites detailed in a later section of this paper were employed for the IDA process. Every dynamic analysis was extended by 5 seconds of free vibration time and the residual deformations of each story were computed by averaging the lateral deformations of the building within this period [11]. The 2%-damped spectral acceleration at the fundamental period, $S_a(T_1,2\%)$, was adopted as intensity measure (IM). From the IDA analyses, the resulting curves were interpreted to pinpoint the exceedance of the collapse limit state, herein assumed as the instance associated with either a reduction to less than 20% of the initial slope of the

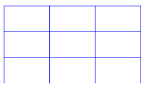
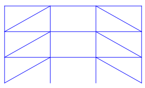
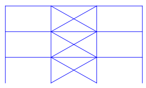
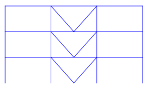
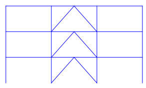
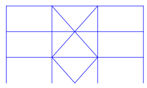
IDA curve or an inter-story drift ratio, ISDR, above 10% [12]. A collapse fragility curve was then constructed by fitting a lognormal cumulative distribution function, defined by a median value, θ, and a lognormal standard deviation, β. The β parameter, reflecting the record-to-record (i.e. aleatory) uncertainty, β_{RTR}, was combined with the modelling (i.e. epistemic) uncertainty, β_{MDL} [13], taken here as 0.2 [12]. Assuming that the two uncertainties independently follow a lognormal distribution, the total uncertainty, β_{TOT}, was quantified through the root sum squared of β_{RTR} and β_{MDL}. In order to compute the demolition-related losses, and following the recommendations by Jayaram et al. [14] in the context of steel buildings, the probability of demolition given a residual inter-story drift ratio, P(D|RISDR), was assumed to follow a lognormal distribution with a median of 1.85% and a logarithmic standard deviation of 0.3. This particular loss quantity was assumed to be equal to the total replacement cost of the building without any potential demolition costs, following the recommendations by Hwang and Lignos [15].

Regarding the repair-related seismic losses, the story-level DV-EDP (decision variable versus engineering demand parameter) loss functions proposed by Papadopoulos et al. [16] have been adopted. According to the authors, these story loss functions are compatible with the more refined (i.e. component-based) FEMA P-58 [6] approach. Depending on story and total building area, the authors propose DV-EDP relationships for: i) X-braced building structural components; ii) chevron-braced building structural components; iii) moment-resisting building structural components; iv) column base plates, v) drift-sensitive typical floor non-structural components; vi) acceleration-sensitive typical floor and roof floor non-structural components; and vii) low/medium/high luxury contents. Any further refinements of the proposed loss functions, perhaps considering more recent seismic fragility characterizations of structural and non-structural components, were not considered in the present study. The derived loss curves are meant to represent modern Eurocode designed steel low/mid-rise office buildings and floor plans typically employed in Greece. Assuming similar building inventories, the aforementioned loss curves can be adjusted for application to different countries and years, provided that its shape is preserved (i.e. assuming a modified C_{max} value, which corresponds to the total replacement cost of the component group). For the context of this study, it was assumed that the derived loss curves are representative of the Portuguese case-study steel building stock. The loss functions were therefore converted to the specific European context (Portugal), from the US-based reality associated with the FEMA P-58 database of repair costs, using the conversion approach proposed by Silva et al. [17]. This approach allows for a practical economy-analysis-based repair cost conversion of FEMA-P58 repair costs, most notably accounting for the differences in material and labor costs between the US and any European country of interest, to Europe. The specific conversion factors adopted for each of the story loss functions used in this study are reported in [17]. Finally, the economic building value, used to normalize the repair costs obtained with the Portuguese-converted story loss functions, was defined based on the construction cost of new steel buildings in Portugal. On the basis of a survey consultation with a number of Portuguese civil engineers, the total construction cost of steel office buildings was quantified at 1200€/m^2.

