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Earthquake loss assessment of steel moment-resisting frames designed according to Eurocode 8

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ABSTRACT

Current design and assessment guidelines define several seismic performance levels, aiming to ensure that structures exhibit adequate behaviour at different seismic intensity levels. Typically, the acceptance criteria at different performance levels are met by ensuring that local deformation demands are lower than pre-defined capacities. Despite the proven effectiveness of such an approach, it provides an ambiguous measure of the performance of the building, which, in most cases, is neither meaningful nor appropriate for building owners, stakeholders or decision-makers. The main objective of the research presented in this paper is to evaluate the expected direct economic seismic losses of steel moment-resisting frame structures designed according to Eurocode 8 (EC8). A set of 120 archetype buildings, representative of the current building stock in Portugal, were designed according to Part 1 of EC8 using three different behaviour factors, q: a) code-prescribed upper bound limits for medium and high ductility classes; b) behaviour factor defined according to an Improved Force-Based Design (IFBD) procedure. The PEER-PBEE methodology with the improvements proposed by Ramirez and Miranda [1] was employed for the estimation of expected seismic losses evaluated for the seismic intensity levels considered in Part 3 of EC8. The results obtained indicate that the buildings designed in accordance with EC8 comply with the non-collapse criteria. However, the level of damage could imply significant repair costs. Importantly, the results also highlight that a rational selection of the behaviour factor can result in a reduction of steel weight but still ensuring acceptable levels of expected annual losses.

1. Introduction

Current seismic design guidelines allow for the inelastic behaviour of the structure to be explored during the design earthquake intensity level and, therefore, some degree of damage is therefore expected to occur. Although this is acceptable from an engineering point of view, given the ductile nature of structures designed according to modern provisions, stakeholders and building owners generally perceive that seismic design ensures both the safety and the development of minor damage levels for any seismic intensity level. It is therefore crucial to provide meaningful metrics of seismic performance (e.g. expected levels of earthquake-induced economic losses, fatalities, business interruption time, etc.) to support the decision making process of these agents in order to help stakeholders and building owners to take an informed selection of design options.

Seismic design according to current practices and standards aims, primarily, at the protection of life-safety, with a heavy focus on strength control, incorporating comparatively minor provisions for deformation and damage control [1,2]. However, even though code design procedures seek to ensure that buildings meet certain levels of seismic performance, the actual performance is not normally assessed throughout the design process [3]. The concept of performance-based design was firstly introduced in Vision 2000 [4] after the 1994 Northridge and 1995 Kobe earthquakes. In these earthquakes, even though most structures exhibited acceptable non-collapse performances, there were high financial losses due to downtime, damage on non-structural components and losses/damage in building contents. These findings triggered the need for a more effective control of building performance at different seismic intensity levels, leading to the concept of performance-based earthquake engineering. The first generation of performance-based design procedures defined a set of discrete performance levels (e.g. collapse prevention, life safety, immediate occupancy, and fully operational), associated with different seismic intensity levels, which were directly linked to deformation and damage in the structural components. This design philosophy was later implemented in the most recent existing seismic assessment standards/guidelines (e.g. ASCE 41-13 [5]; Part 3 of Eurocode 8 (EC8-3) [6]). In the case of the European code, EC8-3 defines three performance levels

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Table 1
EC8-3 building performance levels.

Hazard level	Performance level
2% in 50 years (T_R = 2475 years)	Near collapse (NC): building heavily damaged, very low residual strength and stiffness, large permanent drifts but still standing.
10% in 50 years (T_R = 475 years)	Significant damage (SD): building significantly damaged, some residual strength and stiffness, non-structural components damaged, uneconomic to repair.
20% in 50 years (T_R = 225 years)	Limited damage (LD): building only lightly damaged, damage to non-structural components economically repairable.

for which existing buildings must be assessed, as well as the associated seismic hazard levels (defined either in terms of the probability of exceedance in 50 years or in terms of return period), as shown in Table 1.

Moreover, for steel moment-resisting frames (MRFs), EC8-3 defines acceptance performance criteria at different earthquake intensity levels by specifying that local deformation demands should be lower than predefined local deformation capacities [7]. According to the performance criteria specified by the European code for seismic assessment, the damage on non-structural components is controlled in an indirect manner through the verification of local deformation demands imposed on structural components. Despite being proposed for existing buildings, these provisions are also used in performance-assessment of new buildings. Notwithstanding the significant progress associated to the first generation of performance-based design procedures, it is undeniable that they provide a relatively vague measure of building performance, which, in most cases, is neither meaningful nor useful for stakeholders and decision-makers. Consequently, several research studies [1,8–10] proposed more explicit and improved seismic-performance metrics (e.g. casualties, economic losses associated with repair/replacement, downtime) which can help stakeholders and building owners in their decision making process [11,12]. The Pacific Earthquake Engineering Research (PEER) Center proposed the so-called Performance-Based Earthquake Engineering (PBEE) that is a fully probabilistic framework that can be used to evaluate damage and economic losses resulting from an earthquake [1,13,14]. Moreover, the next generation of PBEE procedures [15,16] have been recently proposed, providing a series of guidelines and companion tools that aim to promote its use among the community [8,12].

The main objective of this research paper is to evaluate the expected direct economic losses, resulting from an earthquake, of steel moment-resisting frame structures designed according to Eurocode 8. A set of 120 buildings, that are representative of the current steel building stock in Portugal, were designed according to Part 1 of Eurocode 8 (EC8-1) adopting three different values for the behaviour factor, *q*. Code-prescribed upper bound limits of *q* for medium and high ductility classes (DCM and DCH, respectively), and the behaviour factor obtained from the Improved Force-Based Design (IFBD) [17,18,26] procedure were used. To this end, the PEER-PBEE methodology, with the improvements proposed by Ramirez and Miranda [1], was implemented and used. The expected economic losses and their disaggregation are evaluated for the seismic intensity levels specified in Part 3 of Eurocode 8 (EC8-3). Additionally, the expected annual losses (EAL) are presented and discussed.

2. Seismic loss assessment of building structures

Within the research study detailed in this paper, seismic losses were assessed on the basis of the building-specific storey-based loss estimation procedure by Ramirez and Miranda [12], considering the contribution of demolition-related losses [1]. By assuming mutually exclusive and collectively exhaustive events of building collapse and no collapse, the mean of the total seismic losses conditioned on given seismic intensity measure level, $E[L_T|IM]$, can be expressed as shown in equation (1) [1]. In the expression: $E[L_T|NC \cap R, IM]$ is the expected total loss given that collapse does not occur and the building is repaired; $E[L_T|NC \cap D]$ is the expected total loss given that collapse does not occur and the building is demolished; $E[L_T|C]$ is the expected total loss given that collapse occurs; $P(D|NC, IM)$ is the probability that the structure will be demolished given that it has not collapsed; $P(C|IM)$ is the probability that the structure will collapse.

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

In this research study, the probability that the building will collapse given that the ground motion intensity is $IM = im$, $P(C|IM)$, was determined based on Incremental Dynamical Analysis (IDA) [19]. For each IDA curve, building collapse was assumed if the slope of the IDA curve reduces to 10% of the initial value, or if the inter-storey drift ratio of any storey exceeds 20%. To what concerns the computation of demolition-related economic losses, the probability of demolition given a residual inter-storey 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 [20].

The expected value of losses given that collapse does not occur and the building is repaired for a ground motion intensity $IM = im$, ($E[L_T|NC \cap R, IM]$), was obtained from the sum of the repair costs at each storey of the building [12]. Storey components can be grouped into three categories: i) drift-sensitive structural components ($L_S|ISDR$); ii) drift-sensitive non-structural components ($L_{NS}|ISDR$); iii) acceleration-sensitive components ($L_{NS}|PFA$). At each storey, these categories, are weighted to translate the value of each component category that exists in a given storey. The weights adopted in this study for residential buildings in Portugal are shown in Table 2. These weights were based on information collected from several design offices and reflect typical cost ratios in the Portuguese construction sector.

