Paper N° 4282 (Abstract ID)
Registration Code: S-T1463001008

SEISMIC DESIGN OF COMPOSITE MOMENT-RESISTING FRAMES WITH CFST MEMBERS

A. Silva[(1)], Y. Jiang[(2)], L. Macedo[(3)], J.M. Castro[(4)], R. Monteiro[(5)] and N. Silvestre[(6)]

[(1)] *Researcher, Faculty of Engineering, University of Porto, Portugal, ajms@fe.up.pt*
[(2)] *PhD Candidate, Istituto Universitario di Studi Superiori di Pavia, Italy, yadong.jiang@umeschool.it*
Researcher, Faculty of Engineering, University of Porto, Portugal, yadong.jiang@fe.up.pt
[(3)] *PhD Candidate, Faculty of Engineering, University of Porto, Portugal, luis.macedo@fe.up.pt*
[(4)] *Assistant Professor, Faculty of Engineering, University of Porto, Portugal, miguel.castro@fe.up.pt*
[(5)] *Assistant Professor, Istituto Universitario di Studi Superiori di Pavia, Italy, ricardo.monteiro@iusspavia.it*
[(6)] *Associate Professor, Technical University of Lisbon, Portugal, nsilvestre@tecnico.ulisboa.pt*

Abstract

The main objective of the research presented in this paper is to gauge the benefits of using concrete-filled steel tube (CFST) members from a seismic design perspective. To this end, a large set of composite moment-resisting (MR) frames was defined and its design was performed according to the requirements of Eurocode 8. In order to outline the set of MR frames, the methodology prescribed in FEMA P695 was used, regarding the definition of the archetypes. The basic performance groups were divided according to two seismic areas in Portugal, and considering frames of 3, 5, 8 and 12 stories. The set of 8 archetypes was designed using combinations of the following criteria: a) CFST columns of circular, square and rectangular cross section; b) behaviour factor (q) calculated according to EC8 ductility class DCM, and according to an Improved Force-Based Design (IFBD) procedure proposed by Villani et al. In total, 48 composite frames were designed. The analysis of the obtained design solutions shows that there are strong limitations when commercial square and rectangular cross-sections are used in seismic design of composite MR frames, as the majority of available members violate the EC8 cross-section slenderness requirements for medium ductility class. However, the opposite conclusion is found for circular CFST columns, as the criterion prescribed in the European code is more relaxed. Additionally, the results show that the consideration of a fixed value of the behaviour factor according to EC8 may greatly defeat its effectiveness, as the members become oversized and the purpose of using q, i.e. taking advantage of the inelastic behaviour of the structure, is compromised. However, if the Improved Force-Based Design (IFBD) procedure is adopted, the designer is able to reach a solution that is different and adjusted for every seismic scenario, with reductions in steel weight of the frame averaging 20% in the 48 designed frames. Additionally, it was also found that the use of circular CFST columns is beneficial to the composite MR frame design, as square and rectangular solutions tend to use more steel for the same archetype. Finally, one composite MR frame with circular CFST columns and an equivalent MRF with steel open profile columns were numerically modelled in OpenSees, and their seismic performance was evaluated through incremental dynamic analysis and collapse fragility assessment. The results obtained clearly indicate that composite moment-resisting frames with concrete-filled steel tube columns have a better seismic performance that an equivalent steel-only structural system.

Keywords: concrete-filled steel tubes; seismic design; Eurocode 8; composite frames; seismic performance assessment

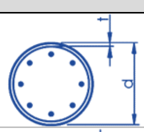
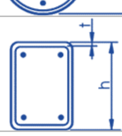
1. Introduction

Concrete filled steel tube (CFST) members have become of particular widespread structural use in high seismicity areas. Such composite members can be fabricated with a wide variety of cross-section typologies (e.g. circular, square, rectangular and elliptical), and have some particular advantages over typical steel or reinforced concrete solutions. The synergy that results from an efficient combination of the two materials, namely by infilling a steel tube with concrete, is reflected in an increase of strength and ductility. In particular, the concrete core has an important contribution in the prevention of inwards local buckling of the steel tube, whilst outwards local buckling is delayed to higher levels of deformation. Based on these considerations, CFST members are seen as an attractive structural solution to adopt in seismic resisting structures due to their ductility and energy dissipation properties under cyclic loading.