3. Archetype population definition and design

In order to properly assess the earthquake-induced losses associated with code-compliant steel buildings, a comprehensive group of archetypes was defined. This was attained through the seismic design to Part 1 of Eurocode 8 (EC8-1) [3] of a portfolio of 72 synthetic steel frame buildings. The archetype population adopted in this study follows the structural designs reported in past literature [11][18]. To what concerns the lateral load resisting systems (LLRS), both moment-resisting and concentrically-braced steel frame configurations permitted by EC8-1 were adopted in this study, as summarized in Table 1. All seismic-resistant frames were designed to resist gravity and seismic loads according to Part 1-1 of Eurocode 3 (EC3-1-1) [19] and EC8-1 [3]. No additional load scenarios (e.g. wind, fire) were considered during the structural design process. All beam member designs were performed neglecting composite action provided by the slab, as permitted by the European code. Steel sections available in the European construction market were adopted for the main structural members: i) for MRFs, IPE and HEB open-section profiles were adopted for the beams and columns, respectively; and ii) for CBFs, IPE/HEA and HEB/HEM sections were adopted for the beams and columns, respectively, whilst commercial circular hollow sections were adopted for the braces. Column and beam sections were changed at every 2-3 stories, whilst brace sections were changed at every story. Regarding the nominal steel grades adopted: i) for MRFs, all structural members were designed using an S275 steel grade; and ii) for CBFs, an S275 steel grade was adopted for the beams and diagonals, whilst S355 was used for the columns. In the case of CBF buildings, all diagonal-to-beam-column gusset plate connections were also seismically designed, using an S275 steel grade, a 30º Whitmore angle and an elliptical clearance model. Brace-to-beam gusset plates located in chevron CBFs were designed with similar conditions, with the exception of the use of a linear clearance model. All seismic designs were conducted for medium ductility class (DCM) defined in the European code, adopting a behavior factor of 4 for MRFs and CBFs with diagonal bracings, and 2 for chevron archetypes.

Table 1 – Moment-resisting and concentrically-braced steel frame configurations considered

Moment-resisting	Concentrically-braced				
	Single-diagonals	X-diagonals	V-diagonals	Inverted-V-diagonals	Split-X-diagonals
MRF	CBF-SD	CBF-X	CBF-V	CBF-IV	CBF-V_IV
					

The current study was conducted with the consideration of regions of both low and high seismicity, thus reflecting different levels of the European seismic hazard reality. In this context, the population of building archetypes was designed for two seismic locations in Portugal, namely Porto (PGA_{475y}=0.1g) and Lagos (PGA_{475y}=0.3g). Regarding the adopted plan configurations, variations of floor area and number of bays in the transverse and longitudinal directions were considered, as described in Table 2. In both plan configurations considered in this study, the longitudinal LLRS is composed by two external frames for the CBF buildings and by all longitudinal frames for the MRF buildings. This differentiation reflects commonly adopted design and construction practices in the case-study country. In the case of CBF buildings, all internal longitudinal frames were assumed to provide no lateral-load resistance to the building, serving only as gravity-carrying systems. The focus of the research study was on the longitudinal direction of the buildings. The beam spans adopted are consistent with the current engineering practice in Portugal. To what concerns the building heights, a range of number of stories that is representative of the Portuguese steel building stock was considered. Specifically, archetypes with 3, 5 and 8 stories were adopted herein. Regarding the story heights, all buildings were set to have a 1st-story height of 4.5m, and 3.5m for the remaining stories. This differentiation reflects typical Portuguese residential and office steel buildings, in which the ground floor is generally used for commercial activities.

Table 2 – Summary of longitudinal and transversal spans (dimensions in meters)

Archetype group	Plan configuration			
	A		B	
	Overall dimensions *	Span widths **	Overall dimensions *	Span widths **
MRF	18 by 12	{6+6+6} by {6+6}	24 by 18	{6+6+6+6} by {6+6+6}
CBF				{4.5+5+5+5+4.5} by {6+6+6}

Notes
* longitudinal by transversal building plan dimension
** {beam spans in the longitudinal frames} by {beam spans in the transversal frames}

4. Ground motion records and numerical modelling

As previously detailed, the synthetic archetype population was assumed to be located in different seismicity regions in mainland Portugal. Probabilistic Seismic Hazard Analysis (PSHA) was performed for the sites in question, using the open source software OpenQuake [20] and the seismic hazard model of the SHARE project [21] with the modifications made by Silva et al. [22]. Disaggregation of the seismic hazard [23] on magnitude, distance and ε was performed, which served as the basis for the ground motion record selection scheme and was used in conjunction with the average shear wave velocity for the first 30 meters of soil, vs,30. For all locations, a suite of 30 ground motion records was selected and scaled so that the 2%-damped median spectrum of the suite matched the prescribed response spectrum of EC8-1 for a return period of 475 years, considering a wide range of periods of interest (between 0.05s and 4s). This approach, which is similar to the one adopted in FEMA P695 [12], was attained with SelEQ [24].

