EARTHQUAKE-INDUCED LOSSES OF STEEL FRAMES DESIGNED TO EUROCODE 8

António SILVA¹, Luís MACEDO², José Miguel CASTRO³, Ricardo MONTEIRO⁴

ABSTRACT

In recent years, a momentum within the Earthquake Engineering community addressing earthquake-induced economic losses has been established. Regarding steel moment-resisting frames (MRFs), recent research has given further highlight to the topic, with archetypes designed to US guidelines, concluding that at seismic intensities associated with service and/or design-basis earthquakes, damage to non-structural contents dominates the total losses. Equivalent studies focusing on the European codes, or related to equally common typologies (e.g. braced frames), are still lacking. Hence, this study focuses on the quantification of seismic losses of steel MRF and concentrically-braced frame (CBF) buildings designed to Eurocode 8. A set of archetypes was defined and designed according to the current code requirements, and the seismic performance of the frames was assessed using representative groups of real ground motion records to characterize the seismic fragility through Incremental Dynamic Analysis (IDA). Losses were quantified with a simplified approach available in the literature, using the expected losses conditioned on seismic intensity as the relevant metric. In general, the results indicate that despite material savings of X-CBF versus MRF archetypes, this comes at no detriment of the expected level of seismic losses. More importantly, earthquake-induced losses of newly-designed buildings may not be within acceptable levels, particularly when considering damage to drift- and acceleration-sensitive non-structural components. Future developments of the European code should address the need for further limitations on damage levels, perhaps imposing limits on floor accelerations, in line with the existing prescriptions on interstorey drift ratios.

Keywords: moment-resisting steel frames; concentrically-braced steel frames; Eurocode 8; earthquake-induced losses

1. INTRODUCTION

In recent years, there has been a clear effort within the Earthquake Engineering community to address the issue of earthquake-induced economic losses. Regarding steel moment-resisting frames (MRFs), Hwang and Lignos (2017) have given further highlight to the topic, with archetypes designed in accordance to the American seismic design framework, as per ANSI/AISC 341-16 (AISC 2016). As denoted by the authors, at seismic intensities associated with service and/or design-basis earthquakes, damage to non-structural content dominates earthquake-induced losses in steel framed buildings, regardless of the number of stories. Furthermore, the authors reported that when composite beam effects and interior gravity framing are neglected in the numerical modelling, the contributions of the non-structural component repairs to total expected losses may be underestimated due to the fact that acceleration demands are also underestimated. Although other aspects are emphasized in the research study, the underlining notion that code-designed buildings are likely to experience significant earthquake-induced losses due to non-structural components, both drift- and acceleration-sensitive, was made clear. However, equivalent studies employing the European code (i.e. EN 1998-1 (CEN 2004) or EC8-1), or even braced-frame typologies, are still lacking in the literature.

¹PhD Candidate, Scuola Universitaria Superiore IUSS Pavia, Pavia, Italy, antonio.moutinho@iusspavia.it
²PhD Candidate, Faculty of Engineering of the University of Porto, Porto, Portugal, luis.macedo@fe.up.pt
³Assistant Professor, Faculty of Engineering of the University of Porto, Porto, Portugal, miguel.castro@fe.up.pt
⁴Assistant Professor, Scuola Universitaria Superiore IUSS Pavia, Pavia, Italy, ricardo.monteiro@iusspavia.it
At the core of the European guideline for structural seismic design there is a primary goal of protection of human life through a heavy focus on strength control, with the consideration of minor provisions to control deformation-related damage. However, the latter aspect is generally unperceived by stakeholders and building owners, and, hence, several research studies propose more explicit and improved seismic-performance metrics (e.g. casualties, economic losses associated with repair/replacement, downtime) which can help these agents in their decision-making process (Ramirez and Miranda 2009). Amongst the loss frameworks available, the highlight goes to the proposal by the Pacific Earthquake Engineering Research (PEER) Center, i.e. the so-called Performance-Based Earthquake Engineering (PBEE), a fully probabilistic framework to evaluate economic losses resulting from an earthquake (Porter 2003), with the improvements proposed by Ramirez and Miranda (2009).