Adopting the procedure proposed by Ramirez and Miranda [1], the storey fragility and consequence models have been derived from HAZUS [21] generic data which, for residential multi-family dwellings, corresponds to the following damage-to-loss model:

For each component category, the adopted storey fragility functions were based on the HAZUS [21] fragility functions for steel moment-resisting frame buildings (S1L, S1M and S1H, corresponding to low-, medium- and high-rise scenarios) designed to a "highcode" level. The parameters of each component fragility functions are shown in Table 3 and Table 4.

By combining the consequence models (see Fig. 1) with the corresponding fragility functions, the storey-based damage functions can be obtained. Moreover, the storey damage functions should be re-scaled with the component category weights presented in Table 2. Fig. 2 shows the storey damage functions used in this research study. In the figure, the loss ratio (i.e. repair cost of the storey-level component group, normalised to the component asset value) is represented as a function of

Table 2
Storey component weight.

Storey component category	Weight
Drift-sensitive, structural components	25%
Drift-sensitive, non-structural components	55%
Acceleration-sensitive, non-structural components	20%

Table 3
HAZUS [21] structural fragility functions adopted.

Damage state	Structural			
	S1L	S1M	S1H	β
	Median (ISDR)	Median (ISDR)	Median (ISDR)	
DS1	0.60%	0.40%	0.30%	0.5
DS2	1.20%	0.80%	0.60%	
DS3	3.00%	2.00%	1.50%	
DS4	8.00%	5.33%	4.00%	

Table 4
HAZUS [21] non-structural fragility functions adopted.

Damage state	Non-structural			
	Drift-sensitive		Acceleration-sensitive	
	Median (ISDR)	β	Median (ISDR)	β
DS1	0.40%	0.5	0.3 g	0.6
DS2	0.80%		0.6 g	
DS3	2.50%		1.2 g	
DS4	5.00%		2.4 g	

the level of the relevant engineering demand parameter (i.e. lateral deformation or floor acceleration).

The expected annual loss (EAL) can be computed as:

$$EAL = \int_0^\infty E(L_T|IM)\left|\frac{d\lambda(IM)}{dIM}\right|dIM \tag{2}$$

where $|d\lambda(IM)/dIM|$ is the absolute of derivative of the seismic hazard curve. Finally, the expected present value (PV) of life-cycle losses are given by Ref. [22]:

$$PV = EAL \times \left(\frac{1 - e^{r \cdot t}}{r}\right) \tag{3}$$

where r is the discount rate and t is the expected lifetime. In the current research study, a 5% discount rate and 50 years lifetime span were assumed.

It is important to note that the aforementioned HAZUS database of consequence and fragility models is based on data collected from the US building stock. Hence, to apply this data to other countries may not be fully realistic. However, due to the lack of specific information for Europe, the HAZUS consequence and fragility models were still adopted in the study presented herein. The authors are, however, of the opinion that future studies addressing this limitation (i.e. damage functions for that reflect the European country-specific building stock and repair costs) should be conducted.

3. Description of the steel buildings

This study aims to quantify the seismic losses in moment-resisting steel frame buildings designed according to EC8 considering different ductility classes. The buildings considered in this study have already been extensively analysed to assess the collapse performance and the performance assessment according to the EC8-1 requirements [23]. The selected steel buildings are representative of the current steel building stock in Portugal and consist of six building configurations with different number of bays and span length. Buildings with 2, 3, 4, 5 and 8 storeys, corresponding to low to medium rise buildings, were considered, making a total of 120 buildings for each location. Three different site locations in Portugal, corresponding to different seismic intensities, were considered, namely i) Porto (low seismic load level), ii) Lisbon (moderate seismic load level) and ii) Lagos (moderate seismic load level). For Lagos, two soil types were considered (Type B and Type C) according to the EC8 classification. Fig. 3 shows the elevation and plan views of one of the building configurations. The analysed frame is also identified in the figure. Seismic resistance was considered to be provided by the MRFs in the longitudinal (x) direction and by a bracing system in the transversal (y) direction. In this research study, only the internal longitudinal frames were subject of investigation. Table 5 shows a detailed description of all the building configurations considered in this study.

The steel buildings were initially designed for gravity loads in accordance with the provisions of Part 1-1 of Eurocode 3 (EC3-1-1) [24] for cross-sectional resistance, stability checks and deflection limits. Seismic design was performed in accordance with the provisions of Part 1 of Eurocode 8 (EC8-1) [25], considering a medium and high ductility class levels (DCM and DCH), corresponding to two different reference values of behaviour factor recommended by the standard (q = 4.0 for DCM and q = 6.5 for DCH). Additionally, a new scenario was considered in which the behaviour factor was selected based on the Improved Force-Based Design (IFBD) procedure [17,26]. This procedure involves a more consistent selection of the behaviour factor which considers the actual properties of the structure during the design process and the seismicity of the building location. As discussed by Peres and Castro [27] and by Elghazouli [28], in the current version of EC8, the inter-storey drift sensitivity (or stability) coefficient, θ, is directly proportional to the value of the behaviour factor, which results, in many cases, in stringent lateral stiffness requirements. Therefore, as shown by Macedo et al. [26], an accurate selection of this parameter is crucial for the achievement of efficient moment-resisting frame structural solutions. It is worth noting that the IFBD procedure fully complies with the design requirements of EC8, including those associated with the enforcement of capacity design of non-dissipative components. The serviceability inter-storey drift ratio (ISDR) was limited to 1% and the stability coefficient, θ, as defined in EC8-1, was limited to 0.2. Capacity design of the non-dissipative members was conducted according to the requirements of EC8-1, with the modifications proposed by Elghazouli [28]. The EC8 capacity design requirement at beam-column joints, $\sum M_{Rc} \geq 1.3 \sum M_{Rb}$, was also taken into account in the design of all

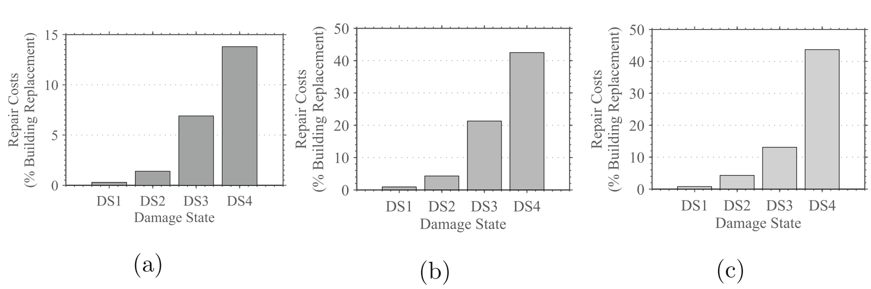
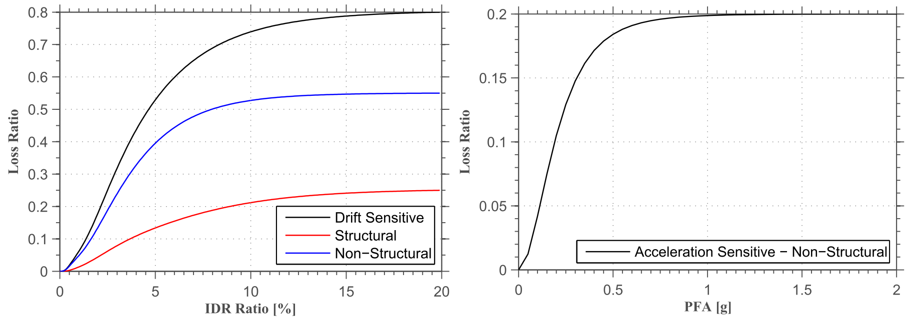
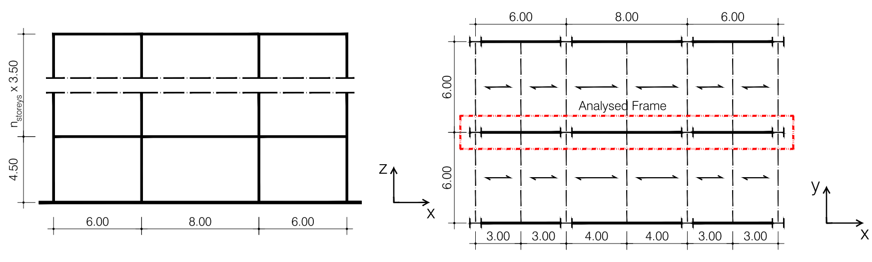


Fig. 1. HAZUS damage-to-loss model: (a) Structural components (drift-sensitive); (b) Non-structural components (drift-sensitive) and (c) Non-structural components (acceleration-sensitive).