In order to realistically simulate the seismic response up to collapse of the steel buildings considered in this study, state-of-the-art OpenSees [25] 2D-based numerical modelling procedures were adopted. For MRFs, the considered modelling approach was based on the recent work of Hwang and Lignos [15]. Beams and columns were modelled through the use of elastic elements connecting concentrated plasticity rotational springs simulating the inelastic response of beam and column members [26], representing the effects of strength and stiffness deterioration associated with plastic hinging. To accurately capture the development of global second-order effects on both perimeter and interior seismic frames, all longitudinal frames were explicitly simulated in the 2D model via rigid-link-connected frames. To what concerns the modelling of the CBF buildings, the models adopted in this study were based on the recommendations of Karamanci and Lignos [27]. For the beams and columns, the same modelling procedure to the one described before for MRFs was adopted [26]. For the braces, displacement-based fiber-discretized elements, with the consideration of ultimate fracture attributable to low-cycle fatigue, were used [27]. Brace-to-frame connections were modelled using the recommendations of Hsiao et al. [28]. Beam-to-column shear tab connections were simulated by fully-pinned connections, following the conclusions of Mohsenzadeh and Wiebe [29]. It should be noted, however, that in beam-column joints in which a diagonal-to-frame gusset plate is present, the possibility of plastic hinging on the beam, which typically occurs away from the physical beam-to-column shear tab connection, was considered in the model [26]. In order to realistically capture the development of sidesway collapse mechanisms [29][30], gravity framing of the CBF buildings was explicitly modelled via rigid-link-connected frames. Fig. 1 illustrates the MRF and CBF modelling approaches considered in this study.

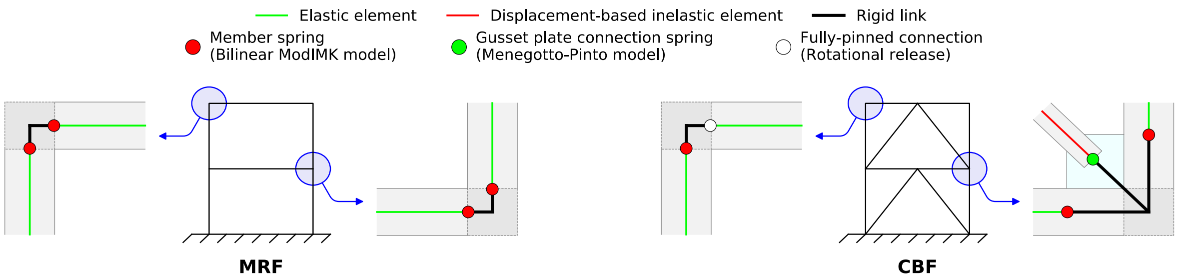


Fig. 1 – Schematic overview of the numerical modelling approaches considered [31]

All frames were modelled as fixed at the base and global second-order effects were explicitly considered via a corotational transformation. Rayleigh damping was adopted by assigning a damping coefficient of 2% [32], assigned to the first and one of the last significant modes of the structure, corresponding to the number of stories [33]. This was attained to avoid underdamping or overdamping of modes of possible interest, by "stretching" the Rayleigh damping model asymptote to ensure similar levels of damping across the range of modes considered. The stiffness component of the damping model was computed only for the elastic elements of the model, thus avoiding large artificial damping forces [26][28]. Based on the recommendations of NIST GCR 17-917-45 [34], the nominal steel yield strength, $f_{y,nom}$, of all modelled components, was adjusted to its effective value, $f_{y,eff}$,, by a multiplication factor of 1.151.25 (to capture cyclic hardening effects and material overstrength).

