COMPARISON OF EARTHQUAKE-INDUCED LOSSES IN REINFORCED CONCRETE AND STEEL FRAME BUILDINGS

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Abstract

In recent years, the consideration of earthquake-induced economic losses has become a topic of great interest within the Earthquake Engineering community. The majority of past studies focused on buildings located in the US, concluding that for seismic intensities associated with service and/or design-basis earthquakes, economic losses are typically dominated by damage to non-structural elements. However, there are no similar studies considering building structures designed in accordance to the European seismic design requirements. In order to address this aspect, this study characterizes and compares the seismic performance and earthquake induced losses of a set of representative archetype reinforced concrete (RC) and steel moment-resisting frames (MRFs), designed according to Eurocode 8, for the same seismic intensity level. Nonlinear Incremental Dynamic Analysis is conducted to investigate the seismic performance of a set of 32 archetypical MRF buildings, ranging in number of stories (4 and 8), plan configuration, design ductility class, seismicity level, and material (RC and steel). Losses are quantified through a simplified approach available in the literature, using several metrics to analyze the results, namely losses conditioned on seismic intensities of interest, expected annual losses (EAL) and present value (PV) of life-cycle losses. Finally, the results obtained for the RC frames are thoroughly compared to the steel MRF counterparts to point out general trends for low and moderate intensity levels, with a view to identify future developments of the European seismic design framework that could lead to more optimized design processes.

Keywords: moment-resisting frames; reinforced concrete; steel; Eurocode 8; seismic losses

1. Introduction

For the purpose of protection of human lives and limitation of earthquake-induced damage, it is important to limit the likelihood of structural collapse at an adequately low level of seismic intensity. On the other hand, the occurrence of damage should also be controlled, which, in the European framework, is attained by considering a design intensity with a higher probability of occurrence (i.e. more frequent events) than the one associated with design against collapse. In order to properly quantify the performance of a given structural system, the Performance-Based Earthquake Engineering (PBEE) methodology proposed by the Pacific Earthquake Engineering Research (PEER) Center [1] has become the reference approach within the Earthquake Engineering community. The PEER-PBEE approach seeks to improve seismic risk decision-making, through a fully probabilistic framework, employing methodologies with a solid scientific basis and expressing the levels of performance in terms that are meaningful to stakeholders and building owners.

This research considers the assessment of the seismic performance of newly designed reinforced concrete (RC) and steel moment-resisting frame (MRF) buildings without infills, which are typically designed for gravity loads (GLD) prior to the consideration of any seismic design provisions. The assessment is carried out in terms of economic losses, quantified through a simplified approach available in the literature [2], which involves nonlinear dynamic structural response simulations up to collapse, damage analysis and loss estimation. As such, extensive numerical analyses are carried out to quantify and benchmark their performance within a PBEE framework that focuses not only on life safety but also on the quantification of direct economic losses. The ultimate purpose is therefore to understand the relative performance between RC frames and steel MRFs. The archetypes considered are located in two regions of the Portuguese territory, reflecting low and moderate seismicity levels. Within the European context, the design requirements of GLD RC and steel buildings is provided in Eurocode 2 [3] and Eurocode 3 [4], respectively, whilst the various requirements for seismic design are provided in Eurocode 8 [5]. In this study, extensive numerical analyses are carried out on a variety of case study frames to quantify the seismic performance, both in terms of collapse fragility and expected levels of losses. A total of 32 archetype buildings, varying in height (4 and 8 stories), type of plan configuration, ductility class (medium and high), seismicity level (low and moderate) and structural material (RC and steel), are analyzed.

2. Case study

To assess the performance and expected economic losses of the RC moment frames, sixteen different configurations were firstly designed according to the provisions of Eurocode 2 [3] and Eurocode 8 [5]. This section describes the archetypes and highlights their differences, which shall be further utilized in the comparisons with the steel counterpart frames designed to Eurocode 3 [4] and Eurocode 8 [5] in terms of performance and expected economic losses. A number of design scenarios, comprising different combinations of beam bay length, seismic site location (low and moderate seismicity at Porto and Lagos, respectively), number of stories and ductility class, were considered, as summarized in Table 1. The benchmark RC frame structure is illustrated in Fig. 1. The building is rectangular in plan, with its main lateral resistance in the East-West direction, provided by three moment-resisting frames (highlighted in red). These consist of four or eight story 3-bay frames, with design gravity loads of 5.75 kN/m^2 and 7.75 kN/m^2 at roof and general floors, respectively. The building is assumed to have an independent lateral load resisting system in the North-South direction, which is not addressed in the current study.

For what concerns the steel MRF counterparts, they were designed assuming the same archetype definition criteria, which include building configuration, gravity loads, seismic masses, seismic location. The set of frames with their design solutions, seismic performance and earthquake-induced losses, was obtained from the literature [6]. The first story differs in height from the upper stories (4.5m versus 3.5m, respectively).