(a) (b)

Fig. 2. Storey-based damage functions: a) Drift-sensitive and b) Acceleration-sensitive.



(a) (b)

Fig. 3. Building configuration 3: (a) Elevation view, (b) Plan view.

frames. The design process followed across the entire archetype population involved the seismic design of the critical frame (i.e. internal), and carrying this solution to all frames that are part of the lateral load resisting system. This decision reflects typical Portuguese steel design practice that generally favours building regularity in a seismic context. In total, 360 steel buildings were designed and member sizes were dictated by strength or stiffness (drift and P-Delta checks) design criteria. Steel buildings designed using EC8-1 recommended behaviour factors for medium and high ductility class were mostly controlled by stiffness requirements related to the control of P-Delta effects. On the other hand, steel buildings designed with the IFBD procedure were typically governed by strength design criteria. Moreover, for a large number of design cases, the steel buildings in which the design has been controlled by the P-Delta criterion (q = 6.5 and q = 4) resulted in the same structural solution independently of the site location (seismic intensity). In Fig. 4, the steel weight of the lateral load resisting system of each design solution is shown, for the location of Porto. In the figure, bar plots summarizing the design solutions weights are shown for the six building configurations, five frame heights (2, 3, 4, 5 and 8 storeys) and three behaviour factors considered (q = IFBD, q = 4 and q = 6.5).

Furthermore, it should be mentioned that, contrarily to what would be expected, the frames designed with higher behaviour factors are associated with higher quantities of steel due to the need to comply with stiffness requirements related to the control of P-Delta effects. Further details about the adopted design criteria and an extensive discussion on the obtained solutions can be found in Macedo [23]. The next sections of this paper will focus on a detailed characterisation of the seismic performance of the archetypes.

4. Site hazard and ground motion record selection

4.1. Site hazard characterisation

As mentioned before, three different site locations in Portugal, corresponding to different seismic hazards, were considered for the buildings, namely Porto, Lisbon and Lagos. For Lagos, two different soil conditions were considered (soil type B and C according to EC8). Probabilistic seismic hazard analysis (PSHA) was performed for the three sites under study, using the open source software OpenQuake (OQ) [29] and the seismic hazard models proposed in the recently finished SHARE project [30]. It should be noted that, for the Portuguese territory, the seismic hazard models developed in the SHARE project were implemented, but with the inclusion of additional hazard sources [31] and using the ground motion prediction equations from Atkinson and Boore [32] and Akkar and Bommer [33], with a weight of 70% and 30%, respectively [34]. The site hazard curves for all buildings were obtained from a PSHA analysis. An example for Lisbon and Lagos is shown in Fig. 5, where site-specific hazard curves for different periods

Table 5
Building configurations and geometrical properties.

Config.	x-z plane		y-z plane		h_1 [m]	$h_{typical}$ [m]
	N. of frames	Bays [m]	N. of frames	Bays [m]		
1	4	6 + 6+6	3	6 + 6	4.5	3.5
2	4	8 + 8+8	3	8 + 8		
3	4	6 + 8+6	3	6 + 6		
4	4	8 + 6+8	3	6 + 6		
5	5	8 + 6+6 + 8	4	6 + 6+6		
6	6	8 + 8+8 + 8+8	5	8 + 8+8 + 8		

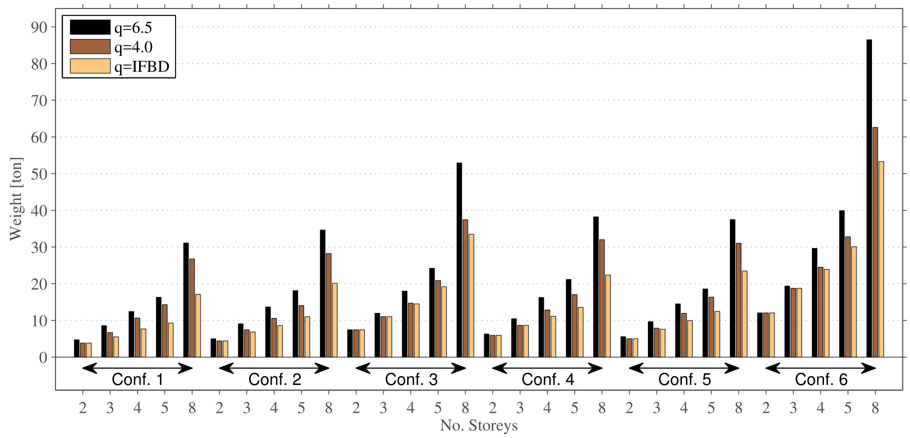


Fig. 4. Steel weight comparison of the various building configurations, for Porto.

of vibration are shown in terms of $S_a(T_1)$. Fig. 6 shows the mean uniform hazard spectra (UHS) for five different hazard levels and the corresponding EC8 Type 1 and 2 response spectra for a probability of exceedance of 10% in 50 years. Despite the differences that can be observed when the UHS (obtained with the considered hazard model) and the EC8 response spectra are compared, the considered seismic hazard model is the most recent model available for the Portuguese territory, being therefore considered the most appropriate for the hazard characterisation. It is important to note that even though the hazard model adopted herein came from the SHARE project, with the above-mentioned modifications for Portugal, different EC8-specified V_{s30} values were employed for B and C soil types sites. This results in different hazard curves from the same base hazard model. Even though the authors recognize that this methodology is simplified, since soil-type-specific GMPEs should be considered, the GMPEs currently defined in the literature for the Portuguese territory lack this important feature. Future studies should consider the application of more refined GMPEs to evaluate the effect of soil type on seismic losses in Portugal.

Additionally, disaggregation of the seismic hazard [35] on magnitude, distance and ε was performed in order to identify the hazard scenario that contributes most to the seismic hazard.

4.2. Ground motion record selection

In the current research study, the ground motion record selection was performed based on disaggregation results and average shear wave velocity for the first 30 m of soil, $v_{s,30}$, in agreement with the requirements of EC8-1. For each site location, a suite of 40 ground motion records were selected and scaled in order to obtain an appropriate matching between the median spectra of the suite and the EC8 elastic response spectra, for the range of periods of interest. A similar technique was applied in FEMA 695 project [36]. The ground motion record selection was conducted using the SelEQ framework [37], an advanced ground motion record selection and scaling framework, that allows the user to obtain not only the ground motion selection but incorporates the possibility to conduct PSHA for the European territory, making use of the open source platform OpenQuake. Fig. 7 shows the response spectra of the selected ground motion records for the site locations under study and the corresponding mean and median spectral ordinates. The EC8 elastic response spectra for a probability of exceedance of 10% in 50 years hazard level are also plotted in the figure.

As shown in Fig. 7, there is a good correlation between the median spectra of the selected ground motions and the code target spectra.