5. Earthquake-induced risk

In the context of this study, earthquake-induced risk was also computed through different metrics, as discussed in the following paragraphs. Firstly, the probability of collapse, P_c, computed through the mean annual frequency of collapse, λ_c, was employed to interpret the collapse risk through integration of the collapse fragility curve and the seismic hazard at the site. Secondly, the computation of the expected annual loss, EAL, was also carried out. This approach can be applied to the total losses but also to the different loss contributors (e.g. repair, demolition, collapse). In addition to EALs, the expected present value (PV) of lifecycle costs, using a 5% discount rate and a 50-year lifetime, were also computed.

Based on the computation of collapse risk, the 50-year collapse probabilities ($P_{c,50}$), across the entire building population considered in this study, were obtained. These results are summarized in Fig. 2, across the different LLRS typologies, building plan configurations and seismic locations. In the figure, the acceptable risk objective specified by ASCE 7-16 [35] is also shown. Fig. 2 reveals that, in general, low collapse risk levels were associated with the EC8-compliant building population. On the other hand, structures located in the region of high-seismicity (Lagos) exhibited worse collapse performance when compared to the low-seismicity cases (i.e. located in Porto), with the uppermost $P_{c,50}$ observed of around 0.25%. Still, such collapse risk compares well against the ASCE 7-16 specified limit of 1%. This denotes the generally good collapse performance of the steel buildings considered in this study, likely suggesting that EC8-compliant steel MRF and CBF buildings have acceptable performance against collapse. Since the primary objective of the European code is set at the protection of human life through the collapse prevention, one may thus infer that steel MRF and CBF designs resulting from the existing recommendations of the code meet this performance objective.

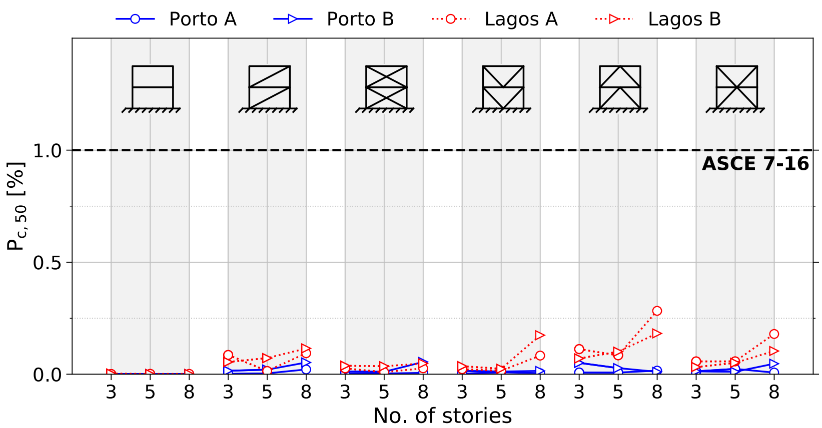


Fig. 2 – Summary of collapse risk results [31]

A more detailed look into Fig. 2 allows observing that the obtained collapse probabilities were largely influenced by the adopted design constraints. A clear influence of the number of stories, particularly reflected in chevron-braced CBFs located in the region of high seismicity (Lagos), was observed, with taller buildings displaying generally higher collapse risk levels. Furthermore, the type of LLRS was also an impactful factor of the Pc,50 results shown in the figure, with radically different collapse performance levels associated with MRF, non-chevron CBF (X, SD), and chevron CBF (V, inverted-V, split-X) cases. Such observation, which points towards a more limited ductility of CBFs in relation to MRFs, has been an important part of the design requirements of EC8-1 since its conception. The European code follows a direct logic, determined through background research, relating the LLRS typology to favorable/unfavorable factors affecting the structure's ductility [36]: i) MRFs may develop a large number of beam plastic hinges thus are associated with comparatively higher ductility and allowable q-factors (as high as 6.5) for design purposes;

