ImproveD seismic design of CONCENTRICALLY   
x-braced STEEL frames to eurocode 8

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**Abstract**. Seismic design of concentrically X-braced steel frames to Eurocode 8 entails several difficulties to practitioners, often resulting in excessively heavy structural solutions. The main objective of the research presented in this paper is to propose modifications to the existing provisions of the code aiming to achieve more efficient designs. A number of possible amendments to the European seismic code are examined, concerning the non-dimensional slenderness parameter and the homogenous dissipative behaviour criterion. It is found that the relaxation of the design criteria leads to identical frame behaviour in comparison to frames fully-designed to Eurocode 8.

Keywords: concentrically-braced frames, seismic design, seismic assessment, seismic behaviour, Eurocode 8,

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# Introduction

Concentrically-braced steel frames (CBFs) are widely used for seismic resistance and are known to be less ductile than moment resisting frames (MRFs). A wide research activity developed since the 80s, in addition to observations carried out in the aftermath of important seismic events, allowed for an improvement in the understanding of the response of CBFs. These contributions played a vital role on the definition of the current European seismic design provisions (Eurocode 8 – Part 1 [CEN, 2004]).

From a European design perspective, Eurocode 3 [CEN, 2005a] provides the methods for the calculation of the capacity of steel members under different loading conditions. These procedures are complementary to the requirements of the European seismic design code (Eurocode 8 – Part 1 [CEN, 2004]), for the design of steel structures for earthquake resistance. EC8-1 targets the development of a controlled plastic mechanism under seismic loading through the application of capacity design principles, in which key dissipative zones are defined in the occurrence of a given seismic event intensity. In the case of CBFs, the code allows for the consideration of the dissipative inelastic behaviour of the diagonal members (braces). Beams and columns should be designed to remain elastic throughout the earthquake, making use of the minimum level of overstrength, , between all dissipative zones. Furthermore, the code also controls the distribution of overstrength throughout these zones, bounding the difference of minimum and maximum levels of to no more than 25%. Although the application of capacity design in EC8-1 allows for some level of control of the seismic response of the structure, in particular the development of plastic mechanisms throughout the structure, it has been shown that this leads to heavier and more expensive CBF design solutions (e.g. De Luca et al. [2006], Tremblay [2007]). Additionally, it has also been recognised that the design according to EC8-1, in particular some issues between over-restrictive or conflictive requirements, can strongly undermine, or in some cases even thwart, the application of the design procedure of the code (e.g. Elghazouli [2003], Elghazouli [2010], Malaga-Chuquitaype and Elghazouli [2011], Tremblay and Elghazouli [2011], Naqash et al. [2014]). In order to define viable alternatives to these limitations, recent research contributions have been achieved, both concerning possible modifications to the current design procedure of the European seismic code (Brandonisio et al. [2012], Bosco et al. [2017]), or even novel design methodologies based on capacity design concepts (Giugliano et al. [2011]).

Another important aspect to consider in the seismic design according to the European code is the adopted value for the behaviour factor, , which directly correlates to the energy dissipation capacity of the structure. According to EC8-1, an upper limit of is prescribed in accordance to the structural system, under different ductility classes. It is important to note that the prescribed values of are not specific to any given structure, but rather upper values to adopt at the structural design stage. Vilanni et al. [2009] thus proposed the so-called Improved Force-Based Design (IFBD) methodology for a rational determination of the adopted value of q. Also, in the case of CBFs, the code ensures that sufficient local ductility is present in the dissipative members (i.e. bracings) by limiting their maximum slenderness level.

This paper mainly focuses on the: 1) seismic design of X-CBFs following the current version of EC8-1, as well as with a number of amendments to the requirements of the code; 2) comparison of the seismic performance of case-study X-CBFs using both seismic design approaches; 3) proposal of possible modifications of existing provisions of EC8-1 for the seismic design of X-CBFs.

# Seismic design of concentrically braced frames to EC8

In this section, the design methodology of Part 1 of Eurocode 8 to the design of novel seismic-resistant concentrically braced steel frames is detailed. Additional requirements of Part 3 of EC8 [CEN, 2005b] concerning existing steel buildings are also presented. Some limitations to the design of CBFs with X bracings are thoroughly discussed.

## Seismic design to Eurocode 8

The seismic design of CBFs according to EC8-1 is aimed at obtaining ductile and dissipative ultimate behaviour, by imposing that the yielding of the diagonal members occurs before damage and premature failure of beams, columns and connections. According to the code, concentrically-braced frames shall be designed assigning the dissipative behaviour only to the braces. Whilst only the beams and columns shall be considered to provide resistance to gravity load conditions, bracing members provide the resistance to lateral seismic loads. However, whilst both tension and compression diagonals may be taken into account for steel frames with V bracings, only tension braces are considered at the design stage. Although this is related with the effect of buckling in the member under compression, which may significantly reduce the resistance of the member, the code allows for the consideration of all braces in the analysis of a concentrically-bracing system provided that: a) a non-linear static (pushover) global analysis or non-linear time-history analysis is used; b) both pre-buckling and post-buckling behaviour is taken into account in the modelling of the behaviour of the braces; c) background information justifying the model used to represent the behaviour of the braces is provided.

Regarding the extent of non-linear behaviour that is used to dissipate energy during a seismic event, Part 1 of Eurocode 8 makes use of upper limits of reference values of the behaviour factor, , for a number of structural systems and different ductility classes. To what concerns CBFs with diagonal bracings, EC8-1 prescribes the same limit to both medium (DCM) and high (DCH) ductility classes, in accordance with Table 1. Depending on the ductility class and the behaviour factor used in the structural design, the code tries to ensure that sufficient local ductility is given to the dissipative elements by requiring certain cross-sectional classes. Table 2 shows these requirements to what concerns the use of dissipative steel elements in the seismic design according to EC8-1.

Table 1 – Upper limit of reference values of for CBFs with diagonal bracings.

|  |  |  |
| --- | --- | --- |
| ***Structural type*** | ***Structural ductility class*** | |
| ***DCM***  ***(medium)*** | ***DCH***  ***(high)*** |
|  |  | |

Table 2 – Cross-sectional requirements for local ductility of steel elements

|  |  |  |
| --- | --- | --- |
| ***Structural ductility class*** | ***Range of the reference values of*** | ***Required cross-sectional class*** |
| DCM  (medium) |  | Class 1, 2 or 3 |
|  | Class 1 or 2 |
| DCH  (high) |  | Class 1 |

Concerning the requirements for X diagonal bracings, EC8-1 limits both the minimum and the maximum allowed value of the non-dimensional slenderness, , according to equation (1). For Class 1, 2 and 3 steel cross-sections, this parameter is calculated according to equation (2), in which is the buckling length in the considered plane, is the radius of gyration in the relevant plane, and is the material yield strength. Whilst the upper limit value (2.0) is also used for diagonal and V bracings, and aims to prevent elastic buckling of the braces, the lower limit (1.3) is specific to X bracings and aims to avoid overloading of the columns in the pre-buckling stage of the compressed diagonals. It is important to note that the limits need not be taken into account for structures of up to two storeys.