The study of the behaviour of beam-column CFST members has become an active research field during the last few decades. Extensive research has been carried out in the past aiming at the characterisation of the behaviour of axially loaded CFSTs (e.g. Schneider [1], Han [2], Sakino et al. [3] and Ellobody et al. [4]). Regarding the flexural behaviour of these elements, research studies such as those carried out by Elchalakani et al. [5], Varma et al. [6], Varma et al. [7] and Han et al. [8] reported ductile behaviour of CFSTs subjected to flexural loading conditions. Additionally, some work has been developed in recent years in the field of CFST members with sustainable infill materials. The use of recycled rubber particles in CFST concrete infill, has been experimentally studied by Duarte et al. [9] for stub columns under compression, by Duarte et al. [10] for stub columns under cyclic bending and by Silva et al. [11] for long circular columns under monotonic and cyclic flexural loading. All studies highlighted the good ductility and overall behaviour exhibited by the composite members. In general, there is extensive research work on the behaviour of concrete filled steel tube members under different loading conditions, notwithstanding some more complex topics that need further development, particularly in seismic performance assessment of composite frames with CFST columns.

From an European design perspective, Eurocode 4 [12] provides the methodology for the calculation of the strength capacity of composite members. Additionally, the code prescriptions aim to prevent the development of local buckling mechanisms in CFST members, before the ultimate loads of the structural system are reached. This is achieved by imposing specific cross-section slenderness limits, as shown in Table 1. Moreover, The European seismic code, Eurocode 8 [13], also adopts the cross-section slenderness for the definition of the ductility class requirements for dissipative elements, as also shown in Table 1, where f_y is the yield strength of the steel tube.

Table 1 – EC4 and EC8 *d/t* and *h/t* limits for CFST members

Cross-section type	***Eurocode 4***	***Eurocode 8***		
		DCM	***DCM***	***DCH***
		$1.5 < q \leq 2$	$2 < q \leq 4$	$q > 4$
	$d/t \leq 90 \times 235/f_y$	$d/t \leq 90 \times 235/f_y$	$d/t \leq 85 \times 235/f_y$	$d/t \leq 80 \times 235/f_y$
	$h/t \leq 52 \times (235/f_y)^{0.5}$	$h/t \leq 52 \times (235/f_y)^{0.5}$	$h/t \leq 38 \times (235/f_y)^{0.5}$	$h/t \leq 24 \times (235/f_y)^{0.5}$

This paper mainly focuses on: 1) the seismic design to EC8 of composite moment-resisting frames (MRFs) with CFST columns; 2) the study of the influence of the use of EC8-prescribed values of q in the design, in comparison to the Improved Force-Based Design (IFBD) methodology [15]; 3) the comparison between the seismic performance of composite MRFs with CFST columns and bare steel MRFs.

2. General framework

The research presented in this paper is part of a research study on sustainable concrete-filled steel tube members with concrete made with recycled rubber aggregates. As such, a comprehensive experimental campaign was carried out in order to characterize the behaviour of CFST columns under simple and combined bending, for both monotonic and cyclic flexural loading conditions. In total, 16 circular CFST members, 12 of which made with rubberized concrete (RuC) and the remaining with standard concrete (StdC), 16 square CFST columns, 12 RuCFST and 4 StdCFST, and 4 rectangular RuCFST specimens were tested resorting to an innovative steel box testing device (Fig. 1 and Fig. 2).