ii) tension diagonals can be very dissipative local mechanisms thus EC8-1 prescribes a lower allowable q-factor of 4 for diagonally-braced (SD, X) steel CBFs; and iii) structures which require the contribution of elements subject to buckling (i.e. chevron CBFs) are recognized as much less dissipative, with a maximum allowed q-factor of 2.5 in EC8-1. Several studies in the literature dealing with the seismic performance of EC8-compliant steel structures have reported results aligned with this EC8-1 logic. Macedo et al. [11] demonstrated, across a very comprehensive suite of MRF buildings designed to EC8-1 for different locations of low to high seismicity in Portugal, the excellent collapse performance of these structures, as seen through an evaluation of collapse risk. On the other hand, D'Aniello et al. [37] demonstrated that the seismic performance of chevron-CBFs is particularly sensitive to the flexural stiffness of braced-bay beams, highlighting the possibility of beam plastic hinging, as well as the consequent loss of gravity-carrying capacity of the floor, at relatively low lateral deformations. Although the authors propose practical strategies to mitigate this issue at the design stage, these modifications are not yet reflected in the current version of the European seismic design guideline. Finally, via the comparison of seismic performance of steel CBFs designed to American and European guidelines, Azad et al. [38] reported the susceptibility of EC8-compliant CBFs to soft-story mechanisms, in particular in the uppermost story. Moreover, the authors also report that unlike US designs, beam yielding in chevron-braced CBFs designed to EC8-1, which prevented tensile yielding of the connected braces and increased the brace ductility demands in compression, was frequently observed. The authors suggest that future research should clarify the level of conservativism of chevron beam design of US and European design requirements.

The results obtained in this research are aligned with the conclusions from the abovementioned literature, confirming that the existing design requirements of EC8-1 may lead to acceptable, yet variable, levels of collapse performance across different steel building typologies. These findings again highlight field for improvement of the European seismic design code. Research lines focusing on improved design procedures targeting enhanced collapse performance of steel CBFs, in particular for chevron-braced cases, should be a focus of development. As discussed previously, a number of research contributions have made relevant suggestions towards this overall objective. The incorporation of more stringent design requirements of brace-intercepted beams in chevron CBFs [37] could also be adopted, although the influence of this proposal on collapse risk has not been extensively investigated. Moreover, the possible emulation of the American seismic design requirements could be considered, which seemingly lead to steel CBFs with better collapse performance [38].

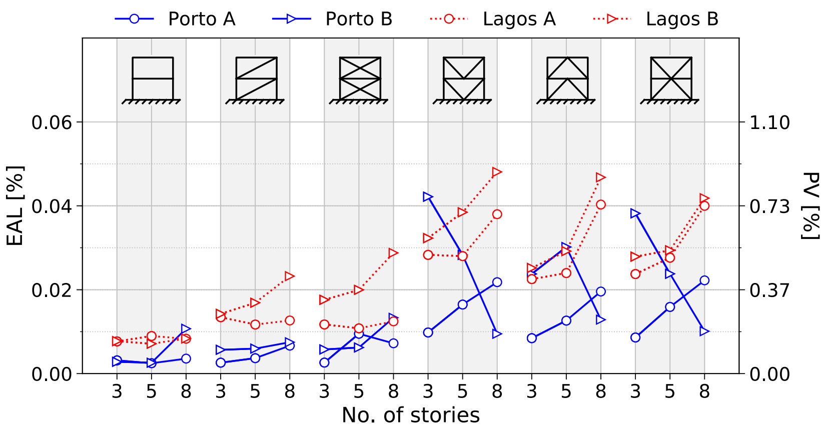
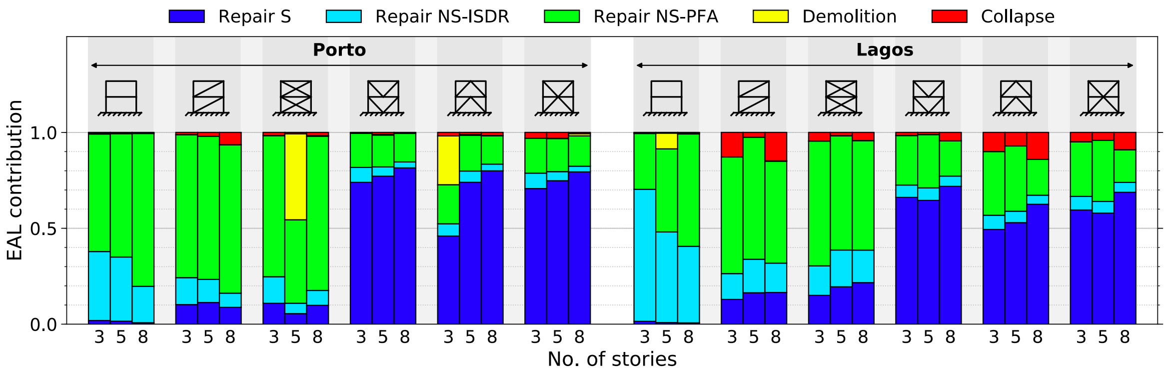