|  |  |
| --- | --- |
|  | (1) |
|  | (2) |

Additionally, a homogenous dissipative behaviour along the structure height should be guaranteed, in order to reduce the potential for soft-storey collapse mechanisms and damage concentration. According to EC8-1, this is attained by limiting the ratio between the maximum diagonal overstrenght, , and the minimum diagonal overstrenght, , according to equation (3). The diagonal overstrenght coefficient,, is defined as the ratio between the design axial resistance, , and the design axial demand, , of diagonal (equation (4)).

|  |  |
| --- | --- |
|  | (3) |
|  | (4) |

Concerning the design of non-dissipative elements (beams and columns), EC8-1 prescribes the application of ensures that the capacity design by imposing the minimum resistance requirement of equation (5). In the expression, is the design buckling resistance of the non-dissipative member in accordance to Eurocode 3, taking into account the interaction of the buckling resistance with the bending moment, , defined as its design value in the seismic design situation; is the axial force in the member due to the non-seismic actions included in the combination of actions for the seismic design situation; is the material overstrenght factor, with a recommended value of 1.25; is the system overstrenght, defined as the minimum value of of the dissipative elements; and is the design value of the axial force in the member due to the design seismic action. It is important to note that, in addition to the aforementioned resistance condition, the requirements of Eurocode 3 for steel elements must be followed in the safety checks of the non-dissipative elements.

|  |  |
| --- | --- |
|  | (5) |

## Limitations to the design of concentrically X-Braced steel frames to Eurocode 8

It is important to acknowledge that the application of Eurocode 8 to the seismic design of concentrically X-braced frames introduces a number of difficulties. Due to the need to verify multiple design criteria that can strongly undermine the design process. In the following paragraphs these limitations are detailed and discussed.

The first issue concerns the tension-only design approach specified by the code for the analysis of the structural system, in which the dissipative members under compression are not considered in the analysis. From an analysis standpoint, it becomes exceedingly problematic to achieve this requirement for tall structures or structures with complex geometry in plan and/or elevation. Whilst, for a given set of lateral seismic loads, it may be relatively easy to identify which diagonals are under compression (multi-storey single frame, regular in height and elevation), the same may not be possible with added structural complexity (multi-storey, multi frame 3D model, irregular in plan and elevation). Although the development of buckling phenomena in compressed steel members can greatly decrease its resistance, other seismic design codes follow different design strategies (e.g. CSA-S16-09 [CSA, 2009], ANSI/AISC 341-10 [AISC, 2010]). Not only are the aforementioned structural analysis issues avoided, but lighter design solutions are obtained in comparison with the European methodology (Tremblay and Elghazouli [2011]). The approach of the Canadian and American codes is further supported by different research contributions (e.g. Black et al. [1980], Tremblay [2002], Shaback and Brown [2003], Nip et al. [2013]), in which the authors highlighted the ability of the bracing system to resist a significant number of cycles before collapse, for a controlled range of both the global and local (cross-sectional) slenderness of the member As shown by Nip et al. [2013] for square hollow steel sections, the cross-sectional slenderness (width-to-thickness ratio) may have an important role on the behaviour of the braces, as low values of local slenderness provide high resistance to local buckling and can delay the occurrence of fracture. A more thorough discussion of the similarities and differences between the design procedures and earthquake responses of steel CBFs designed to the European and American guidelines can be found on Azad et al. [2017].

Another important difficulty is related with the relationship between the requirements of EC8-1 regarding and , given by equation (1) and (3), respectively. In order to meet these conditions for the steel diagonals (if at all possible to follow them) one is led to oversized design solutions. In fact, when the axial demand in the bracing members is minimal, which typically occurs on the diagonals of the top storey, relatively slender elements are required in order to guarantee the safety checks of the diagonals, following equation (5). However, the upper bound limitation of the non-dimensional slenderness, , must be followed, leading to oversized bracings given the axial demand of the member. Although this, in itself, poses minor consequences in the cost of the structure, as only the top storey bracings would have more than needed stockiness, the overstrenght coefficient, , of these bracings will be significantly larger than 1.0, given that the members are oversized. However, the design of the diagonals in the remaining storeys is typically governed by the verification of axial capacity versus axial demand (equation (5)), thus meaning that these members are characterized by low values of , which in some cases may be close to 1.0. In order to guarantee the requirement of homogenous dissipative behaviour along the structures’ height (equation (3)), every bracing member must become oversized, due to the value of the diagonals at the top storey. It is important to note that, since the system overstrenght coefficient, , is the minimum value of of the dissipative members (diagonals), than this parameter becomes affected by the requirements of and , thus influencing the design of non-dissipative elements (equation (5)).

It is clear that these limitations give rise to oversized structural CBF solutions, sometimes even larger than in the case of an elastic, non-dissipative design approach. In addition to the obvious   
cost-ineffectiveness of the design outcome, significant force demands are also imposed on connections and foundations as a consequence of member oversizing. Thus, the need to assess the seismic performance of these structural systems arises, whist following more consistent design requirements of Eurocode 8.

## Seismic performance assessment to Eurocode 8

Eurocode 8 – Part 3 [CEN, 2005b] provides the criteria for the evaluation of the seismic performance of existing building structures, in addition to the requirements for the design of retrofitting measures. It is important to note that the provisions of the European assessment code for steel structures were mainly adapted from the American counterparts (FEMA-350 [FEMA, 2000] and ASCE/SEI 41-06 [ASCE, 2006]), and follow a completely different methodology than the one prescribed in EC8-1. According to EC8-3, fundamental requirements referring to the state of damage in the structure must be addressed (limit states), namely Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC). The return periods associated with these limit states are 225 years for DL, 475 years for SD and 2475 years for NC. For braced frames (except for eccentric systems), Part 3 of EC8 prescribes member inelastic deformation capacities for each limit state, as a function of and . Whilst the inelastic deformation capacity in compression of the braces is expressed in relation to the classification of the cross-section, in accordance to Table 3, in tension only one condition per limit state is provided by the code (Table 4). It is important to note that whilst the code is clear in defining what is , the same cannot be stated to what concerns . As one might infer from the code, there are two possible interpretations to this parameter, namely: 1) the axial deformation of the brace by imposing the theoretical axial resistance of a perfect member (Euler buckling critical load); 2) the axial deformation of the brace obtained by considering an imperfect member. Although both interpretations might be similar for very robust members, in which buckling under compression is not as prone to occur, this is an important point of discussion for slender members.