For the seismic action, importance class II as defined in Eurocode 8, was assumed. Both spectrum types 1 and 2 were taken into consideration in the design of the frames. Under the assumption of a rigid slab diaphragm and equal frames in the E-W direction, the seismic mass is assumed to be equally distributed among the three frames in the E-W direction. Seismicity checks were carried out assuming cracked sections with reduced stiffness values. The damage limitation requirement prescribed in Eurocode 8 was verified by limiting the interstory drift to 0.75% of the story height. Second-order effects in the elastic analysis were accounted for following the recommendations of Eurocode 8. The fundamental periods of the frames were in the range of 0.6s and 1.1s. Member sizing and design was performed to meet the serviceability drift limitation of 0.75% at each story and local ductility detailing requirements were verified. Capacity design requirements, namely the enforcement of a weak beam-strong column condition, at each floor level, were adopted.

Table 1 – Archetype frame summary

Design scenario ID	**Configuration of bays [m]**	**No. of stories**	**Location**	**Ductility class**	**Behavior factor, *q***
DS1/DS2	6 x 6 x 6	4	Lagos/Porto	DCM	3.9
DS3/DS4			Lagos/Porto	DCH	5.85
DS5/DS6		8	Lagos/Porto	DCM	3.9
DS7/DS8			Lagos/Porto	DCH	5.85
DS9/DS10	6 x 8 x 6	4	Lagos/Porto	DCM	3.9
DS11/DS12			Lagos/Porto	DCH	5.85
DS13/DS14		8	Lagos/Porto	DCM	3.9
DS15/DS16			Lagos/Porto	DCH	5.85

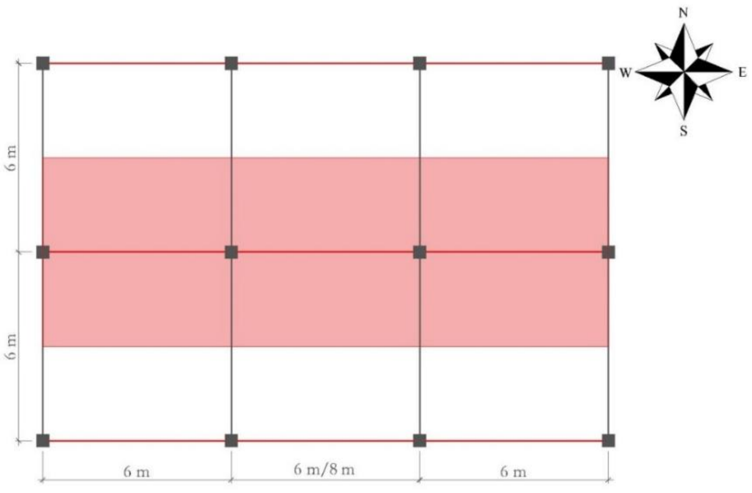

Fig. 1 – Plan view of the building structure

For all design scenarios, the spacing of stirrups at the critical regions of the RC archetypes was taken as 100mm. The spacing was reduced with respect to the elastic part of the members, regardless of the ductility class. Shear demand was negligible compared to flexural and axial demands. This is especially true for the cases of low seismicity region, as the seismic action is low, while the structures were designed considering medium and high ductility classes. Therefore, from an economic perspective, it is more efficient to consider low or medium ductility class in regions of low seismicity.

3. Numerical modelling

Nonlinear static and dynamic analyses were performed for the assessment of the seismic performance of all the frames in the East-West direction. The numerical models were developed in OpenSees [7]. The masses were assumed lumped at each floor and the nodes were constrained in the horizontal direction to mimic the rigid diaphragm behavior assumed in the design. In the models, nonlinear behavior in all structural members was considered through a concentrated plasticity approach by using nonlinear hinges that were allowed to form at the elastic member ends, where strength and stiffness deterioration effects were taken into account [8]. Haselton et al. [9] have shown that the lumped plasticity modelling approach at low levels of deformation provides reasonable results compared to fiber-element models and captures material nonlinearities as the structure collapses. Fig. 2 illustrates the numerical model of an archetypical reinforced concrete MRF.

The joint was constructed through a master node with a spring connected to the hinges through rigid links. Geometrical nonlinear effects were accounted for in the simulations whereas the effects of soil-structure interaction were not considered in the analyses as Haselton and Deierlein [8] suggest their insignificance for the rather flexible (long-period) moment frames. As shown in Fig. 3, plastic hinge models for beam-columns have a trilinear backbone curve described by five parameters (M_y, θ_y, K_s, θ_{cap} and K_c). The model was originally developed by Ibarra and Krawinkler [10,11], implemented in OpenSees by Altoontash [12] and has been adopted by PEER/ATC [13]. The plastic hinge model parameters are based on calibration of RC beam-columns elements for predicting flexural response leading to global collapse [8,14]. The model captures four modes of cyclic degradation: strength deterioration of the inelastic strain hardening branch, strength deterioration of the post-peak strain softening branch, accelerated reloading stiffness deterioration, and unloading stiffness deterioration [8]. Previously proposed empirical functions, relating the seven calibrated model parameters to the physical properties of a beam-column (i.e. axial load, concrete strength, confinement, etc.), were used to predict the mean modelling parameters, including the uncertainties. The influence of column aspect ratio is less obvious compared to the influence of the axial load ratio [14]. Hence, simplified formulations were used in this study, as they do not require the implementation of parameters such as shear span or shear demand at potential flexural yielding, which require optimization through iterative analysis. Residual strength was not incorporated in this study. As [8] suggests, some non-conforming columns tested to large deformations, showed little or no residual strength, while most conforming columns did not experience enough strength deterioration to provide a good estimate of a residual strength.