5. Numerical modelling and nonlinear structural analysis

5.1. Numerical modelling

The assessment of the structures was carried out based on response history analyses conducted with the nonlinear finite element analysis program OpenSees [38]. In order to realistically capture the development of global second-order effects in the building, the entire longitudinal lateral-load-resisting system, which is composed by several steel MRFs, is explicitly simulated by considering all moment-resisting frames modelled in series. Material non-linearity was considered through a concentrated plasticity approach considering strength, stiffness, and deterioration effects [7,39]. The effect of the axial load on the flexural capacity of the columns was taken into account in an

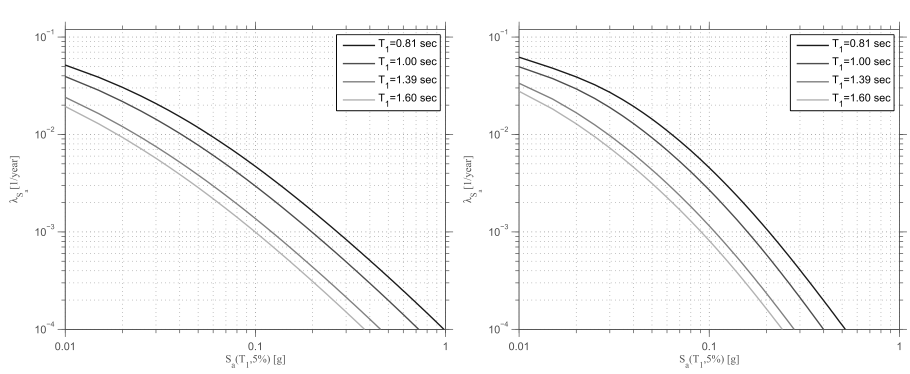
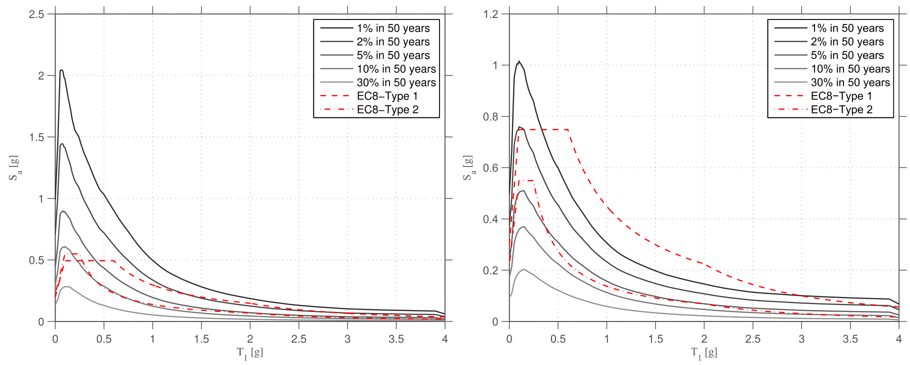


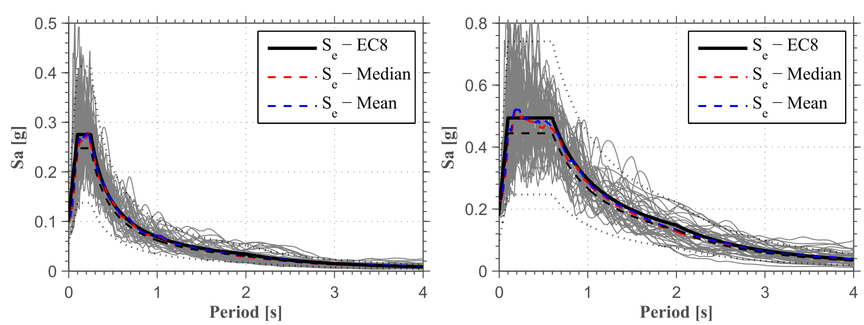
Fig. 5. Seismic hazard curves.



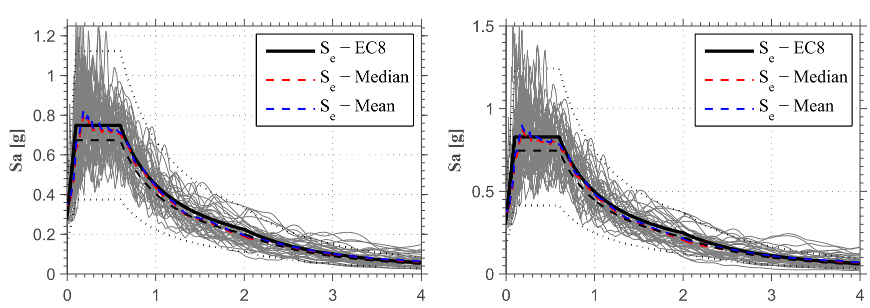
(a) Lisbon (b) Lagos

Fig. 6. Uniform hazard spectra (UHS) and EC8 spectra.

approximate manner: 1) a preliminary pushover analysis was conducted to evaluate the expected average axial force under the combined actions of gravity and lateral loading ($P_{grav} + 0.5 \times P_E^{max}$, where P_{grav} and P_E^{max} are the axial load due to gravity loads and the maximum axial force due to lateral loading, respectively) [40]; 2) the backbone curve is modified by reducing the bending moment capacity according to the interaction equations proposed in EC3-1-1, whilst no modification of the stiffness and deterioration parameters is done. The panel zones were represented with a beam-column joint element, "JOINT2D", that is available in the OpenSees framework. For the panel zone, the Krawinkler [41] tri-linear moment-distortion relation was adopted. Furthermore, the panel zones were designed with a "balanced" design methodology [2], and no strength degradation was considered. It is important to highlight that the modelling approach detailed herein neglects the effect of composite action provided by the floor slab as well as its influence on the panel zone behaviour. These effects, coupled with the explicit simulation of gravity framing (which does not apply to the archetype population assessed in this study), have been shown to have an influence on the behaviour of the joint region [42] as well as on the collapse performance of MRF buildings [8]. [43] proposed a methodology for the modification of the moment-rotation (M-θ) behaviour of steel beams to account for composite action effects. However, it is important to note that this methodology reflects the experimental findings related to American W steel profiles with reduced beam-sections (RBS). Additional research is therefore required in order to validate the application of the aforementioned methodology to the buildings considered in the present study, which reflect European design and construction practices. Inherent damping was included using the Rayleigh damping approach, considering a damping ratio of 2.0% assigned to the first two fundamental periods of vibration. Following the recommendation and coefficient modifications proposed by Zareian and Medina [44], initial stiffness proportional damping was assigned to elements that remain elastic, and mass proportional damping to nodes where the mass is lumped.



(a) Porto (b) Lisbon



(c) Lagos (d) Lagos C

Fig. 7. Response spectra of selected ground motion records and EC8 elastic response spectra.

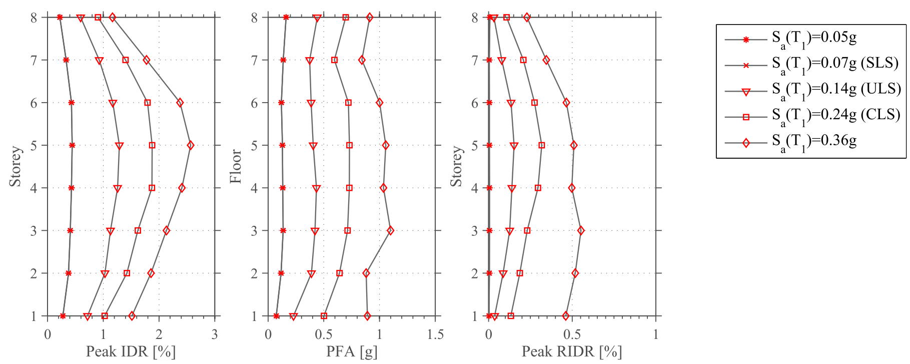


Fig. 8. Maximum ISDR, PFA and RISDR along the building height of a 8-storey building archetype example.

5.2. Nonlinear response history analysis

The seismic performance of the buildings was assessed through incremental dynamical analysis (IDA) [19]. The 5% damped first mode spectral acceleration was considered as the seismic intensity measure, *IM*, and three *EDPs* were considered, namely the maximums of inter-storey drift ratio (*ISDR*), peak floor acceleration (*PFA*) and residual inter-storey drift ratio (*RISDR*) recorded at a given storey or floor for a given response-history. Fig. 8 shows an example of the mean (across the response-history analyses associated with the relevant ground motion suite) of the maximum *EDPs* along an 8-storey building (plan configuration 1 located in Lisbon, with a fundamental period, T_1, of 2.04s and behaviour factor of 1.97) for several hazard levels.

In each analysis, the sidesway collapse was defined as the instant in which dynamic instability occurs, that is, the point where a significant increase of displacements is observed without a relevant increase of lateral inertia forces [1,8,10]. In order to accurately evaluate the maximum *RISDR*, each dynamic analysis was extended by 10s of free vibration time, and the maximum *RISDR* was evaluated for each storey by averaging the *RISDR* obtained within this free vibration time of the response history analysis.