Table 3 – Axial deformation capacity of braces in compression according to EC8-3.

|  |  |  |  |
| --- | --- | --- | --- |
| ***Class of cross-section*** | ***Limit state*** | | |
| ***DL*** | ***SD*** | ***NC*** |
| 1 |  |  |  |
| 2 |  |  |  |

Table 4 – Axial deformation capacity of braces in tension according to EC8-3.

|  |  |  |
| --- | --- | --- |
| ***Limit state*** | | |
| ***DL*** | ***SD*** | ***NC*** |
|  |  |  |

Specifically in the context of the seismic performance of EC8-compliant steel CBFs, a very recent contribution of Del Gobbo et al. [2018] should be noted. The authors exposed the limitations of the damage limitation methodology of the code, which might entail severe amplification of repair costs following ULS and SLS design earthquake events. Furthermore, extensive damage to acceleration-sensitive non-structural contents, which fall outside the scope of the aforementioned methodology for damage control, was reported. Although these observations, as highlighted by the authors, clearly draw attention to the need for structural design procedures which enhance non-structural seismic performance, the research work detailed in this paper focuses only on the modification of the existing design provisions of EC8-1. A larger shift on the design philosophy itself, although increasingly more evident in the literature, falls outside the scope of the research conducted herein.

# numerical investigation

Following the discussed limitations of EC8-1 regarding the seismic design of X-Braced steel frames, a number of possible amendments to the requirements of the code are detailed in this section. These modifications were used in a parametric study.

## Possible amendments to Eurocode 8

As already mentioned, there are some issues related to the design of X-Braced steel frames to EC8-1. Whilst still considering the methodology of the code, a number of possible amendments to the requirements of the current version of the document are evaluated hereafter. The first modification concerns the relaxation of the upper limit of the non-dimensional slenderness, , considering that this parameter not only governs the design of the diagonals of the top storey, but also, ultimately, influences the design of the remaining dissipative elements, as discussed in Section 2.2. This is attained by increasing the maximum limit value to 2.5, whilst maintaining the lower bound requirement (1.3). The remaining code amendments relate to the homogenous dissipative behaviour along the structure, namely the requirement shown in equation (3). Whilst the second proposed modification aims to disregard the dissipative elements of the top storey for the calculation of the ratio, the third variation concerns the relaxation of the maximum allowed value of this ratio to 1.5. Lastly, the fourth modification consists on verifying equation (3) only between adjacent storeys (e.g. for the first storey, consider the first and second storey diagonals; for the second storey, consider the first, second and third storey diagonals), rather than considering all the diagonals of the structure. Table 5 summarizes the possible amendments to EC8-1, henceforth referred as “variants”, to what concerns the seismic design of concentrically X-Braced steel frames.

Table 5 – Proposed ammendments to EC8-1 for X-CBFs.

|  |  |  |
| --- | --- | --- |
| ***Amendment*** | ***EC8 clause*** | ***Description*** |
| Variant 1 | Cl. 6.7.3 (1)  () | Modification of the upper limit value from 2.0 to 2.5 |
| Variant 2 | Cl. 6.7.3 (8)  () | Disregard the diagonals of the top storey for the verification of the clause |
| Variant 3 | Modification of the maximum allowed value from 1.25 to 1.5 |
| Variant 4 | Apply the verification of the clause between adjacent storeys instead of the whole structure |

It is important to note that the considered amendments were applied independently in the design of the frames presented in the next section. Whilst the clause of EC8-1 related to a given variant is modified according to the description presented in Table 5, the remaining methodology of the code is strictly followed. In order to have a base for comparison of the obtained designs, the seismic design in full accordance with Eurocode 8 was also considered in this research study.

## Seismic design of the archetype frames

In order to assess the influence of the proposed variants on the seismic performance of X-CBFs, a number of archetypes were defined. These consist of combinations of two different concentrically   
X-braced frame configurations, with three different number of storeys, in combination with the five seismic design variants (four proposed variants and the design in full accordance with EC8-1). In total, archetypes were considered in this research study.

To what concerns the steel frame configurations, the braced frames are considered to be part of one of two different buildings, in accordance to Figure 1 and Figure 2. Whilst in both buildings the braced frames are positioned in the vertical direction (y-axis) at the edges of the structure, the main difference between the buildings is the number of braced bays (highlighted in red). In Figure 1 only the middle 8.0m span is braced (thus designated as Y1 brace configuration), and in Figure 2 both 8.0m end spans are braced (thus designated as Y2 brace configuration).

|  |  |
| --- | --- |
|  |  |
| Figure 1 – Plan view of the Y1 brace configuration building. | Figure 2 – Plan view of the Y2 brace configuration building. |

|  |  |
| --- | --- |
|  |  |
| Figure 3 – Elevation view of the 4Y1 braced frames. | Figure 4 – Elevation view of the 4Y2 braced frames. |

Depending on the number of storeys, different storey heights were considered for the archetypes. Both 4, 8 and 12 storeys were adopted for the current study, with a common 4.0m of height for the first storey and 3.5m for the remaining storeys. In order to simplify the designation of each archetype, each parametric case study label is a simple concatenation of the number of storeys (4, 8 or 12), the brace configuration (Y1 or Y2), and the design variant (EC8, VAR1, VAR2, VAR3 and VAR4).   
Figure 3 and Figure 4 show an elevation view of archetypes with (4 storey frame with brace configuration Y1) and 4Y2 (4 storey frame with brace configuration Y2), respectively.

A summary of the vertical distributed loading is shown in Table 6, where is the permanent and is the imposed load. These loads are consistent with typical values associated with office buildings. The transmission of vertical loads to the centre span of the braced frames was considered as point loads at each storey level, following the positioning of the secondary beams. Moreover, in order to calculate the storey masses for seismic design, load combination was considered for the intermediate storey and for the top storey, following the requirements of Eurocode 8. The slabs are considered to act as rigid diaphragms, thus, each storey mass can be equally distributed by the two braced frames of the building, as shown in Table 6.

Table 6 – Vertical loads.

|  |  |  |  |
| --- | --- | --- | --- |
| ***Storey*** | ***Load type*** | ***Load***  ***[kN/m2]*** | ***Frame storey mass***  ***[t]*** |
| Top storey |  |  |  |
|  |  |
| Intermediate storey |  |  |  |
|  |  |

In the designs, an S275 steel grade was considered for the beams and diagonals members, whilst S355 was adopted for the columns. European standard steel sections were adopted for the steel members, namely H open sections for the beams and columns and SHS (square hollow sections) for the braces. The buildings are considered to be located in the city of Lisbon. 5% damping and an importance factor, , of 1.0 were used to quantify the design seismic action. The parameters required for the definition of the elastic response spectra of the Portuguese National Annex of EC8 are shown in Table 7.

Table 7 – Elastic response spectra parameters.

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| ***Spectrum*** | ***Ground type*** | ***[m/s2]*** | ***S*** | ***[s]*** | ***[s]*** | ***[s]*** |
| Type 1 | B |  |  |  |  |  |
| Type 2 |  |  |  |  |  |

Seismic design was performed avoiding having to take into account any second-order effects   
( effects), by limiting the maximum value of the interstorey drift sensitivity coefficient, , to 0.1 (due to the low-deformability of X-CBFs, the seismic design of these structures is typically not governed by the verification of , and values well below 0.1 are commonly obtained). The damage limitation performance requirement was considered in the seismic design of the archetypes by limiting the interstorey drift to 0.75% of the inter-storey height. All archetypes were designed using modal response spectrum analysis, and a medium ductility class (DCM) was assumed. The adopted behaviour factor for each case study was estimated on the basis of the IFBD methodology [Vilanni et al., 2009]). The seismic design of the archetypes was based on initial structural solutions designed to resist gravity loads only, taking into account the relevant load combinations of Eurocode 0 [CEN, 2010] and the requirements of Eurocode 3 [CEN, 2005a] for steel buildings.