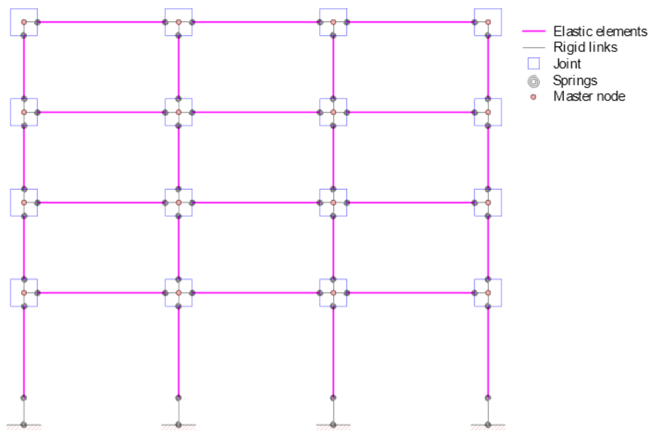

Fig. 2 – Numerical model of the frame

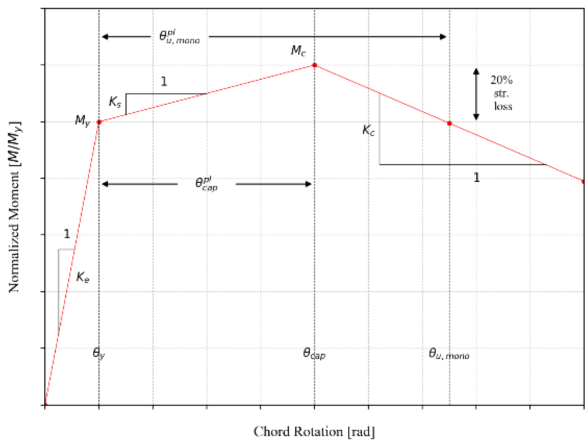

Fig. 3 – Monotonic backbone curve

4. Ground motion record selection

The nonlinear simulation models of RC frames were analyzed using incremental dynamic analysis (IDA) [15]. Two different site locations in Portugal, namely Porto and Lagos, corresponding to different seismic hazard levels, were considered. Probabilistic Seismic Hazard Analysis (PSHA) was performed for the two sites, using the open source software OpenQuake [16] and the seismic hazard model developed in SHARE [17], with the inclusion of additional hazard sources [18]. Ground motion prediction equations from [19] and [20] were used, with a weight of 70% and 30%, respectively [21]. Additionally, disaggregation of the seismic hazard [22] on magnitude, distance and ε was performed. Record selection was conducted based on the disaggregation results and an average shear wave velocity for the first 30 meters of soil, V_{s30}, was considered. For each location, a suite of 40 ground motion records was selected and scaled to match the median spectrum of the suite to the code's spectrum within a range of periods of interest, similarly to what applied in the FEMA P695 project [23]. As proposed by [9], a general ground motion record suite was selected without taking into account the ε values, with the results being post-processed to account for the expected ε at a specific site and hazard level. Records were selected using SelEQ [24], which allowed for a very good correlation between the mean/median spectrum of the selected ground motions and the code spectrum. Fig. 4 shows the response spectra (RS) of the ground motion suite for Porto and Lagos together with the corresponding mean. Furthermore, the EC8-1 response spectra for 10% in 50 years hazard level are also illustrated.