In the evaluation of the collapse fragility curve of each building, aleatory (record-to-record) uncertainty, β_{RTR}, and epistemic (modelling) uncertainty, β_{MDL}, were taken into account. The total uncertainty, β_{TOT}, is computed with equation (4), assuming that both uncertainties are lognormally distributed and independent [9,45].

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{MDL}^2} \tag{4}$$

The value of total system uncertainty was taken as 0.53. The median collapse capacity was adjusted in order to account for the spectral shape effect, in accordance with Method 2 proposed by Haselton et al. [46]. Fig. 9 shows some of the IDA curves and the corresponding collapse fragility curves, before spectral shape adjustment, obtained in this study.

6. Economic seismic losses

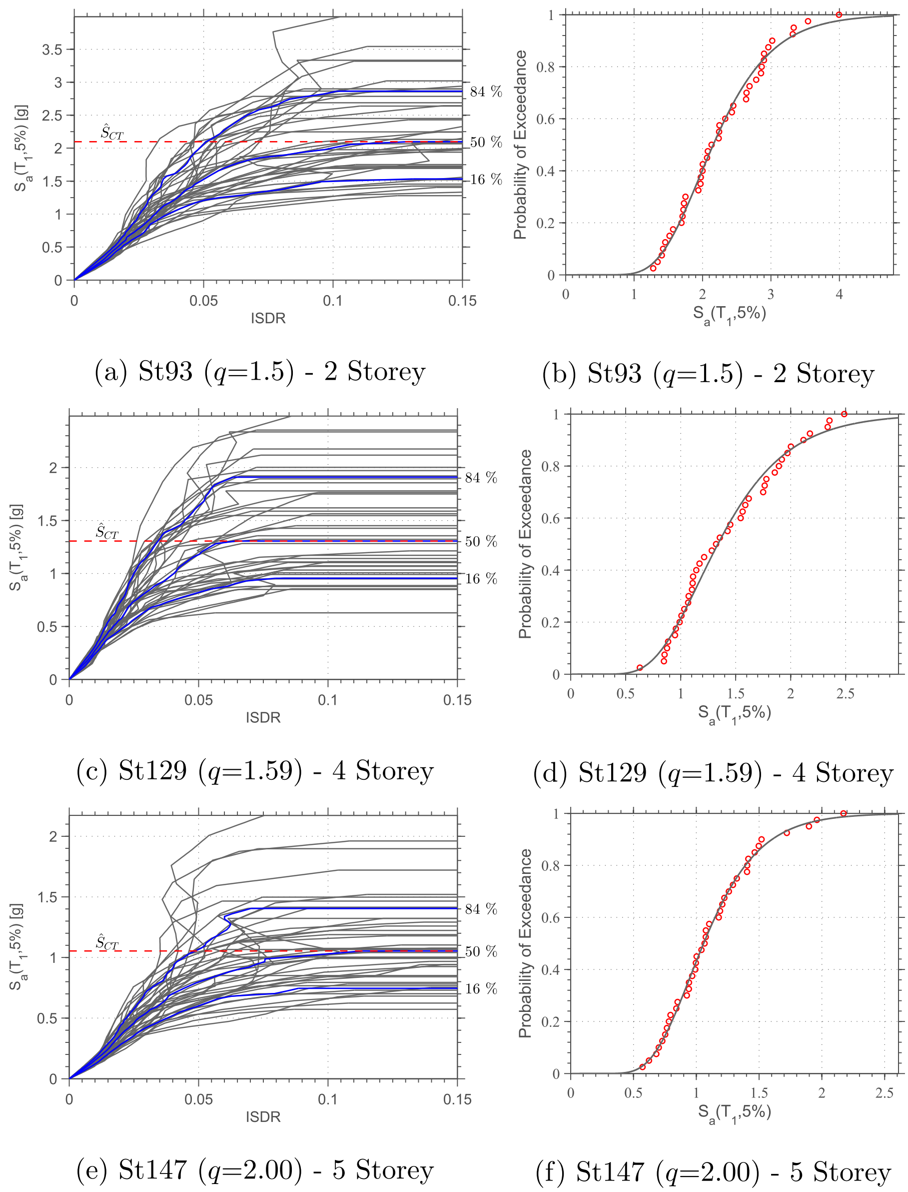
There are several useful metrics for the characterisation of economic seismic losses in buildings [1,8,9,47,48]. Among them, the most used are: i) the expected losses conditioned on the seismic intensity; ii) the expected annual losses (EAL) and iii) the expected present value (PV) of life-cycle costs. Each of these metrics can provide relevant information to stakeholders and building owners. For example, the expected annual losses can be compared to annual insurance premiums and the expected present value of life-cycle costs shows the potential financial expenses during the lifetime of the building. At the design stage, building owners and stakeholders can adopt measures to mitigate these potential expenses. In the following sections, the aforementioned metrics are quantified for all the buildings considered in this study and the results obtained are compared.

6.1. Expected losses conditioned on seismic intensity

The expected losses as a function of the seismic intensity level (i.e. loss vulnerability curves [1]) are shown in Fig. 10. In the figure, the loss vulnerability curves for two buildings with 5 and 8 storeys, as well as the corresponding losses for four different intensities are shown. Both buildings are located in the city of Lisbon. The seismic intensity levels considered are those defined in EC8-3 for checking three performance levels (as shown in Table 1: SLS-3 limited damage, ULS Significant damage or design intensity level, CLS Near collapse) and that prescribed in EC8-1 to check the damage limitation limit state (SLS-1). It should be noted that the loss values shown in the figure curves are defined within a 0 (no loss) to 1 (entire building asset value loss) range. This results from the adopted seismic loss estimation procedure, in which all loss components (e.g. repair-related via the adopted damage functions, collapse-related, demolition-related) are automatically computed as a percentage of the total value of the building.

A trend that can be identified in Fig. 10 is that, in the total loss vulnerability curves, there is a linear increase for lower intensities where most of the damage occurs in non-structural elements. It is also interesting to note the important contribution of demolition losses, which occur, in several cases, for much lower intensities than the collapse intensity. Analysing the losses at the above-mentioned intensities, it is possible to conclude that, for the damage limit states SLS-1 and SLS-3, most of the losses are associated to the repair of non-structural components, with a slight contribution from structural repair losses. It is worth mentioning that a similar behaviour is found for the design intensity level with losses controlled by damage developing on non-structural elements. However, for this intensity level, demolition losses are already present. Finally, for the near collapse seismic intensity level (CLS), there is an increase in the demolition losses, but the most interesting aspect is the absence of losses due to collapse of the buildings.

It is also worth analysing the disaggregated expected losses at a given seismic intensity level. Tables 6–8 and, 9 summarise the average of the disaggregated normalised expected losses at the design intensity level for each behaviour factor and number of storeys. In the tables, parameters *EAL* and *PV* will be the focus of discussion later in this article. From the inspection results shown in the tables, it can be concluded that for Porto (low seismicity), regardless of the adopted behaviour factor, more than 60% of the repair costs are due to damage in non-structural elements, the remainder being from structural damage. For the other locations (Lisbon and Lagos), the contribution of non-structural repair costs decreases with the increase of the seismicity of the site, with significant contributions emerging from demolition losses. Moreover, the losses due to demolition were more noticeable when the behaviour factor adopted in the design was defined according to the IFBD procedure. For example, for three storey MRFs located in Lagos and designed using the IFBD, the contribution for the total expected repair losses at the design intensity level are: 14% from structural components, 74% from non-structural and 12% from demolition losses.