## Preliminary comparison of the design solutions

Having performed the seismic design of the archetypes according to EC8-1 (and the proposed variants), a preliminary comparison of the obtained structural solutions can be made. This comparison is herein conducted in terms of the lateral stiffness of the frames (correlated with the evaluation of the fundamental period, ), the behaviour factor (related to the amount of dissipative behaviour that can be expected to occur under the design earthquake) adopted in the design according to IFBD, and the total steel weight of the archetypes. Table 8 shows a summary of this comparison for the 30 archetypes considered in this research study.

The first important observation concerns the influence of the design variants on the flexibility of the braced frames. As one may infer from Table 8, all the archetypes designed in accordance to the proposed variants exhibit higher values of in comparison to the EC8-1 compliant solution, thus meaning that the modifications to the code result in more flexible structures. As already pointed out in this paper, the proposed variants target the reduction of oversized structural designs, meaning that lighter section are able to be used both for the dissipative and non-dissipative members. This implies that the equivalent seismic forces obtained with the use of the elastic spectrum will be lower, reducing force demands on both the connections and foundations.

It is also important to note the clear trends related to the influence of the different variants in the dynamic properties of the archetypes. Regarding the 4-storey X-CBFs (with both Y1 and Y2 configurations), Variant 2 (disregard the diagonals of the top storey for the verification of ) yields the highest influence on the fundamental period of the structure, in comparison to the reference scenario (EC8 compliant). Variant 1 (modification of the upper limit value of from 2.0 to 2.5), Variant 3 (modification of the maximum allowed value of from 1.25 to 1.5) and Variant 4 (apply the verification of between adjacent storeys instead of the whole structure) amount to identical structural solutions in terms of lateral stiffness. As for the 8-storey and 12-storey archetypes, Variants 2 and 4 yield the uppermost difference (with generally similar results between the variants), whilst Variants 1 and 3 amount to lower differences (with identical results between the variants) to the fundamental period of the reference design.

Table 8 – Dynamic properties and steel weight sumamry.

|  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- |
| ***Archetype*** | ***[s]*** |  | ***Steel weight***  ***[t]*** |  | ***Archetype*** | ***[s]*** |  | ***Steel weight***  ***[t]*** |
| 4Y1\_EC8 |  |  |  |  | 4Y2\_EC8 |  |  |  |
| 4Y1\_VAR1 |  |  |  |  | 4Y2\_VAR1 |  |  |  |
| 4Y1\_VAR2 |  |  |  |  | 4Y2\_VAR2 |  |  |  |
| 4Y1\_VAR3 |  |  |  |  | 4Y2\_VAR3 |  |  |  |
| 4Y1\_VAR4 |  |  |  |  | 4Y2\_VAR4 |  |  |  |
|  |  |  |  |  |  |  |  |  |
| 8Y1\_EC8 |  |  |  |  | 8Y2\_EC8 |  |  |  |
| 8Y1\_VAR1 |  |  |  |  | 8Y2\_VAR1 |  |  |  |
| 8Y1\_VAR2 |  |  |  |  | 8Y2\_VAR2 |  |  |  |
| 8Y1\_VAR3 |  |  |  |  | 8Y2\_VAR3 |  |  |  |
| 8Y1\_VAR4 |  |  |  |  | 8Y2\_VAR4 |  |  |  |
|  |  |  |  |  |  |  |  |  |
| 12Y1\_EC8 |  |  |  |  | 12Y2\_EC8 |  |  |  |
| 12Y1\_VAR1 |  |  |  |  | 12Y2\_VAR1 |  |  |  |
| 12Y1\_VAR2 |  |  |  |  | 12Y2\_VAR2 |  |  |  |
| 12Y1\_VAR3 |  |  |  |  | 12Y2\_VAR3 |  |  |  |
| 12Y1\_VAR4 |  |  |  |  | 12Y2\_VAR4 |  |  |  |

To what concerns the consequence of the proposed variants on the level of dissipative behaviour of the CBFs, evaluated in terms of the value of behaviour factor adopted, whilst for the 4-storey archetypes Variant 2 is associated to the highest differences in comparison to the reference scenario, for the 8-storey and 12-storey structures it is Variant 4 that results in the highest difference of the behaviour factor in relation to the EC8 design. It is important to note that all the proposed variants yield higher values of in comparison to the EC8 design of the archetypes. This means that the response of these structures will further explore the material inelastic behaviour, in comparison to a structure designed in full accordance to EC8. As one may infer from the results of Table 8, some of the designs amount to a behaviour factor below 1.0 with the use of the IFBD procedure (e.g. 8Y2\_VAR1, 12Y2\_VAR3). This means that the initial structural solution of the design (performed for gravity load resistance) is able to sustain the equivalent elastic seismic forces. According to the IFBD, the multiple of the elastic seismic forces that produce the first plastic hinge in the dissipative members must be calculated, as it leads directly (or rather, inversely) to the behaviour factor. In scenarios in which this calculation leads to a multiple higher than 1.0 (thus a lower than 1.0), the seismic forces obtained with the elastic response spectrum do not lead to the formation of any plasticity in the system, and thus no inelastic dissipation of energy should occur.

Finally, it is important to note the consequence of using the proposed variants to the requirements of Eurocode 8, to what concerns the overall steel weight of the archetypes. Figure 5 shows a comparison of the steel weight corresponding to the cases listed in Table 8, normalized by dividing the steel weight of the archetype by the corresponding reference scenarios (EC8 designs). It is important to note that this comparison is made on the basis of the material quantity associated with the main structural members of the 2D X-CBF frames, and does not account for any other components of the structural system (e.g. secondary beams in the transverse direction, connections).

|  |
| --- |
|  |
| Figure 5 – Normalized steel weight comparison. |

As one might infer from Figure 5, the proposed variants lead to reductions in material quantities of the structural members ranging between and %, in comparison to the reference scenario. As expected, the application of the proposed modifications to Y2 frame configuration leads , generally, to more substantial steel savings than Y1, due to the fact that, although both configurations are similar, the number of braced bays in Y2 is higher than in Y1 (therefore with a larger quantities of dissipative members). It is also important to note that Variants 2 and 4 (disregard the diagonals of the top storey in the verification of and application of the verification of between adjacent storeys instead of the whole structure, respectively) lead to lighter steel weight reductions. Similar observations regarding the effect of these variants on the lateral stiffness and behaviour factor of the archetypes, might suggest that these variants have a more significant influence on the seismic design of X-CBFs to Eurocode 8 than Variants 1 and 3 ( and , respectively). Based on these observations, one might presume that similar consequences should occur regarding the effect of the proposed variants on the seismic performance of the archetypes.