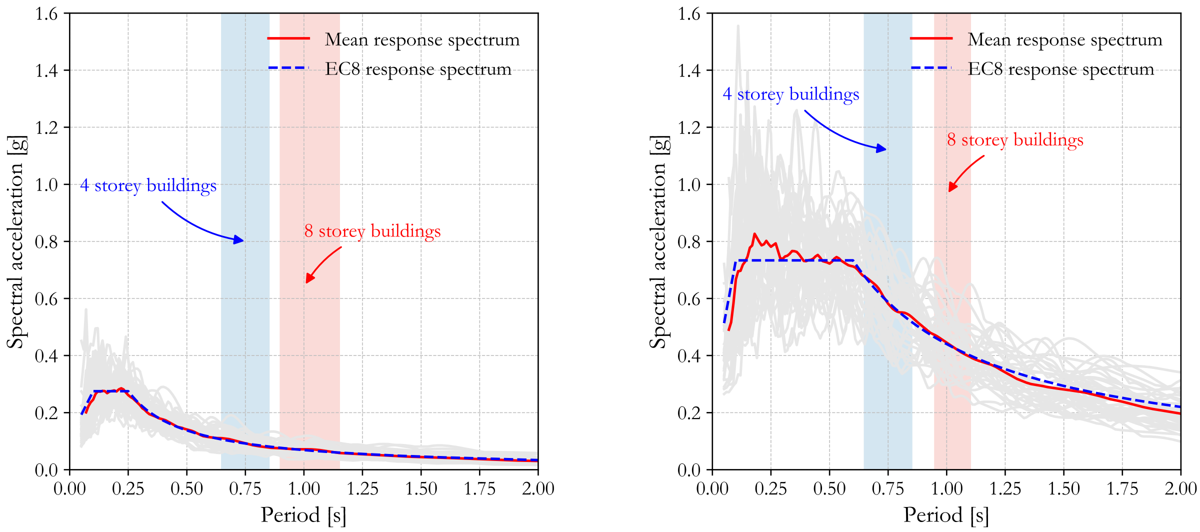

Fig. 4 – Response spectra of selected ground motion records and EC8-1 for Porto (left) and Lagos (right)

5. Nonlinear static behavior

Nonlinear static (or pushover) analyses were performed to assess the behavior of the different structural systems. A first mode proportional loading pattern was considered and the results obtained were evaluated by comparing the pushover curve of each frame normalized by the total seismic mass with the corresponding design base shear, as illustrated in Fig. 5. A number of interesting aspects can be observed in. As expected, for each design scenario, particularly the ones located in a region of low seismicity (Porto), the adoption of a high behavior factor leads to fairly high levels of overstrength (Ω), as observed by comparing the design base shear to the base shear associated with the formation of the first plastic hinge. It is also important to note that, for the 8-storey frames, the overstrength is lower, which indicates that the amount of reserve levels of strength is not as significant as for the 4-storey frames. Regarding the design scenarios for Porto, the elastic design base shear was much lower than the actual capacity of the structure. This confirms that the behavior factor prescribed by EC8 provisions [5] is quite high, which is more obvious for DCH design scenarios, as expected in high overstrength levels. As already mentioned earlier in this paper, seismic design of buildings in low seismicity regions considering ductility class medium (DCM) or high (DCH) is not recommended, as requirements concerning parameters insensitive to the seismic demand govern the member sizing and detailing of the structural system.

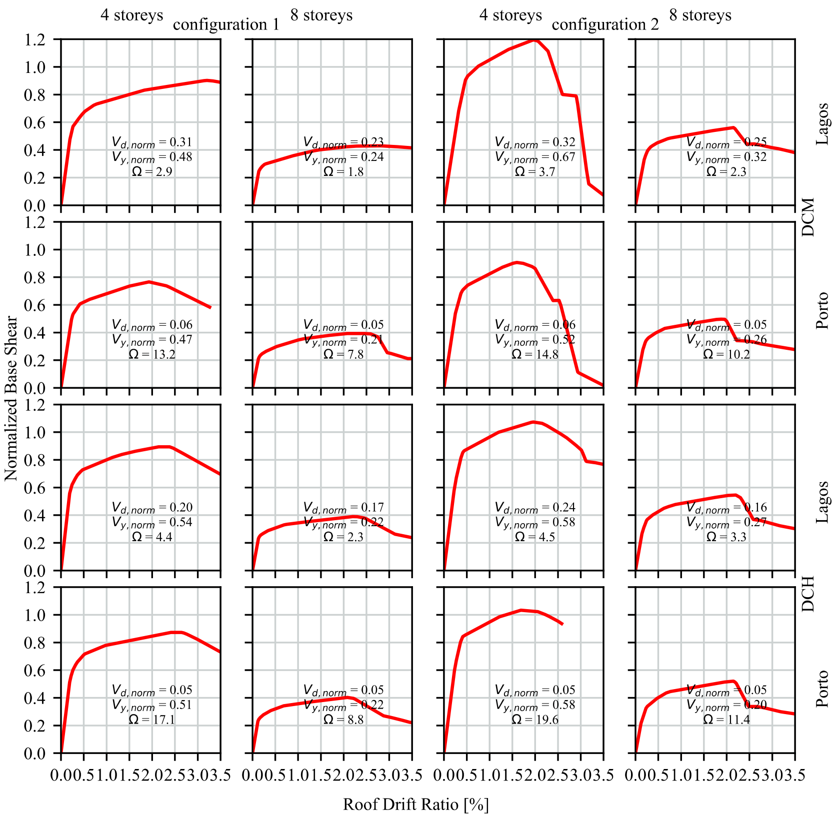

Fig. 5 – Pushover curves showing roof displacement versus base shear coefficient of the frame (normalized by the total seismic mass) for each design scenario