(a) St93 (q=1.5) - 2 Storey
(b) St93 (q=1.5) - 2 Storey
(c) St129 (q=1.59) - 4 Storey
(d) St129 (q=1.59) - 4 Storey
(e) St147 (q=2.00) - 5 Storey
(f) St147 (q=2.00) - 5 Storey

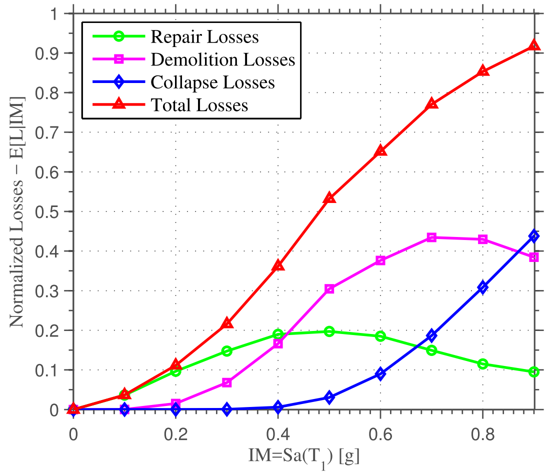
Fig. 9. IDA curves and collapse fragility curves for buildings located in Lisbon.

Buildings designed using the behaviour factors recommended in EC8-1 for medium and high ductility class were mostly controlled by P-Delta stiffness requirements, resulting in much more robust/stiff structural solutions then the buildings designed using IFBD procedure. This is the underlining justification for the higher demolition losses observed. These results indicate that, even though steel MRFs designed in accordance with the requirements of EC8 are expected to have low probabilities of collapse for the design intensity level, the level of induced damage could imply repair costs of 33% of the building replacement value. Even for serviceability limit state (SLS) intensity level, which the code specified for controlling the level of damage for lower seismic intensities, the extent of damage can lead to repair costs of around 12% of the building replacement value. Regarding the effect of considering different soil conditions, the results show an increase of up to 6% in the expected losses when soil type C is considered in comparison to soil type B. However, the results reported herein may not be statistically significant and hence further research should be conducted regarding the soil conditions effect.

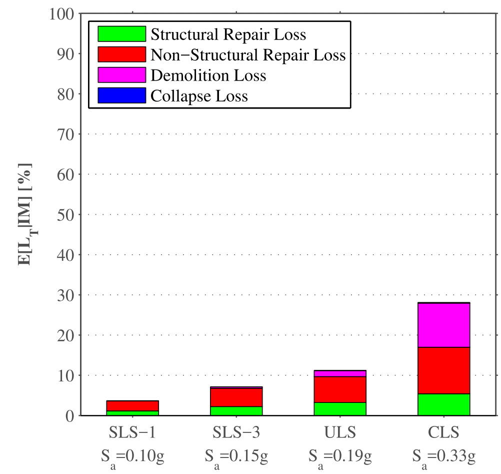
Fig. 11 shows the expected non-collapse losses due to repair disaggregated by storey at the design intensity level for buildings located in Lisbon. The results shown in the figure imply that the uniform distribution of losses over the building height. Since it was assumed that the total building cost was uniformly distributed by all storeys, the results obtained point to relatively uniform lateral deformation patterns along the height. This, in turn, reveals that there is no concentration of deformation in a single or group of storeys, which often leads to the development of unstable collapse mechanisms (e.g. soft-storeys). This observation, regardless of the loss level, points to the adequacy of the capacity design procedure prescribed in EC8. Although not shown here due to space limitations, the same behaviour was observed for higher seismic intensity levels, namely for the near-collapse intensity level, and for the various site locations considered. Furthermore, no relevant influence of the behaviour factor was found in this described behaviour.

6.2. Expected annual losses and present value of life-cycle costs

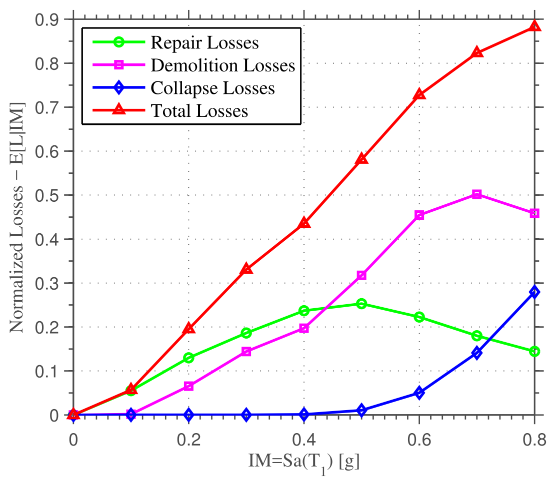
The normalised expected annual losses, EAL, and corresponding present value, PV, of life-cycle costs are shown in Figs. 12 and 13. Additionally, Tables 6–9 summarise the average EAL and PV of life-cycle costs for each behaviour factor and number of storeys.



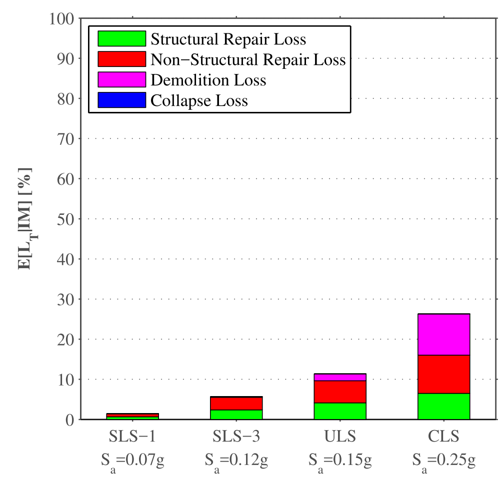
(a) St147 (q=2.00) - 5 Storey



(b) St147 (q=2.00) - 5 Storey



(c) St165 (q=2.10) - 8 Storey



(d) St165 (q=2.10) - 8 Storey

Fig. 10. Vulnerability curves and corresponding normalised expected losses at the design intensity level for buildings.

As one may infer from the results, the *EAL* values range between 0.0023% and 0.0048%, 0.016%–0.036% and 0.01%-0.0.028% for Porto, Lisbon and Lagos, respectively. The consideration of a more flexible soil typology (i.e. soil type C versus B) in Lagos significantly increased the *EAL* values, with these varying between 0.028% and 0.070%. Additionally, the *EAL* values typically decrease with the

Table 6
Mean disaggregated expected losses for the design intensity level, EAL and PV, for Porto.

Site	q	N. Stories	$E[L_S]$	$E[L_{NS}]$	$E[L_D]$	$E[L_C]$	$E[L_T]$	*EAL*	*PV*
			[%]	[%]	[%]	[%]	[%]	[%]	[%]
Porto	6.5	2	0.02	0.36	0	0	0.38	0.003	0.059
		3	0.03	0.47	0	0	0.50	0.003	0.052
		4	0.13	0.53	0	0	0.65	0.003	0.051
		5	0.14	0.54	0	0	0.68	0.003	0.049
		8	0.07	0.12	0	0	0.19	0.002	0.045
	4	2	0.05	0.53	0	0	0.58	0.004	0.066
		3	0.08	0.70	0	0	0.77	0.004	0.065
		4	0.05	0.15	0	0	0.20	0.003	0.055
		5	0.07	0.21	0	0	0.28	0.003	0.056
		8	0.24	0.35	0	0	0.59	0.003	0.055
	IFBD	2	0.05	0.53	0	0	0.58	0.004	0.066
		3	0.03	0.38	0	0	0.42	0.004	0.067
		4	0.17	0.43	0	0	0.60	0.004	0.072
		5	0.33	0.81	0	0	1.14	0.004	0.072
		8	0.42	0.64	0.02	0	1.05	0.004	0.064

Table 7
Mean disaggregated expected losses for the design intensity level, EAL and PV, for Lisbon.