This research paper, making use of two archetype buildings (low- and mid-rise elevations), designed with MRF (moment-resisting frame) and CBF (concentrically-braced frame) lateral load resisting systems to EC8-1, using a single structural ductility class and two seismic intensity levels, aims to evaluate and compare the expected direct economic losses, resulting from an earthquake, of steel MRFs and CBFs designed to EC8-1.

2. DEFINITION AND DESIGN OF THE ARCHETYPE FRAMES

2.1. Archetype definition

In order to define the archetypical structures, a single building plan configuration was considered, as summarized in Figure 1. Only the critical 2D frames in the longitudinal (x) direction were subject of investigation. Since two different lateral load resisting systems (LLRS) were considered (MRFs and CBFs), for the former it was assumed that all frames in the longitudinal direction would be a part of the LLRS, whilst for the latter only the external frames would contribute to the lateral resistance (i.e. two external seismic frames and one interior gravity frame). These assumptions are aligned with typical steel construction practice in Europe. Using this building configuration, both low- and mid-rise buildings were considered, with 4 and 8 storeys, respectively. Floor masses for seismic design were computed by combining the full dead load and 30% of the live load. Slabs were considered to act as rigid diaphragms and, thus, in each floor the mass was equally distributed by the seismic frames. For the definition of the archetypes, all bay widths shown in Figure 1 were set at 6.0m, whilst in elevation, the first storey was set at 4.5m and the remaining storeys with 3.5m. The distributed permanent gravity load was assumed as 4.75kN/m² and 5.75kN/m² for the roof and other floors, respectively, whilst 1.0kN/m² and 2.0kN/m² were defined as the live gravity load at those same floor levels. The distributed gravity loads were considered as point loads at each floor level, in accordance with the positioning of the primary and secondary beams in the transversal direction (see Figure 1).

![Figure 1. Geometry of the building configuration: a) plan view; b) elevation view.](image-url)
Regarding the design process, whilst for MRFs the columns were assumed as fixed at ground level and beam-column connections as rigid, for CBFs a pinned base and simply supported beams were adopted. All frames were firstly designed to resist gravity loads, as per EN1993-1-1 (CEN 2005a) or EC3-1-1. The EC8-1 capacity design requirement was considered in the design process of MRFs, with the proposal of Elghazouli (2009) for the definition of the overstrength parameter, Ω. For both typologies, the interstorey drift sensitivity coefficient, θ, was limited to 0.2, and the serviceability interstorey drift ratios (ISDRs) were restricted to 1%. Bracing diagonals in CBF archetypes were assumed with an X configuration in the middle bays of the frame, and, as recommended by the code, the compression diagonals were ignored at the design stage. All archetypes were designed with the modal response spectrum analysis method. For MRFs, European IPE and HEB commercial sections were adopted for beams and columns, respectively, with an S275 steel grade throughout. For the braced frames, grade S275 HEA and IPE sections were adopted for the braced and unbraced bays, respectively, whilst S355 HEB profiles were used for the columns. Commercial S275 square hollow sections were adopted for the diagonals.

Two different locations in Portugal, corresponding to different seismic intensities, were considered, namely Porto (\(a_{gR,B} = 0.08g\), for a return period of 475 years) and Lagos (\(a_{gR,B} = 0.25g\), for a return period of 475 years), with low and moderate-high intensities, respectively. A single soil type, B, an importance class II and a behaviour factor of 4 was adopted across all designs. In total, 8 archetypes were considered in this study (4 MRFs and 4 CBFs).