6. Incremental Dynamic Analysis (IDA)

Collapse probability was determined based on IDA [15], assumed to occur when the slope of the IDA curve reduces to 10% of the initial value, or when the interstory drift ratio of any story exceeds 20%. The definition of such a drift limit was implemented for a numerical analysis to stop when a structure accumulates excessive drift demand and structural instabilities occur resulting in collapse (i.e. the IDA curve begins to flatten). This was a quantitative definition of collapse at a global structural level that allowed the drift to be checked against, without the need to individually check the various structural members’ chord rotations, etc. IDA was performed using the ensemble of 40 earthquake records. It involved increased amplitude scaling of individual ground motion records to estimate the relationship between intensity measure (IM), namely the 1st mode spectral acceleration, $Sa(T_1)$, and engineering demand parameters (EDPs), namely interstorey drift ratio (IDR), floor acceleration and residual interstory drift ratio (RIDR). With the definition of collapse, the IDA returned a number of collapses, which allowed obtaining the probability of collapse for a given intensity through the construction of cumulative distribution functions (CDFs). The adopted process for fitting fragility functions using truncated data sets is described in [25]. The fragility functions were fitted assuming a lognormal distribution that was checked using the Lilliefors goodness-of-fit test at the 5% significant level. In a similar fashion, IDA was performed for the steel MRFs.

As interstory drift threshold values vary for steel and RC structures, for comparative purposes, they were based on FEMA 356 [27]. For Immediate Occupancy (IO) and Life Safety (LS) limit states, 1% and 2% interstory drift limits were defined, respectively. The resulting collapse CDFs are presented in Fig. 6 and the fragility curve parameters for each damage state used are given in Table 2. Up to 0.5g there is basically no probability of having a collapse for any of the structures. For 8-storey frames, the slope of the curve is steeper, and the probability of collapse increases more significantly in contrast to 4-storey structures. At around 2g there is practically 100% probability that nearly all RC frames located in Porto (low seismicity) and 8-storey RC frames located in Lagos (moderate seismicity) observe collapse. Some of the IDA curves show no softening, but collapse after an elastic behavior of the frames. This is possibly due to having very sensitive springs modelled for the elements. As described earlier, these have low total rotation capacities, and no residual strength was taken into account. Another reason could be that the springs were calibrated based on US data, while a better calibration and empirical expressions should be developed for modelling beam or column lumped springs designed according to European seismic code provisions.

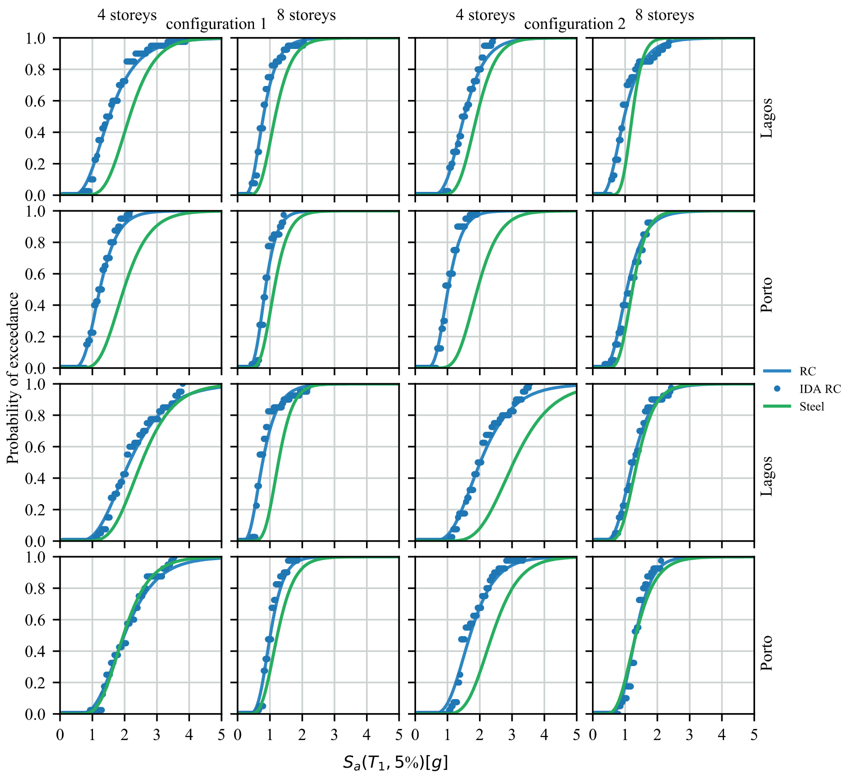

Fig. 6 – Collapse fragility curves for each of the design scenarios

Table 2 – Structural fragility curve parameters

Design scenario	Interstory drift at threshold of damage state [%]			
	Slight	Moderate	Extensive	Complete
C1M	0.33	0.67	2.00	5.33
C1H	0.25	0.50	1.50	4.00
S1M	0.40	0.80	2.00	5.33
S1H	0.30	0.60	1.50	4.00