Site	q	N. Stories	$E[L_S]$	$E[L_{NS}]$	$E[L_D]$	$E[L_C]$	$E[L_T]$	EAL	PV
			[%]	[%]	[%]	[%]	[%]	[%]	[%]
Lisbon	6.5	2	1.24	6.83	0.27	0	8.34	0.028	0.508
		3	1.07	5.78	0.11	0	6.96	0.023	0.415
		4	1.87	5.10	0.01	0	6.97	0.021	0.391
		5	1.66	4.44	0.06	0	6.17	0.019	0.340
		8	2.25	3.64	0	0	5.90	0.018	0.326
	4	2	1.35	6.76	0.2	0	8.32	0.029	0.536
		3	1.25	5.92	1.56	0	8.74	0.027	0.494
		4	2.5	5.78	0.68	0	8.96	0.025	0.467
		5	2.31	5.34	1.21	0	8.86	0.023	0.423
		8	3.24	4.67	0.36	0	8.27	0.022	0.403
	IFBD	2	1.35	6.77	0.2	0	8.32	0.029	0.536
		3	1.37	6.19	2.22	0	9.77	0.029	0.527
		4	2.96	6.39	1.14	0	10.48	0.030	0.549
		5	2.85	6.20	1.51	0	10.56	0.028	0.516
		8	4.14	5.65	2.77	0	12.57	0.025	0.465

increase of building height. This can be explained by the reduction of the seismic hazard with the increase of the fundamental period of vibration. Ramirez and Miranda [1] and Haselton et al. [49] have reported similar trends. The buildings designed with the IFBD procedure exhibited, in many cases, higher values of expected annual losses. However, it is worth noting that, according to Cosenza et al. [50], *EAL* values of the order of those reported in this paper would lead to a building risk class A_{EAL}^{+} ($EAL \leq 0.5\%$), which is the lowest risk class specified in the recent Italian guidelines for building risk classification. With regard to the present value of life-cycle costs of seismic damage, the average value varies between 0.042% and 0.09%, 0.30%–0.66%, 0.19%–0.52% and 0.52%–1.29% of the total replacement value for Porto, Lisbon, Lagos and Lagos soil type C, respectively. This means that stakeholders and building owners should expect to spend between 0.042% and 0.66% of the initial construction cost in repair costs due to earthquake damage occurring during the buildings service life, depending on the site location. These results are useful, at the design stage, for building owners and stakeholders to adopt measures aiming at the improvement of seismic performance.

Importantly, the results shown in Figs. 12 and 13 clearly show the advantage of adopting the *EAL* as a seismic-performance metric, in comparison with the expected loss conditioned on seismic intensity. Conversely to *EAL*, which considers all possible levels of seismic intensity weighted by the corresponding probabilities of occurrence, the expected loss conditioned on seismic intensity only considers a single intensity level [1,8]. These differences may have a considerable effect on the perception of losses in the building. For example, the buildings located in Lisbon exhibited lower values of expected losses for the design intensity level in comparison with the buildings located in Lagos. However, when the EAL of the buildings is considered, the results point to the opposite conclusion, with buildings located in Lagos exhibiting the lower values of EAL. This demonstrates the importance of taking into account the effect of the hazard at the site location for a more appropriate quantification of the expected losses.

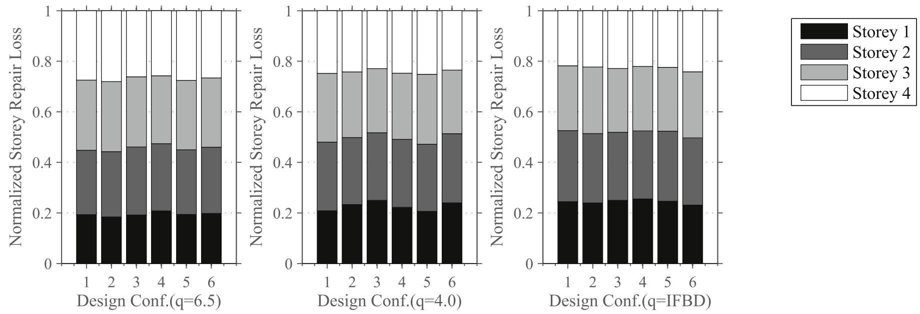
Finally, it should be emphasised that the obtained results of *EAL* are conditioned on the seismic hazard considered and, therefore, the use of a different hazard model could lead to different results. However, the hazard model used in this research study is the most recent and available hazard model for European countries, meaning that the most up-to-date information was considered.

7. Conclusions

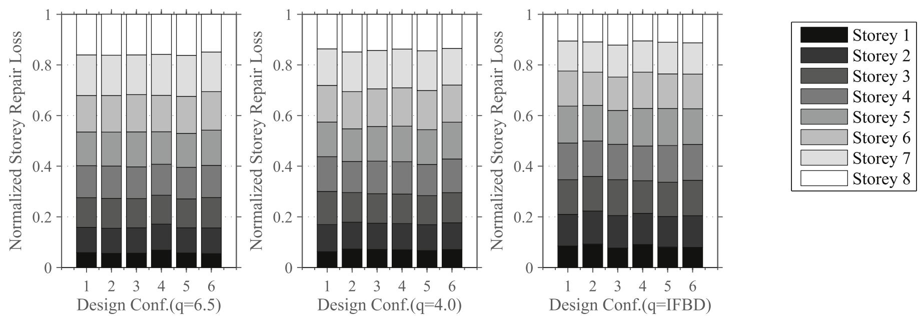
In this paper, a research study concerning the evaluation of earthquake losses of steel moment-resisting frames (MRFs) designed in accordance with the requirements of Part 1 of Eurocode 8 (EC8-1), for different ductility classes, is reported. The steel buildings were designed using the reference values of the behaviour factor proposed in the European seismic code for moderate and high ductility class structures. Additionally, the Improved Force-Based Design (IFBD) procedure for the definition of the behaviour factor was also adopted. The PEER-PBEE methodology procedure with the improvements proposed by Ramirez

Table 8
Mean disaggregated expected losses for the design intensity level, EAL and PV, for Lagos.

Site	q	N. Stories	$E[L_S]$	$E[L_{NS}]$	$E[L_D]$	$E[L_C]$	$E[L_T]$	EAL	PV
			[%]	[%]	[%]	[%]	[%]	[%]	[%]
Lagos	6.5	2	2.10	10.84	1.98	0	14.92	0.016	0.291
		3	1.84	9.68	1.59	0	13.10	0.013	0.248
		4	3.29	8.90	0.94	0	13.13	0.012	0.221
		5	3.00	7.98	1.08	0	12.06	0.012	0.216
		8	4.10	6.80	1.21	0	12.14	0.012	0.218
	4	2	2.30	11.33	2.72	0	16.34	0.018	0.332
		3	2.04	9.40	4.66	0	16.10	0.017	0.312
		4	3.75	8.91	6.13	0	18.79	0.018	0.330
		5	3.76	8.87	4.99	0	17.63	0.016	0.299
		8	5.24	8.12	3.51	0	16.87	0.020	0.376
	IFBD	2	2.30	11.33	2.72	0	16.34	0.018	0.332
		3	2.25	9.80	8.78	0	20.83	0.020	0.036
		4	4.28	9.86	7.18	0	21.34	0.023	0.431
		5	4.32	9.79	4.15	0	18.28	0.025	0.452
		8	6.19	9.15	4.96	0	20.31	0.023	0.417



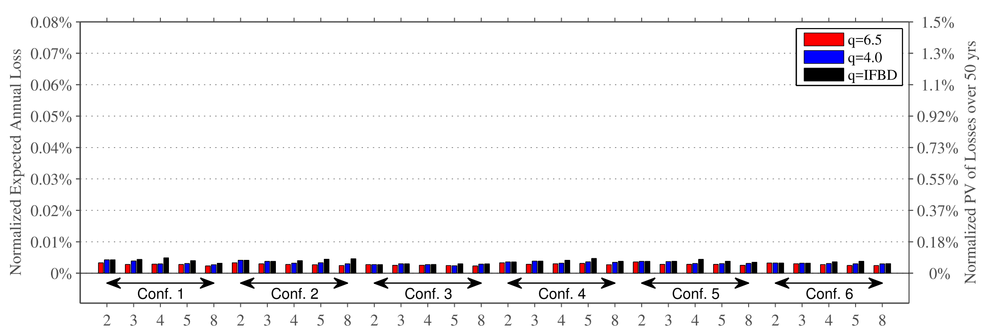
(a) 4 Storeys



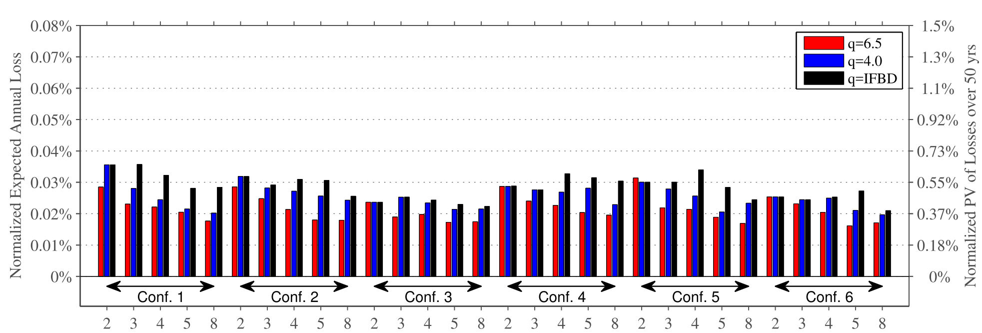
(b) 8 Storeys

Fig. 11. Normalised storey repair losses for buildings located in Lisbon.