Fig. 1 – Overview of the designed steel box

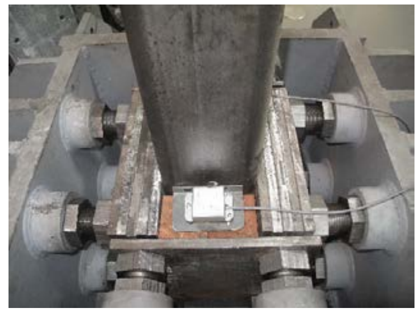


Fig. 2 – Specimen placement in the steel box

The results obtained in this experimental study allowed concluding that such composite members (both RuCFST and StdCFST) exhibit very stable behaviour when subjected to flexural loading conditions. In particular, circular CFSTs exhibited negligible and reduced lateral load degradation when subjected to monotonic and cyclic bending, respectively. This was observed up to significant levels of lateral drift, in some cases reaching 10% of the specimen's free length (1.35m), as illustrated in Fig. 3. Another important observation was that concrete infill does not play a significant role on the flexural behaviour of these composite members. This conclusion was attained through the comparison between the flexural responses of CFST members with fundamentally different concrete infills (concrete strength of 50MPa for StdC, 40MPa for RuC with a 5% aggregate replacement ratio and 20MPa for a 15% aggregate replacement ratio), whilst maintaining the same steel tube (Fig. 4).

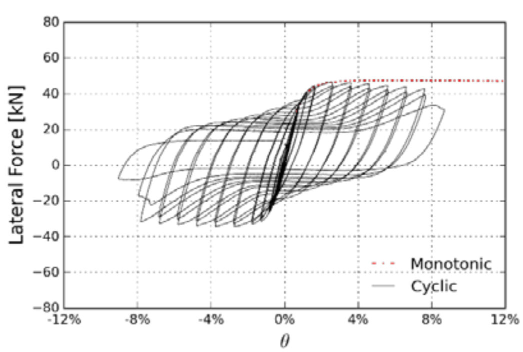


Fig. 3 – Measured monotonic and cyclic behaviour of circular CFSTs

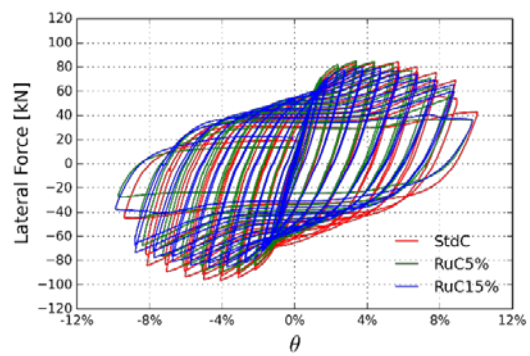


Fig. 4 – Influence of concrete type on the cyclic behaviour of circular CFSTs

The test results also allowed concluding that Eurocode 4 is conservative in predicting the bending capacity of the tested CFST and RuCFST specimens. Average differences of 24% and 13% between the code and the experimental results were identified for circular and square/rectangular specimens, respectively. More importantly, no significant differences between StdC and RuC type members were found regarding the ratios between the maximum bending moment measured in the tests and the code prediction. This confirmed, once again, the reduced influence of concrete type on the flexural behaviour of CFST members. In spite of the differences between the material properties of the standard and rubberized concrete types, it was concluded that the design assumptions of EC4 in the context of RuCFST columns are still valid, confirming in this way the applicability of the European code to the design of both CFST and RuCFST members subjected to bending.

In order to study the seismic performance of moment-resisting frames with CFST columns, a large set of MRFs was defined. Due to the large number of possible frames considered in engineering practice, the methodology prescribed in FEMA P695 [14] was used. In this document, the development of structural system archetypes is described, providing the guidelines for a systematic technique to characterize key features and behaviour related to the performance of a seismic-force-resisting system. Thus, it defines how to develop a set of building configurations that describe the overall range of allowable configurations of a system, which is then separated into groups sharing common features or behavioural characteristics (performance groups).

For the current research study, a number of criteria were considered to define the aforementioned performance groups, namely the seismic intensity and the number of stories. As such, the basic performance groups are divided according to two seismic areas in Portugal (Porto and Lagos), corresponding to regions of low and moderate-to-high seismicity, respectively, and considering frames of 3, 5, 8 and 12 stories. This set of 8 archetypes was then designed using combinations of the following criteria: a) CFST columns with circular, square and rectangular cross section; b) behaviour factor calculated according to EC8 ductility class DCM, and according to the Improved Force-Based Design (IFBD) methodology proposed by Villani et al. [15]. In total, 48 composite frames were designed in accordance to the provisions of the European seismic design code, as shown in Fig. 5.