2.2. Preliminary comparison of design solutions

For the most part, MRFs were governed by compliance with the θ parameter, related to the control of P-Δ effects, whilst the designs of the braced frames were dictated by \(\lambda\) limits and \(\Omega_{max}/\Omega_{min}\) requirements. Since in both cases the governing parameter is independent of the seismic intensity (the former is related to a minimum level of lateral storey stiffness in the structure, whilst for the latter a minimum slenderness level is required for all diagonals), for equivalent frames between Porto and Lagos, the same design solution was obtained. Figure 2 summarizes the fundamental periods (\(T_1\)) of the design solutions against the elastic response spectrum for both Porto and Lagos. It is important to note that the fundamental period reported for the CBF archetypes concerns the “real” dynamic properties of the 2D frame, i.e. considering both the tension and compression diagonals of the bracing system. Furthermore, the figure also compares the total weight (W) of the design solution in the longitudinal direction. For MRFs, the latter equates to the sum of the steel quantities of 3 seismic frames, and for CBFs to the sum of 2 seismic frames (with two diagonals at each frame storey level) and one gravity frame (design conducted as per EC3-1-1 (CEN 2005a)). Since the design solutions obtained for Porto and Lagos were the same across all archetypes, the comparisons shown in the figure are based on unique design solutions. In the figure, the archetype labelling refers to the plan configuration (A) and the number of storeys (4 or 8).

Figure 2. Summary of dynamic properties (a) and weight of the considered archetypes (b).
As denoted by the comparison of code-compliant designs, adopting a CBF solution led to material savings between ~20% and 30% in comparison to MRFs, depending on the number of storeys. However, one should also evaluate how the MRF design solution would change if the same approach was adopted as in CBFs (i.e. external LLRS and internal gravity framing). This aspect was not addressed in this research study, since, as mentioned before, the definition of the archetypes follows the typical European construction practice.

3. NUMERICAL MODELLING

The assessment of seismic performance of the archetypes was conducted through nonlinear static and dynamic analyses using OpenSees (PEER, 2006). Different modelling approaches were adopted for MRFs and CBFs. Regarding the former, material nonlinearity was considered through a concentrated plasticity approach where strength and stiffness deterioration effects were taken into account (Lignos and Krawinkler 2011, Araújo et al. 2015). Figure 3-a illustrates the comparison between the global behaviour obtained with a detailed 3D model in ANSYS and numerical hysteresis obtained from a calibration process of the Ibarra–Krawinkler–Medina deterioration parameters of a European steel open cross-section HEB300, according to the aforementioned proposals. The effect of the axial load on the flexural capacity of the columns was taken into account in an approximate manner (Zareian et al. 2010). The panel zones of the steel frames were simulated with a beam-column joint element (JOINT2D) that is available OpenSees. For the panel zone, the Krawinkler (1978) tri-linear moment-distortion relation was adopted. Figure 3-b shows the adopted modelling strategy for the MRF archetypes.

![Figure 3. Example of the calibrated response of an HEB300 steel profile (a) and numerical modelling illustration of the structural elements and panel zones for MRFs (b).](image)

As for X-CBFs, the non-dissipative elements (beams and columns) were modelled with a distributed plasticity approach using a single inelastic force-based (FB) beam-column element per member with 10 integration points (IPs) per element. The material behaviour was defined with the bilinear Hardening material model (0.5% strain-hardening ratio). Diagonals were also simulated with a distributed plasticity approach, 10 FB elements along the length (10 IPs per element), hinges at both member ends and an initial deformed triangular shape with a mid-span amplitude of 1‰ of the brace length. The behaviour of the braces under compression was accurately replicated, particularly due to the simulation of an initial deformed shape of the member, allowing for the development of global buckling phenomena. A low-cycle fatigue (Fatigue) material model, with the parameters proposed by Hsiao et al. (2012), was adopted. Rigid floor diaphragms were modelled and additional second-order effects associated with the gravity frames were considered with a lean-column approach.
4. LOSS ASSESSMENT FRAMEWORK