and Miranda [1] was implemented and used to evaluate a set of 360 steel MRF archetypes, that are representative of the Portuguese building stock. Due to the large number of buildings and corresponding difficulty to generate a realistic inventory of architectural components, the seismic losses were evaluated using a storey-based building-specific loss estimation methodology, adjusting the storey fragility and consequence models from the proposed HAZUS generic data [1]. Probabilistic seismic hazard analysis (PSHA) was conducted for the three site locations considered, with the aim of computing the hazard curves and performing the collapse adjustment due to the spectral shape effect

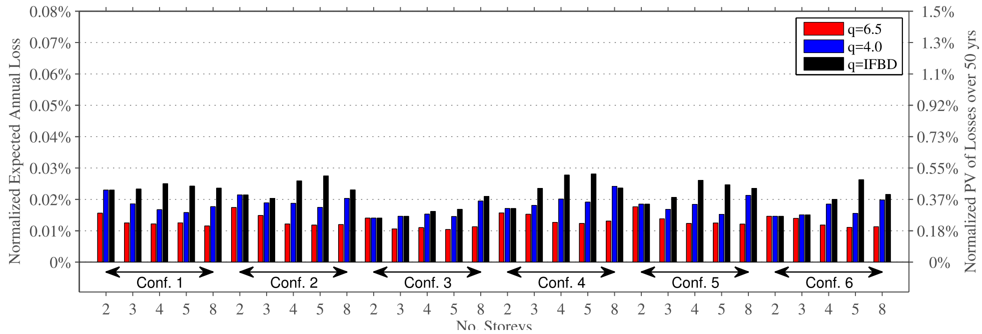


(a) Porto

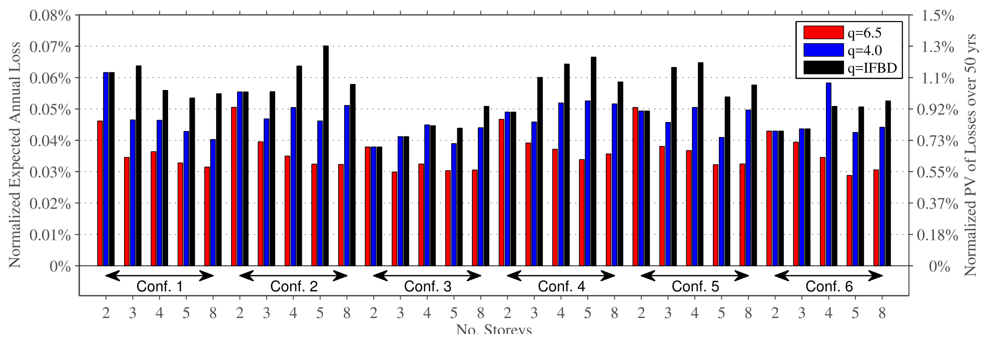


(b) Lisbon

Fig. 12. Normalised expected annual loss for buildings.



(a) Lagos (soil type B)



(b) Lagos (soil type C)

Fig. 13. Normalised expected annual loss for buildings.

[49]. The hazard model used in the PSHA was obtained from the SHARE project [30], updated with seismic sources proposed in recent studies [34]. This is the most recent and available hazard model for European countries. The expected economic losses were based on three loss metrics: the expected losses conditioned on the seismic intensity (evaluated at EC8-3 intensity levels), the expected annual losses and the expected present value of life-cycle costs. Within the scope of the results obtained in this study, the following conclusions can be withdrawn:

- The steel MRF archetypes designed in accordance with the requirements of EC8-1 comply with the non-collapse criteria defined in the code for the design intensity level. However, the level of damage could imply repair costs of 33% of the building replacement value. For the serviceability limit state (SLS) seismic intensity level, most of the repair losses are attributed to non-structural components. The values of repair costs range from 0% to 23% for the intensity level specified in EC8-3 and from 0% to 12% for the intensity level defined in EC8-1.
- The buildings designed with the IFBD procedure, which were characterized by lower values of steel weight, exhibited, in many cases, higher values of losses for all the code defined intensity levels. Moreover, an increase in the demolition losses was observed when this design procedure was adopted. Nevertheless, the values of expected annual losses obtained were well below the limits defined in the recent Italian guidelines for the lowest risk building class.
- A uniform distribution of losses over the height of the buildings was observed, revealing that there was no concentration of deformation in a single or a group of storeys, which is usually associated with the

Table 9
Mean disaggregated expected losses for the design intensity level, *EAL* and *PV*, for Lagos (soil type C).

Site	q	N. Stories	$E[L_S]$	$E[L_{NS}]$	$E[L_D]$	$E[L_C]$	$E[L_T]$	EAL	PV
			[%]	[%]	[%]	[%]	[%]	[%]	[%]
Lagos C	6.5	2	2.42	12.42	2.73	0	17.57	0.046	0.840
		3	2.00	10.56	2.65	0	15.21	0.037	0.675
		4	3.67	9.98	1.97	0	15.62	0.035	0.649
		5	3.26	8.55	2.12	0	13.94	0.032	0.583
		8	4.67	7.87	1.47	0	14.09	0.032	0.590
	4	2	2.51	12.43	3.68	0	18.63	0.049	0.906
		3	2.32	10.57	5.45	0	18.34	0.045	0.825
		4	4.15	9.93	7.83	0	21.92	0.050	0.925
		5	4.14	9.81	6.42	0	20.38	0.044	0.808
		8	5.78	9.09	3.58	0	18.46	0.047	0.859
	IFBD	2	2.51	12.43	3.68	0	18.63	0.049	0.906
		3	2.52	11.04	11.53	0	25.12	0.055	1.002
		4	4.80	11.33	8.19	0	24.32	0.057	1.053
		5	4.45	10.39	7.70	0	22.57	0.056	1.035
		8	6.39	9.82	10.57	0	26.83	0.055	1.017

development of unstable collapse mechanisms.

- The influence of different soil conditions on the design caused an increase of up to 6% in the expected losses, namely when considering soil type C in comparison to soil type B for buildings located in Lagos (Portugal). However, the results reported herein may not be statistically significant and hence further research should be conducted.
- The advantages of using the Expected Annual Loss, EAL, as a performance metric or the Present-Value of life cycle costs, PV, was demonstrated. The value of EAL ranges from 0.0023% to 0.070% of the building replacement value, depending on the building location. The corresponding present-value of annualised losses over an assumed lifespan of 50 years ranges from 0.042% to 1.29%.
- Conversely to the conclusions obtained when assessing the expected losses for the design intensity level, the buildings located in Lagos exhibited lower values of EAL and PV of life cycle costs relatively to buildings located in Lisbon.

Finally, it should be mentioned that, despite the good performance of the buildings for the design intensity level defined in EC8-1, the associated value of repair costs for that seismic intensity level is significant. Moreover, repair costs up to 22% for the SLS limit state seem fairly excessive. As previously mentioned, most of the repair costs for the lower intensities are associated to non-structural components. It is therefore critical that the next generation of seismic codes incorporates more detailed guidance that takes into consideration the relevant influence of these components on the performance of buildings.

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Appendix A. Supplementary data

Supplementary data to this article can be found online at https://doi.org/10.1016/j.soildyn.2019.05.020.

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