Fig. 3 – Summary of total expected annual losses and present value of lifecycle costs [31]

Based on the computation of seismic economic losses, the EALs and PVs across the entire building population considered in this study were evaluated. As summarized in Fig. 3, very low losses were observed for the suite of steel structures assessed in this study, with EALs lower than 0.05%. According to the recently proposed Italian seismic risk classification system [7], buildings with EALs lower than 0.5% correspond to the best risk class of the rating scheme. This comparison stands for the good overall seismic performance of EC8-compliant steel MRF and CBF buildings. It should also be reported that, as shown in Fig. 3, similar trends to those referring to the analysis of collapse risk were observed for EALs and PVs. Seismic losses associated with MRFs were generally lower than the comparable non-chevron CBFs, which in turn were lower than those associated with chevron CBFs. It should be noted, however, that the hierarchy of collapse performance reported in the previous section plays a limited role on the hierarchy of seismic losses. This may be observed from the disaggregated EALs related to the various loss components (i.e. repair, demolition, collapse), represented in Fig. 4.



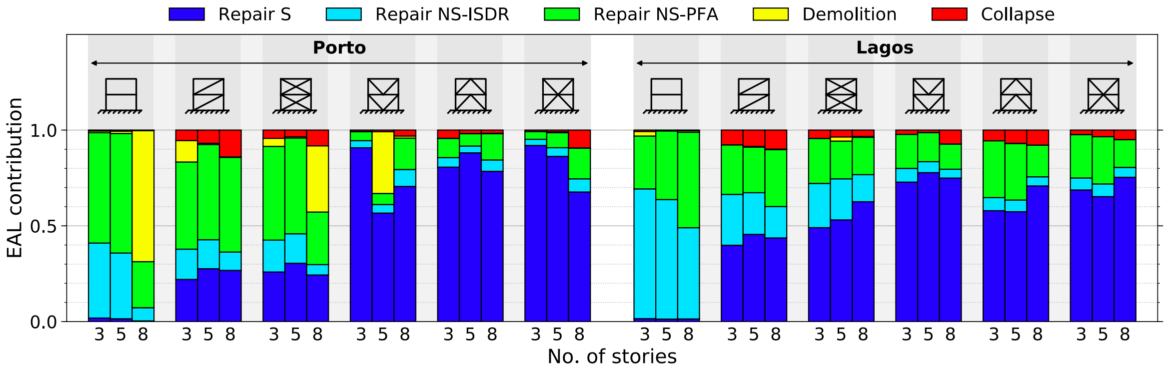


Fig. 4 – Summary of EAL contributions (plan configuration A and B on top and bottom, respectively) [31]