As one can observe, at the collapse limit state, the steel MRFs show better performance. For values up to around 0.5g all buildings perform similarly, which is expected, since all buildings are designed to satisfy code requirements and they should perform well at these levels. For intermediate intensity levels, steel MRFs are less susceptible to collapse, when compared to RC MRFs. For high intensity levels, all buildings exhibit similar performance; however, these levels of intensity are very rare and have a very low probability of occurrence. It is important to note that, especially for 8-storey structures, RC MRFs start showing almost 100% collapse probability at much lower intensity levels when compared to steel MRFs. In general, at collapse limit state, since there is no pre-defined value of lateral deformation, the values of spectral acceleration leading to collapse are purely based on the formation of a plastic mechanism. As expected, steel buildings require a higher intensity level to develop a mechanism and that is reflected in a fragility curve shifted to the right in comparison to that obtained for the RC analogous frames.

7. Seismic loss assessment

There is an increasing tendency within the Earthquake Engineering community to adopt analysis procedures aimed at providing stakeholders and building owners with meaningful performance objectives that can help in decision making. These performance metrics were categorized as the “3Ds” (deaths, dollars and downtime). The PEER-PBEE approach [1], among other possible methodologies, has become the reference procedure to evaluate damage and economic losses resulting from an earthquake. In order to properly evaluate the RIDRs, each dynamic analysis was extended significantly and the maximum RIDR was evaluated for each story based on averaging the RIDR obtained in the last 5 seconds of free vibration of the response history analysis. A simplified story-based building-specific loss estimation method [28] was adopted to estimate the total losses based on the sum of the repair costs at each story of the building. At each story, the elements were grouped into drift-sensitive structural elements, drift-sensitive non-structural elements and acceleration-sensitive non-structural elements. To translate the value of each component category that exists in a given story [29], assumptions regarding the weights of typical Portuguese RC frames were used. On the other hand, a more simplistic approach was employed regarding the non-structural elements. The sum of the weights of different typologies of non-structural elements was combined into a single weight of non-structural elements for our case. In this study, for translation of the values of all component categories of the frames, these categories were weighted at: 77% attributed to the IDR-sensitive element losses (L|IDR), from which 32% was assigned to the structural elements (S|IDR) and 45% to the non-structural counterpart (NS|IDR); 23% of the global story losses were attributed to the repair needs of acceleration sensitive elements (L|PFA). By adopting the procedure proposed by Ramirez and Miranda [2], the story fragility functions (Fig. 6) and consequence models have been derived from HAZUS generic data. For each component category, the adopted story fragility functions were based on the HAZUS fragility functions for reinforced concrete MRFs (C1L, C1M and C1H) designed to a “high-code” seismic design level (Table 2).

With the combination of consequence models and corresponding fragility functions, story damage functions could be obtained, which were re-scaled with the assumed component category weights. In this study, a single loss metric was considered, namely the EAL. Computing the loss vulnerability curves [2] for a building allows obtaining the expected losses at all seismic intensities. Additionally, vulnerability curves were disaggregated, with the identification of major contributors to the losses. The examined intensities match the performance levels specified in Part 3 of Eurocode 8 [30] (SLS-3 – Limited damage – return period of 225 years, ULS – Significant damage or design intensity – return period of 475 years, CLS – Near collapse – return period of 2475 years) and EC8-1 serviceability limit state (SLS-1 – return period of 31 years) [5]. The obtained results, in terms of normalized expected losses at intensities of interest are shown in Fig. 7 and Fig. 8 for RC and steel MRFs, respectively.

The Fig. 7 and Fig. 8 show a linear increase of the total expected losses at low intensities for both RC and steel cases, where most of the losses are associated to non-collapse repair of non-structural elements. Particularly, for all frames located in the low seismicity region, the expected losses at the intensities of interest are only due to non-structural damage, while a very small portion is related to structural damage. Losses rise as the earthquake intensity level increases. For the CLS intensity, the total expected losses for the structures located in Porto appear to be limited to approximately 20% of the mean replacement cost of the structure, while for structures located in Lagos, the normalized total expected losses vary in between 30 to 60%. Only for a couple of cases, at the CLS performance level, the portion of expected losses is associated to structural damage.