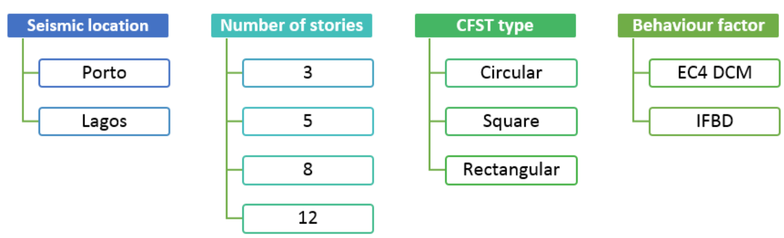


Fig. 5 – Archetype definition criteria

Taking into consideration the wide range of archetype in this study, only a subset will be described in this research paper, namely the results obtained for the 5-storey MRFs (12 of the 48 archetypes).

3. Seismic design of 5-storey composite MRF archetypes

3.1 Design considerations

In this section, the obtained structural designs for the 5-storey composite MRF archetypes are described, and the implication of the use of different tubular cross-sections and behaviour factor definition methodologies is discussed. The frames are part of a 5-storey building structure with the plan layout and elevation shown in Fig. 6.

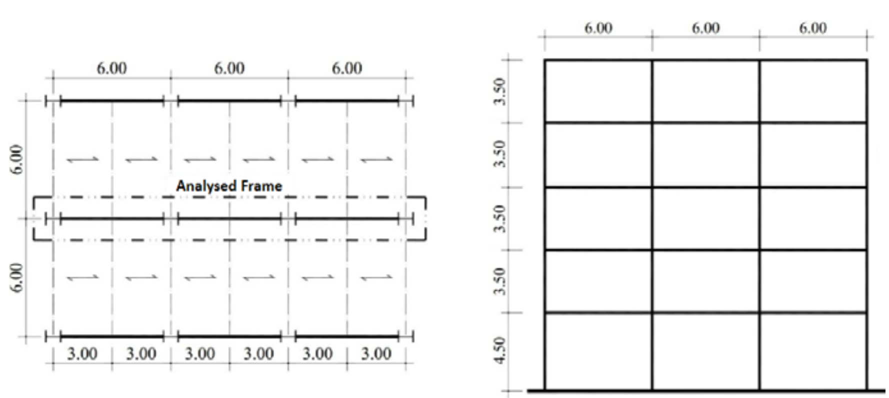


Fig. 6 – Building layout

In the longitudinal (X) direction the seismic resistance is provided by moment-resisting frames spaced at 6 meters. In the transverse (Y) direction the seismic resistance is assured by a bracing system. The investigation will focus on the central moment-resisting frame identified in Fig. 6.

All frames were designed in accordance with Eurocode 8, considering the Portuguese National Annex. A dissipative structural behaviour was assumed in the seismic design of the frames, by considering both a behaviour factor of 4 (corresponding to the medium ductility class, DCM, as defined in EC8) and a behaviour factor calculated according to the IFBD methodology. The steel grade considered for all steel elements (steel beams and steel tubes) was S275, and a concrete class C30/37 was assumed for the concrete infill of the CFST members. European standard steel open sections with I shape (IPE) were used for the steel beams, and commercial steel tubular sections were considered for the CFST members.

A summary of the vertical distributed loading is shown in Table 2, where g_k and qk are the permanent and imposed loads, respectively. The transmission of the vertical loads to the central frame was considered through point loads applied at each storey level, in accordance with the layout of the secondary beams. Additionally, and in order to calculate the storey masses for the seismic analysis and design, a load combination given by g_k + $0.3q_k$ was considered for the intermediate storeys and $g_k + 0.0q_k$ for the top storey, in accordance with the EC8 design requirements. The slabs were considered to act as rigid diaphragms, thus, each storey mass can be equally distributed by the three longitudinal frames, as shown in Table 2. The parameters required for the definition of the elastic response spectra that are specified in the Portuguese National Annex of Eurocode 8 are shown in Table 3.