# Seismic performance assessment

Having defined the X-CBF archetypes in accordance to the seismic design variants of the parametric study, it is important to assess the seismic performance of the structures, thus evaluating if the behaviour of the frames is compromised by the possible modifications to the European seismic code. In order to achieve an accurate representation of the structural behaviour of the archetypes, a numerical model was developed and is herein detailed. In the following sections, this model is used to simulate the response of the CBFs to both nonlinear static (pushover) and dynamic (response-history) loading conditions.

## Numerical modelling

The seismic performance assessment of the X-CBFs was performed in OpenSees [PEER, 2006] by adopting a simplified numerical modelling approach. For the 4-storey and 8-storey frames,   
non-dissipative elements were simulated with distributed plasticity, with a single inelastic force-based (FB) beam-column element per structural member, with 10 integration points (IPs) per element. Archetypes with 12 storeys were modelled with more detail, with one FB element (10 IPs) for the beams and 10 FB elements (10 IPs) for the columns. At base-storey level, the columns were considered to be pinned at the bottom end. The steel material behaviour for beams and columns was simulated with the bilinear *Hardening* *Material* model with a 0.5% strain-hardening ratio.

To what concerns the modelling of the braces, these members were simulated with a distributed plasticity approach, 10 FB elements along the length (10 IPs). The behaviour of the brace-to-frame gusset connections was not simulated in the model, assuming bracings that are articulated at both member ends using *zeroLenght* elements. The behaviour of the braces under compression was accurately replicated on the numerical model, particularly due to the simulation an initial deformed shape of the member, allowing for the development of global buckling phenomena. An initial deformed triangular shape with a maximum mid-span imperfection of , where is the length of the brace, was considered. Braces were modelled with the *Steel02* *Material* model with a 0.3% strain-hardening ratio for the nonlinear static analysis and the low-cycle fatigue constitutive model *Fatigue* *Material* model. The defining parameters of this model were determined with the methodology proposed by Hsiao et al. [2012].

Rigid floor diaphragms were simulated through an equal degree-of-freedom approach, in addition to horizontal rigid elastic truss elements connecting the end nodes of all steel beams. effects associated with the gravity frames were considered with a lean-column approach.

## Nonlinear static analysis

In order to identify potential behavioural problems of the proposed variants to the seismic design to EC8, the numerical response of the archetype X-CBFs to nonlinear static (pushover) lateral loading conditions may be used. To this end, an assessment of the behaviour of the frames was carried out by applying a lateral load pattern proportional to the mass and to the fundamental vibration mode. In Figure 6, the lateral load-deformation behaviour of the archetypes is shown in terms of capacity curves representing the total base shear versus the global drift ratio. The results are shown for both frame configurations (Y1 and Y2) and the three considered storey levels (4, 8 and 12 storeys), taking into consideration the five seismic design variants (EC8 or fully compatible seismic design to Eurocode 8, and the four proposed amendments to the European code, VAR1 to VAR4, listed in Table 5).

|  |  |  |  |
| --- | --- | --- | --- |
|  | | **Frame configuration** | |
| **Y1** | **Y2** |
| **Storeys** | **4** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\01 - Pushover\4Y1__pushover.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\01 - Pushover\4Y2__pushover.tiff** |
| **8** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\01 - Pushover\8Y1__pushover.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\01 - Pushover\8Y2__pushover.tiff** |
| **12** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\01 - Pushover\12Y1__pushover.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\01 - Pushover\12Y2__pushover.tiff** |
|  |  | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\01 - Pushover\_legend.tiff** | |
|  | Figure 6 – Pushover load-deformation behaviour of the archetypes. | | |

As one may infer from Figure 6, the consideration of different modifications to the provisions of Part 1 of Eurocode 8 has clear effects on the lateral behaviour of the structures. All EC8 compliant archetypes exhibited stiffer and stronger characteristics in comparison to the proposed variants. This occurs due to the fact that the design to EC8 tends to lead to oversized structural solutions, whilst with the use of the proposed variants, lighter (and therefore more flexible) structures are obtained. It is important to note that the post-yield branches of the pushover curves showed identical gradients between the design variants, particularly between EC8 and Variants 1, 2 and 3. Although all archetypes showed negative post-yield behaviour branches, Variant 4 exhibited rapid branch gradient increase for levels of lateral deformation close to 1.5% of global drift ratio (particularly for 12Y1 and 12Y2 archetypes). This indicates the formation of undesirable structural collapse mechanisms, such as the development of a soft-storey mechanism. In order to have a more detailed look into the lateral deformations along the height of the archetypes, the inter-storey drift ratios at a global drift ratio of 1% were obtained, as shown in Figure 7.

|  |  |  |  |
| --- | --- | --- | --- |
|  | | **Frame configuration** | |
| **Y1** | **Y2** |
| **Storeys** | **4** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\02 - Pushover 1% Global Drift Ratio\4Y1_globaldriftratio.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\02 - Pushover 1% Global Drift Ratio\4Y2_globaldriftratio.tiff** |
| **8** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\02 - Pushover 1% Global Drift Ratio\8Y1_globaldriftratio.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\02 - Pushover 1% Global Drift Ratio\8Y2_globaldriftratio.tiff** |
| **12** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\02 - Pushover 1% Global Drift Ratio\12Y1_globaldriftratio.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\02 - Pushover 1% Global Drift Ratio\12Y2_globaldriftratio.tiff** |
|  |  | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV2\01 - Pushover\_legend.tiff** | |
|  | Figure 7 – Inter-storey drift ratios of the archetypes at 1% global drift ratio. | | |

As expected, the analysis of the results shown in Figure 7 allows confirming that Variant 4 generally showed unsatisfactory lateral behaviour, as it leads to the development of a soft-storey collapse mechanism (in particular for archetypes with 8 and 12 storeys). Although the remaining variants showed similar lateral deformations to those concerning the design to the current requirements of EC8, Variant 2 exhibited higher concentration of deformation in the intermediate storeys for archetypes 12Y1 and 12Y2. This indicates higher levels of damage concentration. It is important to note that these observations clearly point towards the conclusions of Section 3.3 of this paper, in which the preliminary comparisons suggested that Variants 2 and 4 are substantially more impactful in the seismic design of X-CBFs to Eurocode 8 than Variants 1 and 3. Based on the nonlinear static behaviour of the archetypes, Variants 2 and 4 (disregard the diagonals of the top storey for the verification of and application of the verification of between adjacent storeys instead of the whole structure, respectively) are discarded as possible modifications to the European seismic design code, as they lead to undesirable behaviour of the X-CBFs.