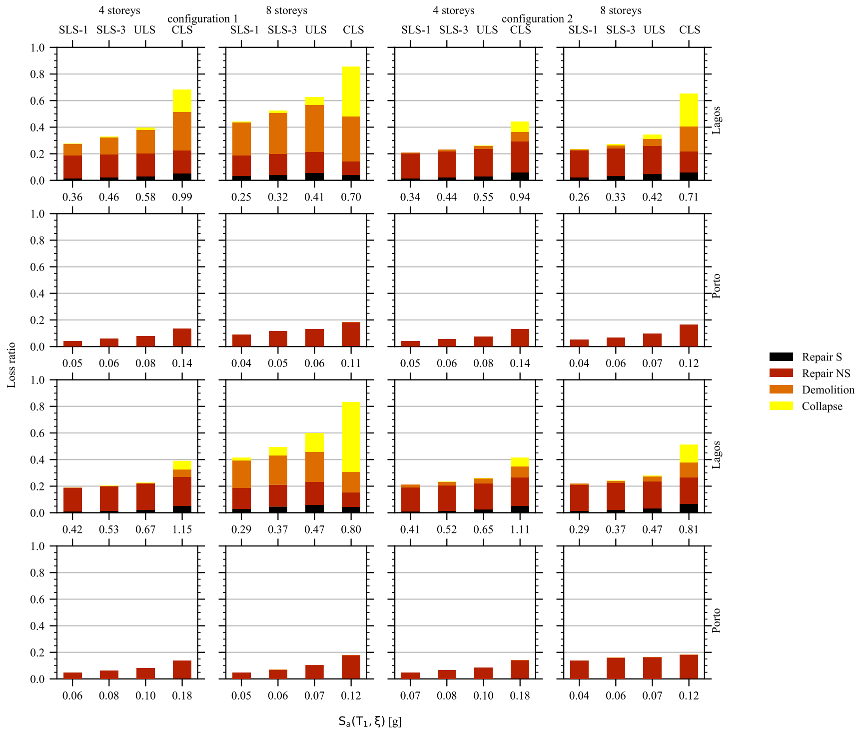

Fig. 7 – Normalized expected losses at intensities of interest for the RC MRFs

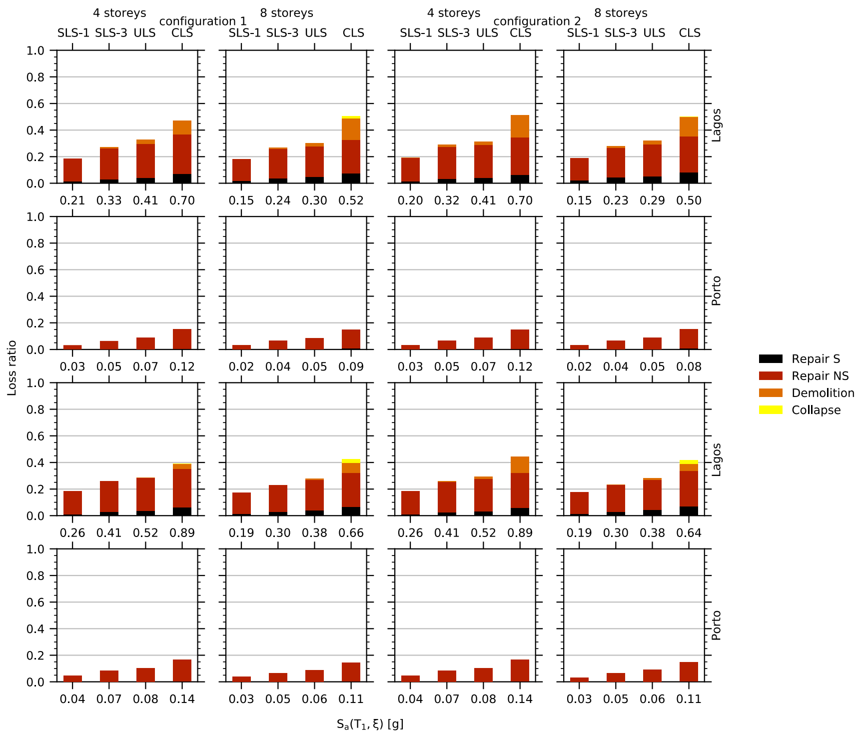

Fig. 8 – Normalized expected losses at intensities of interest for the steel MRFs

Expected demolition or collapse losses are negligibly low for the intensities considered herein. This was expected, since, as mentioned earlier, the design of the frames located in Porto was governed by requirements that ended up being independent of the design seismic intensity. Hence, the design solutions are associated with high overstrength levels and, except for a few cases, similar solutions were obtained for the structures located in regions with higher seismicity (i.e. Lagos). The sole difference came from high ductility requirements, so once again it could be confirmed that designing frames as DCM or DCH at low seismicity regions would be economically unfavorable. Seemingly, the methodologies of the code applying to low-seismicity scenarios demand some revision for RC frames. It is important to note that, for RC frames, Eurocode offers a very limited choice for the selection of the behavior factor. Hence, these observations point towards the need for future revisions of the code recommendations for addressing low-seismicity designs. With regards to the frames located in Lagos, different conclusions from those detailed for low seismicity scenarios can be inferred. At low intensities, the damage is associated to both repair and demolition losses. The substantial increase of total losses due to demolition losses could be associated to the fact that the structure is more likely to experience large residual deformations that will lead to demolition, as opposed to the structures that will collapse due to an earthquake for the given level of ground motion intensity. At low performance levels, most of the costs are associated to non-structural component repair and demolition. In comparison to the structures located in Porto, the structures located in Lagos perform similarly in terms of structural damage. Even at the CLS intensity level, losses due to structural damage are negligible, which, again, highlights the high overstrength level of the design solutions.