Table 2 – Vertical distributed loads

Storey	***Load type***	***Load [kN/m^2]***	***Frame storey mass [t]***
Top storey	g_k	4.75	34.20
	q_k	1.00	
Intermediate storey	g_k	5.75	45.72
	q_k	2.00	

Table 3 – Elastic response spectra parameters

Spectrum	***Ground type***	***Location***	***a_g [m/s^2]***	***S***	***T_B [s]***	***T_C [s]***	***T_D [s]***
Type 1	B	Lagos	2.50	1.175	0.10	0.60	2.00
		Porto	0.35	1.350			
Type 2		Lagos	1.70	1.268	0.10	0.25	2.00
		Porto	0.80	1.350			

Seismic design was performed taking into account second-order effects, by limiting the maximum value of the interstorey drift sensitivity coefficient (θ) to 0.2. The EC8 capacity design weak beam-strong column requirement, evaluated at the joint level, was also performed in the design of all frames. Moreover, the damage limitation performance requirement was considered in the seismic design by limiting the inter-storey drift to 0.75% of the storey height. All archetypes were designed based on the modal response spectrum analysis method. It is important to note that all rectangular members are considered to be loaded under the major bending axis.

3.2 Analysis of the design solutions

Based on the design considerations described in the previous section, the seismic design of the composite MRF archetypes was carried-out. Table 4 summarizes the obtained design solutions for the 5-storey frames, in terms of the fundamental period T_1, the total steel weight of the structural members (steel beams and steel tubular columns), and the total volume of concrete of the CFST columns. Moreover, in the cases in which the IFBD methodology was used for the determination of the behaviour factor, the obtained value of *q* is also shown. The main observations concerning the obtained results are herein discussed.

Table 4 – Summary of the obtained 5-storey design solutions

		EC8 DCM				IFBD			
Seismic location	***CFST type***	***T_1 [s]***	***Steel weight [t]***	***Concrete volume [m^3]***	***q***	***T_1 [s]***	***Steel weight [t]***	***Concrete volume [m^3]***	***q***
Porto	Circular	1.14	10.4	13.0	4.0	1.63	7.7	5.4	1.00
Lagos		1.14	10.4	13.0		1.35	8.3	7.3	2.86
Porto	Square	1.18	11.2	5.3		1.63	9.2	3.8	1.00
Lagos		1.18	11.2	5.3		1.30	10.2	4.9	2.08
Porto	Rectangular	1.18	10.7	3.6		1.63	8.5	2.9	1.00
Lagos		1.18	10.7	3.6		1.30	9.9	3.6	2.89

Firstly, the results obtained in this study show some influence of the type of CFSTs on the flexibility (and steel weight) of the frame. With the use of circular columns, lighter and hence more flexible structures were obtained, in comparison to the square and rectangular alternatives. This is, in part, due to the fact that the cross-section slenderness limits of EC8 for square/rectangular CFSTs are much more stringent than those prescribed for circular members. This prevents the use of the majority of the available commercial steel tubular sections, as they violate the EC8 requirements. Therefore, only less slender (and heavier) members may be used in the seismic design. On the contrary, the majority of standard circular tubes comply with the code requirements, and a wider database of possible members for the seismic design is attained. Another factor that contributes to this influence of the type of CFST is the fact that square members (and also rectangular, but not with such severity) tend to have much lower flexural stiffness than an equivalent circular column. For structures in which stiffness requirements (e.g. limitation of the interstorey drift sensitivity coefficient, limitation of the inter-storey drift) govern the design (as it tends to occur for MRFs), the obtained design solution is very dependent on providing sufficient lateral stiffness to the frame. As such, circular members tend to offer the highest level of lateral stiffness for the same range of member dimensions. Notwithstanding the aforementioned advantages of circular members, it is important to note that such CFST types also lead to larger volumes of concrete.