## Nonlinear response-history analysis

## Methodology

In order to further evaluate the seismic behaviour of the archetypes to real earthquake events, an assessment of the nonlinear response of the considered X-CBFs was carried-out. The SelEQ tool [Macedo and Castro, 2017] was employed to select groups of ground motions from real earthquake events, which were scaled in order to have spectral shape compatibility between the average ground motion spectrum and elastic response spectrum adopted in the design [Araújo et al., 2016]. In order to ensure full compatibility between and ( is the fundamental period of vibration of the structure), one group of ground motions should be defined for each combination of frame configuration and number of storeys (4Y1, 4Y2, 8Y1, 8Y2, 12Y1 and 12Y2). Taking into consideration that some of these combinations have similar values of , a total of four ground motion groups were defined for this assessment (4Y1 and 8Y2 were considered to have the same group, as was the case with 8Y1 and 12Y2). Figure 8 shows, for each ground motion group, the average ground motion spectrum and the targeted Eurocode 8 spectrum, in addition to the relevant period range (between and ).

|  |  |
| --- | --- |
| **Ground motion group 1 (4Y2)** | **Ground motion group 2 (4Y1, 8Y2)** |
| D:\Google Drive\Antonio Silva\Luis Santos\Analises\Sinais\V1\Group1\Spectra.tiff | D:\Google Drive\Antonio Silva\Luis Santos\Analises\Sinais\V1\Group2\Spectra.tiff |
|  |  |
| **Ground motion group 3 (8Y1, 12Y2)** | **Ground motion group 4 (12Y1)** |
| D:\Google Drive\Antonio Silva\Luis Santos\Analises\Sinais\V1\Group3\Spectra.tiff | D:\Google Drive\Antonio Silva\Luis Santos\Analises\Sinais\V1\Group4\Spectra.tiff |
| Figure 8 – Acceleration response spectra of the sets of ground motion records. | |

In accordance to the requirements of Eurocode 8, a total of ten ground motions were selected for each group, thus meaning that the average between the numerical responses of the structure to each ground motion may be considered to accurately replicate the response to the design value of the action effect.

## Verification of the performance requirements of Eurocode 8 – Part 1

To what concerns the verification of the serviceability limit state (SLS) of EC8-1, formally designated by damage limitation limit state, Figure 9 (frame configuration Y1) shows the inter-storey drift ratio distribution of the archetypes designed according to EC8, in addition to Variants 1 and 3 ( and , respectively). For this comparison, the ground motions were scaled to match the design seismic intensity for the verification of the SLS requirement. In the plots, the average inter-storey drift ratio distribution of the ground motion group is compared with the EC8-1 SLS limit considered in the design (0.75% of the interstorey height) of the archetypes.

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | | **Variant** | | |
| **EC8** | **VAR1** | **VAR3** |
| **Storeys** | **4** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\03 - Time-History ELS Drift\inter-storey drift__4Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\03 - Time-History ELS Drift\inter-storey drift__4Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\03 - Time-History ELS Drift\inter-storey drift__4Y1_3.tiff** |
| **8** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\03 - Time-History ELS Drift\inter-storey drift__8Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\03 - Time-History ELS Drift\inter-storey drift__8Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\03 - Time-History ELS Drift\inter-storey drift__8Y1_3.tiff** |
| **12** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\03 - Time-History ELS Drift\inter-storey drift__12Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\03 - Time-History ELS Drift\inter-storey drift__12Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\03 - Time-History ELS Drift\inter-storey drift__12Y1_3.tiff** |
|  |  | **D:\Analyses\_Plots\_labels\__.tif** | | |
|  | Figure 9 – Interstorey drift ratios and EC8-1 limit of Y1 archetypes for the SLS seismic level. | | | |

The analysis of the results depicted in Figure 9 allows concluding that all Y1 archetypes, i.e. seismically designed in full accordance with the European code and with the proposed Variants 1 and 3, verify the SLS limit of EC8-1 regarding the damage limitation requirement. Regarding the Y2 X-CBFs, the typology of the bracings in these archetypes (two X-braced spans) makes for much stronger frames than only one X-braced span (Y1). Thus, these archetypes further validate the verification of the EC8-1 SLS limit, as the range of interstorey drift ratios is even lower than that those observed for the Y1 archetypes. It is important to note that, as shown by the results obtained from the nonlinear static analysis, the nonlinear dynamic behaviour of the archetypes is generally identical between the design variants. This points towards the efficiency of the proposed amendments to the code (Variants 1 and 3), considering that, despite leading to lighter, more flexible and more dissipative structures, the intended seismic performance is not compromised.

Regarding a ground motion intensity equivalent to the design seismic intensity used for the verification of the ultimate limit state of EC8-1 (ULS), Figure 10 and Figure 11 show the interstorey drift patterns obtained for frame configurations Y1 and Y2, respectively. As one may conclude, once again the average deformation distribution of the X-CBFs is identical between the archetypes fully designed to the current version of the seismic code, and with the proposed Variants 1 and 3. It is also worth noting that despite allowing for a less homogenous dissipative behaviour along the CBF height, the use of Variant 3 ( instead of ) does not lead to significant differences concerning damage concentration in comparison to EC8 and VAR1 archetypes. Thus, it is possible to conclude that the proposed modifications (Variants 1 and 3) to the current requirements of EC8-1 for X-CBFs lead to satisfactory, and generally identical, seismic behaviour.

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | | **Variant** | | |
| **EC8** | **VAR1** | **VAR3** |
| **Storeys** | **4** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__4Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__4Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__4Y1_3.tiff** |
| **8** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__8Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__8Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__8Y1_3.tiff** |
| **12** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__12Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__12Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__12Y1_3.tiff** |
|  |  | **D:\Analyses\_Plots\_labels\_.tiff** | | |
|  | Figure 10 – Interstorey drift ratios of Y1 archetypes for the ULS seismic level. | | | |

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | | **Variant** | | |
| **EC8** | **VAR1** | **VAR3** |
| **Storeys** | **4** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__4Y2_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__4Y2_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__4Y2_3.tiff** |
| **8** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__8Y2_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__8Y2_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__8Y2_3.tiff** |
| **12** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__12Y2_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__12Y2_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\04 - Time-History ELU Drift\inter-storey drift__12Y2_3.tiff** |
|  |  | **D:\Analyses\_Plots\_labels\_.tiff** | | |
|  | Figure 11 – Interstorey drift ratios of Y2 archetypes for the ULS seismic level. | | | |

## Verification of the performance requirements of Eurocode 8 – Part 3

In addition to the comparisons with the requirements of EC8-1, the seismic response of the archetype X-CBFs (in particular the ductility demands of the dissipative members) was also compared with the requirements of EC8-3 for existing buildings and for different limit states (Damage Limitation or DL, Significant Damage or SD and Near Collapse or NC). This was attained by determining, for each imposed ground motion, the maximum axial deformation of the braces in each storey. Different methods can be used in order to calculate the axial deformation of the dissipative members in the response-history analysis: i) directly, i.e. the sum of the axial elongations of each finite element used to simulate the brace member and ii) indirectly, i.e. making use of the measured displacements at the floor level, considering the horizontal and/or vertical components of the main frames nodes.

It is important to note that differences in the calculated brace elongations may arise from the use of the different calculation methods, particularly for the upper storey braces on taller higher structures. In such cases, the elongation of the brace indirectly determined with the horizontal component of the displacement of the main nodes may be significantly reduced by the vertical displacement component. Additionally, whilst with the use of the direct method to obtain brace elongation takes into account the occurrence of buckling under compression, the indirect method does not capture this phenomenon as only the displacements of the end nodes of the braces are considered.