As previously discussed, 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. Among the possible methodologies, the PEER-PBEE approach (Porter 2003) has become the reference procedure to evaluate damage and economic losses resulting from an earthquake. In this research study, the 1st mode spectral acceleration, $S_a(T_1)$, was used as the relevant intensity measure, IM, whilst the engineering demand parameters, EDPs, considered were the maximum and residual interstorey drifts (RISDR), as well as the peak floor accelerations. In order to accurately evaluate the maximum RISDR, each dynamic analysis was significantly extended and the maximum RISDR was evaluated for each storey by averaging the RISDR obtained in the last 5 seconds of the response history analysis. The damage functions, DM, were derived from the HAZUS (Kircher et al. 2006) consequence and fragility models. Collapse probability was determined based on IDA (Vamvatsikos and Cornell 2001), assumed to occur if the slope of the IDA curve reduces to 10% of the initial value, or if the interstorey drift ratio of any storey exceeds 20%. A simplified storey-based building-specific loss estimation method (Ramirez and Miranda 2009) was adopted to estimate the total losses based on the sum of the repair costs at each storey of the building. Moreover, at each storey the components can be grouped into drift-sensitive structural components; drift-sensitive non-structural components and acceleration-sensitive non-structural components. At each storey, these categories were weighted (25%, 55% and 20%, respectively) to translate the value of each component category that exists in a given storey. Adopting the procedure proposed by Ramirez and Miranda (2012), the storey fragility and consequence models have been derived from HAZUS generic data which, for residential multi-family dwellings, corresponds to the damage to loss model shown in Figure 4. For each component category, the adopted storey fragility functions were based on the HAZUS fragility functions for steel MRFs (S1L, S1M and S1H) designed to a “highcode” level.

![Figure 4](image_url)

Figure 4. Damage-to-loss model: drift-sensitive structural components (a); drift-sensitive non-structural components (b); acceleration-sensitive non-structural components (c).

Combining the consequence models with the corresponding fragility functions, the storey damage functions could be obtained, and storey damage functions re-scaled with the component category weights assumed. In this research study, a single loss metric was considered, namely the expected losses conditioned on the seismic intensity.

5. SITE HAZARD AND GROUND MOTION SELECTION

Two different site locations in Portugal, namely Porto and Lagos, corresponding to different seismic hazards, were considered. Probabilistic Seismic Hazard Analysis (PSHA) was performed for the two sites, using the open source software OpenQuake (Pagani et al. 2014) and the seismic hazard models developed in SHARE (Woessner et al. 2015), with the inclusion of additional hazard sources (Vilanova and Fonseca 2007) and using the ground motion prediction equations from Atkinson and Boore (2006) and Akkar and Bommer (2010), with a weight of 70% and 30%, respectively (Silva et al. 2015). Additionally, disaggregation of the seismic hazard (Bazurro and Cornell 1999) on magnitude, distance and $\varepsilon$ 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. A similar technique was applied in the FEMA P695 project (FEMA 2009). As proposed by Haselton et al. (2011), a general ground motion record suite was selected without taking into account the \( \varepsilon \) values, with the results being post-processed to account for the expected \( \varepsilon \) at a specific site and hazard level. Records were selected using SelEQ (Macedo and Castro 2017), which allowed for a very good correlation between the mean/median spectrum of the selected ground motions and the code spectrum. Figure 5 shows the response spectra of the ground motion suite for Porto and Lagos together with the corresponding mean and median. Furthermore, the EC8-1 response spectra for 10% in 50 years hazard level is also illustrated.