From the analysis of the relative contributions to the EALs shown in Fig. 4, one may infer that collapse-related EALs generally accounted for a limited proportion (lower than 10 to 15%) of the total expected loss. On the other hand, a dominant contribution of repair-related losses was observed across the majority of the considered archetype buildings. This important observation confirms, specifically for steel frame buildings, that future revisions of the European seismic design framework should favor the resilience of buildings and infrastructures, namely through a more stringent control of structural and non-structural damage. However, it is also important to note that the observed relative contribution of the repair loss components differed substantially between LLRS typologies. In line with the results reported previously in the paper, repair EALs of MRF archetypes located in the high-seismicity region (Lagos) generally experienced significant losses stemming from damage to drift-sensitive non-structural components (NS-ISDR), with the damage to acceleration-sensitive contents (NS-PFA) playing a secondary role. The opposite trend was observed for the low-seismicity (Porto) archetypes, which exhibited governing NS-PFA losses. This is due to the fact that the EC8-compliant MRF buildings located in the high-seismicity region experienced much higher lateral deformations than the comparable low-seismicity-region-located MRFs for the same return period. Although peak floor accelerations for the same return period were also lower for the latter archetypes, this reduction was not as pronounced, which subsequently entails that the proportion between NS-ISDR and NS-PFA losses in high- and low-seismicity was not comparable. To what regards the non-chevron CBF buildings (i.e. SD, X), a prevailing contribution of NS-PFA repair losses was observed. This observation highlights the importance of controlling floor acceleration demands in future versions of EC8-1, thus imposing some sort of limitation of non-structural damage during the design process. It is important to note that for the case of buildings with plan configuration B, located in the high-seismicity region, the contribution of NS-PFA losses was lower than those associated to structural damage. This is related to the fact that these archetypes were associated with higher lateral deformations demands, as denoted in a previous section of the paper. Finally, with respect to chevron-braced CBFs, the observed EALs were largely controlled by repair losses related to damage of structural components. As previously discussed in this paper, the braces of these types of CBFs tend to buckle at low ISDRs, which entails that repair operations must be conducted to either correct the out-of-straightness of the member or replace it. This effect is also associated with the non-chevron typology, albeit for more pronounced, yet still low, lateral deformation levels. On the contrary, MRFs are expected to observe minimal structural losses up until much more significant lateral deformations [16]. Noting that: i) by accounting for the probability of occurrence of all seismic intensities (i.e. seismic hazard) within the computation of EALs, the contribution of low intensities (high probability, low return period) may be predominant, and ii) ISDR levels conducting to brace-buckling were generally observed at lower return periods for chevron CBFs than for non-chevron CBFs; and iii) for those same return periods, MRFs typically exhibited larger deformations, yet low enough as not to produce significant structural damage; one may therefore justify the higher levels of losses associated with chevron-braced CBFs and the fact that the contribution of structural repair losses was generally more significant in these systems. Future revisions of the European seismic design framework should consider these findings, perhaps enforcing more stringent brace slenderness limits to avoid buckling-induced structural damage at low lateral ISDRs, which are often encountered even for frequent events. Alternatively, the adoption of buckling-restrained braces may be a more efficient solution to reduce structural economic losses in CBFs.

6. Towards direct consideration of seismic losses in Eurocode 8

With the goal of providing solid evidence to facilitate a more expressive incorporation of seismic losses in the European seismic design framework in the future, several analyses were reported so far in this paper, in which various aspects related to the seismic performance of steel MRF and CBF buildings were discussed. In particular, a relevant significance of seismic losses attributable to the repair of structural and non-structural components/contents has been identified (see Fig. 4). Consequently, additional analyses were conducted to identify potentially useful data trends from the extensive building population considered in this study.

Firstly, the repair losses shown and discussed in the previous section of this paper referred to a cumulative loss observed in a given building. A more detailed assessment was therefore conducted to gauge the variation of losses across the height of the buildings. In this context, the repair-related EAL of each building was disaggregated into the contribution of every story of the structure (e.g. uppermost story contributes by 35% to the repair EAL of the building). The results stemming from this analysis were found to be relatively insensitive to the LLRS typology and the seismic location, yet very dependent on the number of stories of the building. Based on this observation, this repair-related EAL story contribution was evaluated for the whole population of 72 EC8-compliant steel MRF and CBF buildings, as summarized in Fig. *5*. In the figure, the percentage of repair EAL attributable to each story is shown as a function of the position of the story along the building's height. The original results are shown via scatter symbols separated in plots relating to archetypes with 3, 5 and 8 stories, and the associated 16th to 84th percentiles are also included in the plots as shaded red regions.