In order to highlight the aforementioned observations, Figure 12 shows a comparison of the effect of the different axial deformation calculation methods, for one steel brace located at the top-storey of archetype 8Y1\_VAR1, and for a single imposed ground motion. In the plots, the results are shown in terms of the variation of the brace axial deformation, , over the duration of the earthquake, for the direct (brace deformation) and indirect methods (drift component X, i.e. accounting only for the horizontal component of the displacements of the end nodes of the braces, and drift component X and Y, i.e. accounting for both the horizontal and vertical components of the displacements of the end nodes of the braces).

|  |
| --- |
| **D:\Google Drive\Antonio Silva\Luis Santos\Analises\_PlotsV3\05 - Brace Elongation Methods\dL comparison__8Y1 Brace Deformation Storey 8.tiff** |
| **D:\Analises\ELU\8Y1\_Plots\_legend.tiff** |
| Figure 12 – Brace axial deformation based on different calculation methods (8Y1\_VAR1). |

As one may infer from Figure 12, the use of only the horizontal component of the floor displacements greatly overestimates the real axial deformation of the braces, for both the maximum deformation in tension and under compression. Accounting for both the horizontal and vertical components of the end node displacements improves the accuracy of the results but is still somewhat inaccurate. Whilst in tension the axial deformation envelope between the direct (brace deformation based on the OpenSees output) and indirect (drift component X and Y) is identical, the same does not occur for compressive deformations. Since the indirect methods only account for the displacements of the main nodes of the frame, they are insensitive to the development of non-linear geometrical phenomena that might occur on the braces (such as buckling under compression). However, with the use of directly generated output from OpenSees [PEER, 2006], it is possible to know at which level of compressive deformation does buckling of the brace starts to occur, as the member does not exceed a certain value of negative (Figure 12, brace deformation).

In the following paragraphs, the brace axial deformations are compared with the limits defined in EC8-3 for existing structures, as a function of the ductility demand of the braces, . This parameter is calculated by dividing the axial deformation obtained from the indirect method by the axial deformation of the braces in compression at the buckling load (equation (6)) or in tension at yielding (equation (7)). As mentioned in Section 2.3 of this paper, Part 3 of Eurocode 8 is unclear in the definition of the response parameter for braces under compression (axial deformation of braces in compression at buckling load or ). For the current research study, it was interpreted that this parameter is obtained by considering an imperfect member, and thus an axial resistance governed by buckling phenomena. In the equations, is the yield strength and is the Young modulus of the steel, is the length of the brace and is the buckling reduction factor calculated according to Eurocode 3 [CEN, 2005a].

|  |  |
| --- | --- |
|  | (6) |
|  | (7) |

In order to further demonstrate the influence of the different methods to estimate brace axial deformations, the ductility demands obtained with the three methods are now compared for three of the archetypes considered in this study, fully designed to the requirements of the European code, covering the full range of frame heights evaluated herein, as shown in Figure 13. The response of the braced frame was analysed under the corresponding set of ten ground motion records, and the mean response history obtained was selected as representative of the response of the archetype. Since there is a full compatibility between the return periods of EC8-1 for the ULS and EC8-3 for the SD limit state, no additional scaling of the records was applied. In the plots shown in the figure, the ductility demand limit specified in EC8-3 for the SD seismic level is also shown. As denoted in the results, although the tension ductility demands between the three methods are relatively similar, the same does not occur in compression. As previously observed from Figure 12, estimating brace deformations from relative horizontal floor displacements entails a significant overestimation of the compressive ductility demands imposed to the braces. This is improved with a more realistic evaluation, i.e. accounting for both the horizontal and vertical relative displacements of the extremity nodes of the diagonals. Nevertheless, the results are very different when compared with the brace elongation obtained from OpenSees.

|  |  |  |  |
| --- | --- | --- | --- |
|  | **4Y1\_8** | **8Y1\_8** | **12Y1\_8** |
| **Brace deformation** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\4Y1\Brace ductility method comparison\brace ductility__element__4Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\8Y1\Brace ductility method comparison\brace ductility__element__8Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\12Y1\Brace ductility method comparison\brace ductility__element__12Y1_8.tiff** |
| **Drift component X** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\4Y1\Brace ductility method comparison\brace ductility__x__4Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\8Y1\Brace ductility method comparison\brace ductility__x__8Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\12Y1\Brace ductility method comparison\brace ductility__x__12Y1_8.tiff** |
| **Drift component X and Y** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\4Y1\Brace ductility method comparison\brace ductility__x+y__4Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\8Y1\Brace ductility method comparison\brace ductility__x+y__8Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\12Y1\Brace ductility method comparison\brace ductility__x+y__12Y1_8.tiff** |
|  | **D:\Analyses\_Plots\_labels\SD.tiff** | | |
| Figure 13 – Comparison of brace ductility demands obtained from different estimate methods. | | | |

As already established, different methods can be used to estimate the ductility demands of the bracing diagonals, with the results obtained differing quite substantially in the compressive response range of the members. However, it is important to note that the ductility demand limits currently specified in the European assessment code were derived from experimental testing where the global axial deformation of the brace is usually considered as the main deformation parameter (Tremblay [2002], Goggins et al. [2005]). Hence, the code limits should not be directly compared with the ductility demands derived from the real elongations of the diagonals provided by OpenSees. For consistency, the brace ductility demands should be evaluated based on the horizontal and vertical relative displacements of the extremity nodes of the diagonals. This approach was adopted in this study to evaluate the differences in the ductility demands, and comparison with code-specified limits, for the three design variants investigated in this paper, as demonstrated in the following paragraphs.

Considering a ground motion group scaling compatible with the DL, SD and NC limit states of EC8-3, it is possible to assess if the design of X-CBFs to EC8-1 guarantees the requirements for the seismic performance of existing structures (Table 3 and Table 4), to what concerns the axial deformation of the dissipative steel members. Additionally, it is possible to compare the ductility demands of the braces between the archetypes designed to the current version of the European code and with Variants 1 and 3. Figure 14 (DL), Figure 15 (SD) and Figure 16 (NC) show this comparison for the Y1 archetypes, considering, again, the mean response obtained from the response history analyses conducted with ten ground motion records as representative of the actual response of the archetypes.