Another important metric for property stakeholders is the expected annual loss (EAL) (Fig. 9), which is the average economic loss due to seismic events that will be incurred each year; in other words, it considers the frequency and severity of different seismic events. The EAL is computed by integrating the expected losses as a function of ground motion intensity, using the site's seismic hazard curve. Therefore, EAL results weight all possible levels of seismic hazard by accounting for their probability of occurrence. Accordingly, seismic events with higher frequency of occurrence may have a larger effect on this loss-metric compared to seismic events of high severity but lower frequency of occurrence. Fig. 9 shows the normalized expected annual total loss results for all the archetypes under consideration. EALs range from 0.05% to 0.13% and from 0.19% to 0.91% of the replacement cost of the respective building of interest for Porto and Lagos, respectively. The high values for RC frames are mainly related to non-structural elements as described in Fig. 7.

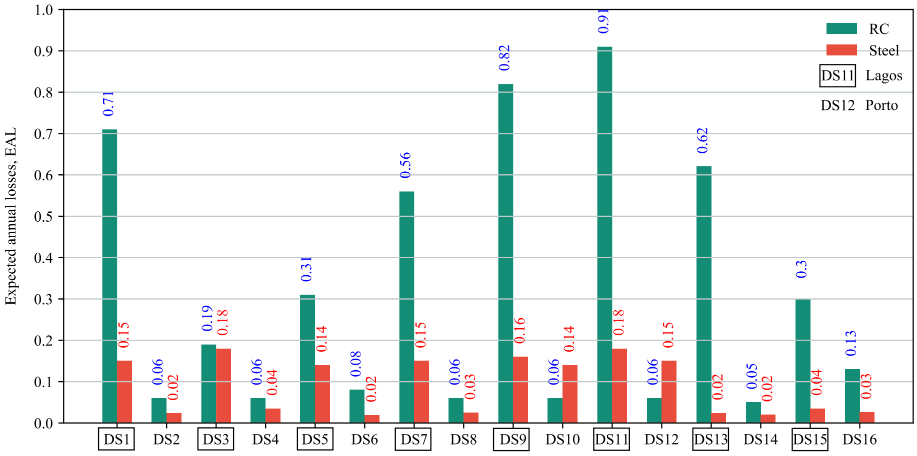

Fig. 9 – Comparison of EAL and expected life cycle cost values of RC and steel MRFs

The results indicate that expected annual losses normalized by the building replacement cost for the steel frames are more than twice lower than the corresponding losses observed for RC frames. On that account, one can say that steel MRFs show better performance during the life-span of the building compared to RC MRFs.

8. Final remarks

In this study, the earthquake-induced losses of a set of 16 RC MRF archetypes, designed according to Eurocode 8 for low and moderate seismicity locations, were assessed. This performance was also compared with equivalent steel MRFs. From the limited scope of the research conducted and described in this document, a number of preliminary conclusions may be withdrawn.

For low seismicity scenarios, the results indicated that the current design methodology prescribed in Eurocode 8 may lead to high overstrength levels. This resulted evident also during the loss estimation, where most of the losses are related to damage of non-structural elements, while structural elements remain intact, even at very high intensity levels. The design of buildings for low seismicity scenarios, by considering medium or high ductility levels, could be exaggerated. A different design approach for this type of scenarios could thus be developed in order to avoid high overstrength levels in such buildings. Furthermore, assuming high levels of behavior factor leads to fairly high overstrength levels. Overstrength factors for structures designed for low seismicity are ranging between values of 1.5 to 4.5 and 7.8 to 20 for Lagos (moderate seismicity) and Porto (low seismicity) structures, respectively. This raises questions regarding the need for new provisions concerning buildings in low seismicity regions.

In general, steel archetypes exhibited lower economic losses compared to RC analogous archetypes at almost every limit state seismic intensity. In low seismicity scenarios, economic losses are mainly comprised of non-structural repair losses. At the ULS intensity, RC frames started developing collapse losses, while the steel counterparts resulted in significant collapse losses only at the CLS intensity level and only for the 8-storey structures, indicating a better seismic performance of the steel archetypes. Even at ULS and CLS intensities, the economic losses of steel structures are dominated by losses induced in non-structural elements, while RC buildings, due to high residual drifts, result in high demolition costs. EAL values of RC frames for low seismicity scenarios were more than twice higher than the corresponding ones for steel frames, while the ones of moderate seismicity scenarios were 20% to 90% higher than steel frames.

Finally, from the structural performance point of view, there is no difference in the choice of material type in low seismicity regions. Nevertheless, given the much lower EAL of the steel archetypes, one might be inclined to choose steel over reinforced concrete. However, the decision will depend on many other variables as well, such as construction time, ease of material transportation or nominal cost of the building.

9. Copyrights

10. References

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