![Figure 5. Response spectra of selected ground motion records and EC8-1 for Porto (a) and Lagos (b).](image)

6. EARTHQUAKE-INDUCED LOSSES

There are several useful metrics for the characterization of economic seismic losses in buildings (e.g. Ramirez and Miranda 2012, Tzimas et al. 2016). Among them, one of the most used is the expected losses conditioned on the seismic intensity. Computing the loss vulnerability curves (Ramirez and Miranda, 2012) for a building allows obtaining, for each seismic intensity, the expected losses. Additionally, if the vulnerability curves are disaggregated, the major contributors to the total losses can be identified. The examined intensities match the performance levels specified in Part 3 of Eurocode 8 (CEN 2005b) (SLS-3 – limited damage, ULS – Significant damage or design intensity, CLS – Near collapse) and EC8-1 serviceability limit state (SLS-1). The results obtained, in terms of vulnerability curves and the corresponding normalized expected losses at the aforementioned intensities of interest, are shown in Figure 6 and Figure 7.
Figure 6. Loss vulnerability curves and corresponding normalized expected losses at intensities of interest for a low seismicity scenario (Porto).
Figure 7. Loss vulnerability curves and corresponding normalized expected losses at intensities of interest for a moderate seismicity scenario (Lagos).
Figures 6 and 7 show a linear increase of the total expected losses for lower intensities, where most of the damage is associated to non-structural contents. Particularly for Porto, in both LLRSs, the expected losses at the intensities of interest are only due to non-structural damage. This observation was already expected, since, as mentioned before, the designs of the archetypes located in Porto were governed by compliance with certain requirements that end up being independent of the design seismic intensity. As such, the design solutions are associated with fairly high overstrength levels, since the same solution was obtained for the structures located in regions with higher seismicity (i.e. Lagos). Another clear indicator of this aspect is that even at an intensity associated to the collapse performance level (CLS), there are no losses associated with demolition, collapse, or even repairs of structural members. Hence, the overstrength level is such that the archetype is likely to exhibit an elastic response, even at a hazard level associated to CLS (return period of 2475 years), which, one might add, is even higher than the one the structure was designed for (475 years at ULS). Evidently, the methodologies of the code pertaining low-seismicity scenarios demand some revision, at least for steel MRFs and X-CBFs. Admittedly, the scope of the results shown herein is fairly narrow. Notwithstanding, these observations point towards the need for future revisions of the guideline in order to address low-seismicity designs. Finally, one should also note that between MRF and CBF solutions, the expected losses, as well as their distribution at the intensities of interest, are fairly similar. This clearly implies that, at least for Porto, the considered bracing system not only entails lower material quantities of the main structural members (~20% less), but also carries no detrimental effect to the expected level of earthquake induced losses.

Regarding the archetypes located in Lagos, specifically those pertaining 4-storey buildings, similar conclusions to those detailed above can be inferred. Losses at the intensities of interest are only linked to non-structural damage, which, again, highlights the high overstrength level of the design solution: even for a CLS intensity level, losses due to structural damage are negligible. Furthermore, the comparison between the MRF and X-CBF archetypes yields fairly similar earthquake losses. To what concerns the 8-storey archetypes, the results clearly indicate that the braced structure is more likely to suffer higher levels of damage due to earthquake excitation. For the MRF archetype, losses due to damage to the structural elements are more noticeable than for the low-seismicity scenario, even though damage to non-structural contents still dominates the total losses. More importantly, at the CLS intensity level, losses associated with demolition costs, which relate to high levels of residual deformations of the structure, are clearly visible. Furthermore, losses due to structural collapse, which entail the development of global or local collapse mechanisms along the height of the structure (e.g. soft-storey), even though fairly low, can be seen at the CLS intensity level. Comparing to the braced LLRS scenario, the results differ substantially, particularly for higher intensity levels. Losses due to demolition and collapse “start” to appear at ULS, being mostly comparable to the losses of the MRF archetype at CLS. At the highest intensity level of interest (CLS), losses due to structural and non-structural losses drop significantly, whilst collapse and demolition losses take over. Clearly, at this intensity level, the X-CBF archetype has collapsed, and the loss can be considered total. To further validate this observation, the distribution of a number of EDPs along the structure’s height, considering the intensity levels of interest detailed before, is summarized in Figure 8 and Figure 9, for the 4-storey and 8-storey X-CBF archetype located in Lagos. It can be seen that the distribution of lateral deformations along the height of the 4-storey frame is fairly constant, indicating no concentration of plastic behaviour associated with the development of unstable collapse mechanisms. The opposite occurs for the 8-storey frame: a concentration of lateral deformations (both peak and residual) is visible in storeys 4 and 5 at the CLS. It is important to note that, perhaps, one should not expect the structure to survive such high level of seismic intensity, since Eurocode 8, in which the design of the archetype was based on, adopts a lower 475-year return period (ULS) to design against no local collapse. Nevertheless, and considering that: i) the results also show some early indicators of the same concentration of damage at ULS; ii) for a storey collapse mechanism to occur, plastic hinges have to form at the end of the columns of the storey; one could infer that the capacity design principles applied in the code seem to be ineffective in its current form. However, this can only be confirmed by analysing a more comprehensive set of archetypes.
7. CONCLUSIONS