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | | **Variant** | | |
| **EC8** | **VAR1** | **VAR3** |
| **Storeys** | **4** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\4Y1\Brace ductility method comparison\brace ductility__x+y__4Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\4Y1\Brace ductility method comparison\brace ductility__x+y__4Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\4Y1\Brace ductility method comparison\brace ductility__x+y__4Y1_3.tiff** |
| **8** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\8Y1\Brace ductility method comparison\brace ductility__x+y__8Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\8Y1\Brace ductility method comparison\brace ductility__x+y__8Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\8Y1\Brace ductility method comparison\brace ductility__x+y__8Y1_3.tiff** |
| **12** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\12Y1\Brace ductility method comparison\brace ductility__x+y__12Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\12Y1\Brace ductility method comparison\brace ductility__x+y__12Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\DL\12Y1\Brace ductility method comparison\brace ductility__x+y__12Y1_3.tiff** |
|  |  | **D:\Analyses\_Plots\_labels\DL.tiff** | | |
|  | Figure 14 – Brace ductility demands and EC8-3 limit of Y1 archetypes for the DL seismic level. | | | |

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | | **Variant** | | |
| **EC8** | **VAR1** | **VAR3** |
| **Storeys** | **4** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\SD\4Y1\Brace ductility method comparison\brace ductility__x+y__4Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\SD\4Y1\Brace ductility method comparison\brace ductility__x+y__4Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\SD\4Y1\Brace ductility method comparison\brace ductility__x+y__4Y1_3.tiff** |
| **8** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\SD\8Y1\Brace ductility method comparison\brace ductility__x+y__8Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\SD\8Y1\Brace ductility method comparison\brace ductility__x+y__8Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\SD\8Y1\Brace ductility method comparison\brace ductility__x+y__8Y1_3.tiff** |
| **12** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\SD\12Y1\Brace ductility method comparison\brace ductility__x+y__12Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\SD\12Y1\Brace ductility method comparison\brace ductility__x+y__12Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\SD\12Y1\Brace ductility method comparison\brace ductility__x+y__12Y1_3.tiff** |
|  |  | **D:\Analyses\_Plots\_labels\SD.tiff** | | |
|  | Figure 15 – Brace ductility demands and EC8-3 limits of Y1 archetypes for the SD seismic level. | | | |

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | | **Variant** | | |
| **EC8** | **VAR1** | **VAR3** |
| **Storeys** | **4** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\NC\4Y1\Brace ductility method comparison\brace ductility__x+y__4Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\NC\4Y1\Brace ductility method comparison\brace ductility__x+y__4Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\NC\4Y1\Brace ductility method comparison\brace ductility__x+y__4Y1_3.tiff** |
| **8** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\NC\8Y1\Brace ductility method comparison\brace ductility__x+y__8Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\NC\8Y1\Brace ductility method comparison\brace ductility__x+y__8Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\NC\8Y1\Brace ductility method comparison\brace ductility__x+y__8Y1_3.tiff** |
| **12** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\NC\12Y1\Brace ductility method comparison\brace ductility__x+y__12Y1_8.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\NC\12Y1\Brace ductility method comparison\brace ductility__x+y__12Y1_1.tiff** | **D:\Google Drive\Antonio Silva\Luis Santos\Analises\EC8-3\_Plots\NC\12Y1\Brace ductility method comparison\brace ductility__x+y__12Y1_3.tiff** |
|  |  | **D:\Analyses\_Plots\_labels\NC.tiff** | | |
|  | Figure 16 – Brace ductility demands and EC8-3 limits of Y1 archetypes for the NC seismic level. | | | |

To what concerns the comparison of design variants, the results shown in Figure 14, Figure 15 and Figure 16 indicate increased ductility demands of the archetypes designed with the amended clauses of the code, in comparison to the archetypes fully designed to Eurocode 8. This was expected, taking into consideration that both amendment scenarios effectively target the reduction of system overstrength levels, entailing that the nonlinear response of the structure is further explored. Even though ductility demands increase, one should recall that these design variants allow for the use of less stocky diagonals, which not only unburdens some of the difficulties imposed by the full compliance with the entire design framework of the code for X-CBFs, but also leads to lighter structural solutions. Furthermore, and as demonstrated in Section 4.3.2 of this paper, such increased ductility demands appear to be of no relevant consequence to the lateral deformation patterns experienced by the structure. Finally, and as already stated in this paper, the considered Y2 X-CBFs are much stronger frames in comparison to the Y1 configuration, thus the obtained ductility demands for these archetypes are much lower than those shown in Figure 14 to Figure 16.

An important observation that can be extracted from the results shown in Figure 14 to Figure 16 is the fact that almost all Y1 archetypes, including the braced frames fully designed to the European code, violate the axial ductility demand limits for braces under compression for the three (DL, SD and NC) limit states defined in EC8-3. Clearly, this indicates some level of inconsistency between the requirements of Parts 1 and 3 of Eurocode 8, considering that a newly designed structure according to EC8-1 does not meet the seismic performance requirements defined for existing steel buildings. This is indicative that some level of conformity between the different European seismic code parts is still lacking.

# Conclusions

In this paper, an improvement of the seismic design of concentrically X-braced steel frames to Eurocode 8 was achieved. From the limited scope of the results obtained, the following conclusions can be withdrawn:

* The requirements of EC8-1 for the seismic design of X-CBFs lead to severe limitations to the design process, to what concerns the compatibility between the different safety verifications prescribed by the European code, in particular the maximum allowed value of the non-dimensional slenderness, , and the homogenous dissipative behaviour related criterion, ;
* A total of four variants were proposed in this research paper, namely the relaxation of the maximum allowed value of (from 2.0 to 2.5) and the maximum value obtained for the ratio (from 1.25 to 1.5), in addition to two independent modifications to the calculation procedure of the latter (disregard the diagonals of the top storey, and application of the verification of the criterion between adjacent storeys instead of the whole structure). In total, 30 archetype X-CBFs were designed according to the different variants proposed;
* The proposed variants proved to have a significant influence on the design solutions, namely in terms of the lateral stiffness and total steel weight of the frames, in addition to the amount of dissipative behaviour that can be expected to occur during the design seismic event. In particular, Variant 2 (disregard the diagonals of the top storey for the verification of ) and Variant 4 (application of the verification of between adjacent storeys instead of the whole structure) proved to be the most impactful;
* The results from non-linear static (pushover) analyses showed that whilst Variants 1 and 3 ( and ) lead to satisfactory behaviour (and generally similar to the behaviour of the archetypes fully-designed to EC8-1), the results obtained for Variants 2 and 4 indicated the formation of undesirable structural collapse mechanisms, such as the development of a soft-storey mechanism, and were therefore discarded;
* The results from non-linear response-history showed that the seismic design to Variants 1 and 3 lead to identical behaviour in comparison to the archetypes fully-designed to EC8-1. This points towards the efficiency of these amendments to the European code, considering that despite leading to lighter, more flexible and more dissipative structures, the intended seismic performance is not compromised;
* The use of relative displacements to quantify the deformation of a brace in compression overestimates the real elongations experienced by the brace. This may result in an overestimation of the brace ductility demand in compression;
* The relaxation of the maximum allowed value of (from 2.0 to 2.5) and the maximum value obtained for the ratio (from 1.25 to 1.5) proved to be suitable modifications to the current design provisions of Eurocode 8. Both amendments lead to generally identical non-linear static and dynamic behaviour, whilst mitigating some of the difficulties associated to the application of EC8 to the design of X-CBFs. Notwithstanding, more comprehensive studies should be conducted to not only validate the conclusions obtained in this research study, but also conclude on which of the two possible modifications is the most appropriate;
* The seismic performance assessment of the frames revealed an inconsistency between the requirements of EC8-1 and EC8-3, as an X-CBF designed according to the current provisions of Part 1 of the European code does not meet the requirements for the seismic performance assessment of existing buildings. This lack of compatibility between the different parts of the European seismic code should be addressed in future research studies.

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