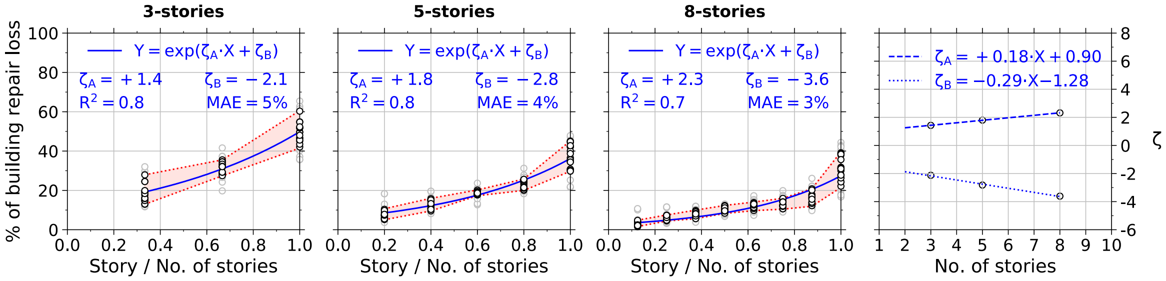


Fig. 5 – Summary of repair-related EAL contributions of each story [31]

A general overview of the results shown in Fig. 5 indicates that the contribution of each story to the EAL of the building tended to increase along the height of the building. It was also observed that such relationship follows a somewhat linear profile, with bottom stories having a lower relevance to the building's repair EAL when compared to the uppermost stories. This important observation highlights the fact that potential loss control criteria in Eurocode 8 should perhaps reflect this non-uniform distribution of repair losses across the height of the building. Based on the building population considered in this study, data trends that facilitate the depiction of the story repair-EAL contributions shown in Fig. 5 were constructed. This assessment was attained by developing simplified relationships that reflect the trends discussed in the previous paragraph, relating the story loss contribution to the position of story along the building's height, as summarized by the blue curves shown in the plots of the figure. These regressions highlight the fact that story loss contributions become more diluted as the building becomes taller, as denoted by the decreasing ζ values. Whereas the uppermost story of the 3-story buildings accounts for 50% of the repair EAL, a lower contribution of 20% of the last story was observed for the 8-story archetypes. By assembling the regressed slopes (ζ) associated with each scenario of number of stories (i.e. 3, 5 and 8), a linear regression was also established. The proposed equation for the estimation a story's repair loss contribution, %E[LR|S], is expressed in Equation (2), where S stands for the story under evaluation and NS for the total number of stories of the building.

$$\%E[L_R|S] = e^{\zeta_A \cdot (S/NS) + \zeta_B}, \text{ where } \zeta_A = 0.18 \cdot NS + 0.90 \text{ and } \zeta_B = -0.29 \cdot NS - 1.28 \quad (2)$$

7. Conclusions

In the research study detailed in this paper, the seismic performance of EC8-compliant steel MRF and CBF buildings was characterized. This was attained with the use of a comprehensive population of 72 buildings, whose definition criteria reflect different lateral load resisting systems (LLRSs), seismic hazards, plan configurations and building heights. Several metrics, from measures of engineering significance (e.g. lateral deformations, peak floor accelerations) to aspects related to a broader risk-related standpoint (e.g. collapse risk, direct seismic economic losses), were used to convey the observed performances. From the results obtained, a number of conclusions were drawn:

- The analysis of collapse risk underlined a clear hierarchy as a function of the LLRS adopted. Although all buildings showed good collapse performance when compared to the limit specified in ASCE 7-16, clear indications on the poorer collapse performance of chevron-braced CBF buildings designed to EC8-1 were observed. These findings are aligned with past literature, and support the need for improved procedures for the seismic design of steel CBFs in future versions of the European seismic code;
- The analysis of expected annual losses (EALs) revealed low losses across all steel buildings considered. A limited influence of collapse-related losses (lower than 10 to 15% of the total EAL) was observed, whilst the majority of the archetypes showed a predominance of EALs attributable to structural and non-structural damage. Furthermore, the proportion between the various repair loss components was found to be sensitive to the LLRS, an important observation that could play a role in future revisions of EC8-1 towards a more stringent control of structural and non-structural damage at the design stage;
- Various trends of potential significance to future research aimed at the incorporation of loss-related performance objectives in EC8-1 were also devised. It was shown that the importance of each story to the building's EAL follows a non-uniform distribution along the building height, with higher stories being the most relevant. This effect becomes diluted as the total number of story increases.

6. References

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