In this research study, the earthquake-induced losses of a set of 4 steel building archetypes, designed according to Eurocode 8, considering two lateral load resisting system (LLRS) alternatives (moment resisting frames or MRFs and concentrically X-braced frames or X-CBFs) for low and moderate seismic intensity levels, were assessed. From the set of results obtained, the following conclusions can be inferred:

- Adopting X-CBF LLRS in detriment of MRF solutions leads to material savings between approximately 20% and 30%, depending on the number of storeys. These results are a function of one of the main design options, namely the consideration of different combinations of lateral load and gravity carrying frames (for MRFs, all longitudinal frames were considered part of the LLRS, while for X-CBFs only the external frames, with an internal gravity frame, were used). However, assuming the same scenario for both LLRS types would be inconsistent with the current European construction practice;

- For low seismicity scenarios, the results seem to indicate that the current design methodology of the code may lead to high overstrength levels for both LLRSs. This is evidenced by negligible levels of structural losses (associated with damage or plastic behaviour of the main structural elements), even at an intensity level (RP = 2475 years) well above the one used in EC8 for collapse control (475 years per ULS). This issue should be investigated in a more comprehensive manner in future studies.

- For archetypes designed for low seismicity, structural losses of MRF and X-CBF archetypes were very similar, for both 4 and 8 storey frames. This clearly implies that the braced system considered not only entails lower material quantities of the main structural members (~20% less), but also carries no detrimental effect to the expected level of earthquake induced losses. This conclusion also carries to the 4-storey archetypes design for a higher seismic seismicity
The MRF and X-CBF 8-storey archetypes designed for Lagos yielded some fairly different loss patterns. X-CBF losses are generally higher than those associated with the MRF LLRS, particularly at higher intensity levels (e.g. ULS and CLS). At ULS, losses due to collapse (associated with the development of local or global collapse mechanisms) and demolition (linked to high residual lateral deformation levels) are clearly visible for the X-CBF, while for the MRF they were not present. This is even clearer at the CLS intensity level, in which the losses of the former are close to the total ones, due to large contributions of the two same factors (collapse and demolition losses). This might indicate that the EC8 capacity design procedure for the design of X-braced steel frames may require some revision, and future studies addressing this issue should confirm whether or not this is accurate.

In general, earthquake-induced losses of the investigated archetypes had major contributions from damage to non-structural contents. This indicates a need for a shift in the current European seismic design approach, in which control of non-structural damage at the design stage is overlooked. Even though EC8 requires control of the levels of deformation-related damage at SLS, no limits on floor accelerations are prescribed. Knowing the importance that these two EDPs have on earthquake-induced losses, to have a better control of seismic losses at a design stage cannot be fulfilled without a better control of both lateral deformations and floor accelerations in the design process.

8. ACKNOWLEDGEMENTS

The first author gratefully acknowledges the financial support of the ReLUIS Consortium for this research via Line 7 of the ReLUIS/DPC 2014-2018.

9. REFERENCES