The second important observation concerns the methodology used in the seismic design for the level of ductility intended for the structure, i.e. the behaviour factor. For the EC8 DCM scenario (*q* of 4), despite having fundamentally different seismic zones, the obtained design solutions were the same. This occurred because the designs were governed by the limitation of θ, which is related to the sufficient lateral stiffness of the frame, and is not correlated with the seismic intensity. On the contrary, with the use of an improved methodology (IFBD), every structure is designed in accordance to a specific seismic loading scenario. The use of IFBD makes clear the limitations of a prescribed fixed *q* value strategy. If one analyses the structures designed for Porto (low seismicity region), the obtained *q* value was equal to unity. This means that the structure designed to sustain gravity loads (which was the initial design solution for every seismic design) was able to withstand the design earthquake whilst remaining elastic. Therefore, there would be no need to explore nonlinear behaviour in such MRFs, as intended when a behaviour factor of 4 is used. Thus, it becomes clear that the consideration of a fixed value of the behaviour factor according to EC8 in the design may greatly defeat its effectiveness, as the members become oversized and the purpose of using *q*, i.e. taking advantage of the inelastic behaviour of the structure, is compromised. Additionally, as one may infer from Table 4, important savings in steel weight were attained with the use of the IFBD procedure. If the IFBD procedure proposed is adopted, the designer is able to define a seismic design solution that is different and adjusted for every seismic scenario, with reductions in steel weight of the frame averaging 20% in the 48 archetypes.

4. Seismic performance assessment

4.1 Introduction

An important objective of this research study was to evaluate the benefits in terms of seismic performance that may be achieved through the use of CFST columns in MRFs, in detriment to steel-only open section solutions. To this end, steel frames (IPE steel beams and HEB steel columns, IPE and HEB being designations of European standard steel open sections with I or H shape, respectively) equivalent to the composite archetypes were designed according to EC8. The cases are equivalent in what concerns the building and frame layout, gravity loads, seismic location, ductility class, design criteria (e.g. P-Delta effects, capacity design, and damage limitation) and design method. It is important to note that the only difference between frames is the type of columns (steel or CFST) that were considered in the design of the structures. The seismic performance of both the composite and steel archetypes was then accessed through application of the Incremental Dynamic Analysis (IDA) procedure proposed by Vamvatsikos and Cornell [16]. In the following paragraphs, only the results obtained for a single composite archetype (seismic location Lagos, circular CFST columns, behaviour factor of 4 according to EC8 DCM) and an equivalent steel-only frame, are detailed.

The obtained design solutions for the considered frames are summarized in Table 5. For the composite moment-resisting frames, available commercial steel tubular sections were considered for the CFST columns. For the circular CFST members, the tubular section designation is shown as *d x t*, *d* being the external diameter of the steel tube cross-section, and *t* its thickness. Additionally, Table 6 compares the corresponding design solution's fundamental period, steel weight and concrete volume associated to the column infill.

Table 5 – Seismic design solutions of the considered steel and composite MRFs

	Case 1 – Steel archetype			***Case 2 – Composite archetype***		
Storey	***Beams***	***Exterior Columns***	***Interior columns***	***Beams***	***Exterior Columns***	***Interior columns***
5	IPE300	HEB180	HEB200	IPE300	323.9x6	404.6x6
4	IPE330	HEB220	HEB280	IPE330	323.9x6	404.6x10
3	IPE330	HEB220	HEB280	IPE330	323.9x8	404.6x10
2	IPE400	HEB240	HEB340	IPE400	323.9x8	404.6x10
1	IPE400	HEB240	HEB340	IPE400	323.9x10	404.6x12

Table 6 – Dynamic properties and steel weight sumamry

Case	***T_1 [s]***	***Steel weight [t]***	***Concrete volume [m^3]***
1	1.29	11.4	-
2	1.14	10.4	13.0

The analysis of the obtained design solutions allows concluding that some reductions in steel weight can be achieved with the use of CFST columns, as an alternative to standard steel sections, such as the ones used in the steel moment-resisting frame, i.e. Case 1. Although this was attained by the introduction of some concrete in the solution, the considerable difference in material cost between concrete and steel results in an almost insignificant contribution of CFST infill to the overall structural cost. However, it is important to note that the overall cost of the structure may increase with the use of CFST columns, given that member joints, foundations and construction time, are aspects that will become more complex and costly. Nonetheless, even though the overall cost of the composite frame is equivalent or higher than that of a steel frame, this may be justifiable if benefits are achieved from a seismic performance perspective.

The seismic performance assessment of the two frames (Cases 1 and 2) and the description of the developed numerical modelling framework will be conducted in the following sections.

4.2 Numerical modelling

The seismic performance assessment of the frames of Cases 1 and 2 was performed in OpenSees [17] by adopting a simplified numerical modelling approach. Both beam and column members were simulated with a concentrated plasticity approach at both element ends, using the numerical parameters calibration procedure proposed by Araújo et al. [18]. This procedure makes use of advanced full 3D numerical models of isolated cantilever elements which are subjected to both monotonic and cyclic bending loading conditions, in order to calibrate the deterioration model parameters of the concentrated plasticity elements in OpenSees. The Modified Ibarra-Medina-Krawinkler deterioration model [19] was used to simulate the nonlinear material behaviour of steel and CFST members. Whilst bilinear hysteretic response was adopted for steel beams and columns, peak-oriented hysteretic response was utilized to simulate the behaviour of CFST columns. The advanced numerical modelling of the steel beams and columns was performed in ANSYS [20], and of the CFST elements in ABAQUS [21]. Fig. 7 and Fig.8 show one example of the aforementioned calibration procedure, namely in terms of comparing the behaviour of both the advanced 3D model (ANSYS and ABAQUS, respectively) and the concentrated plasticity simplified model in OpenSees.

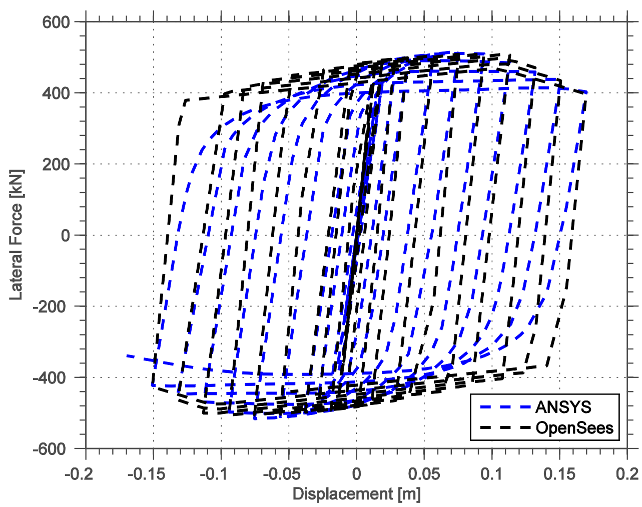


Fig. 7 – Concentrated plasticity calibration procedure for a steel HEB340 member

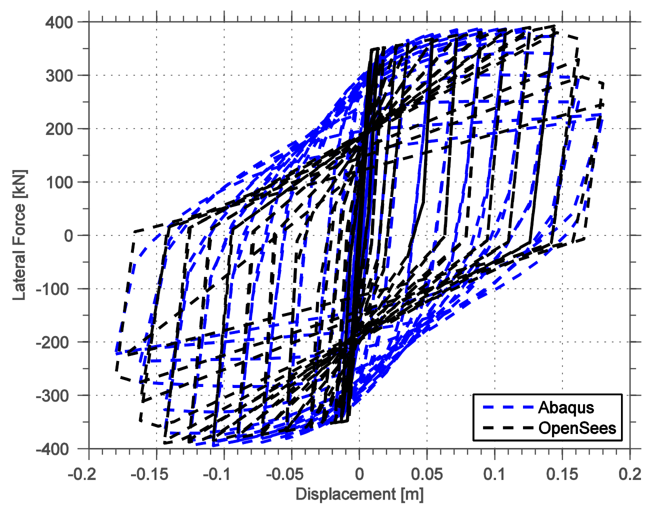


Fig. 8 – Concentrated plasticity calibration procedure for a circular CFST 404.6x12 member

Overall, a good correlation between both models was achieved with the use of a calibration procedure to determine the deterioration model parameters, allowing for a realistic simulation of the response of the moment-resisting frames in OpenSees.

4.3 Seismic performance assessment

The seismic performance assessment of the moment-resisting frames was performed for Case 1, and a single composite frame, namely Case 2, through application of the Incremental Dynamic Analysis (IDA). The 5% damped first mode spectral acceleration was considered as the seismic intensity measure (IM) and the maximum inter-storey drift as the engineering demand parameter (EDP). The SelEQ tool [22] was employed to define a group of thirty ground motions from real earthquake events that were selected and scaled in order to have spectral shape compatibility with the Eurocode 8 spectrum adopted in the design of the two frames, as shown in Fig. 9.

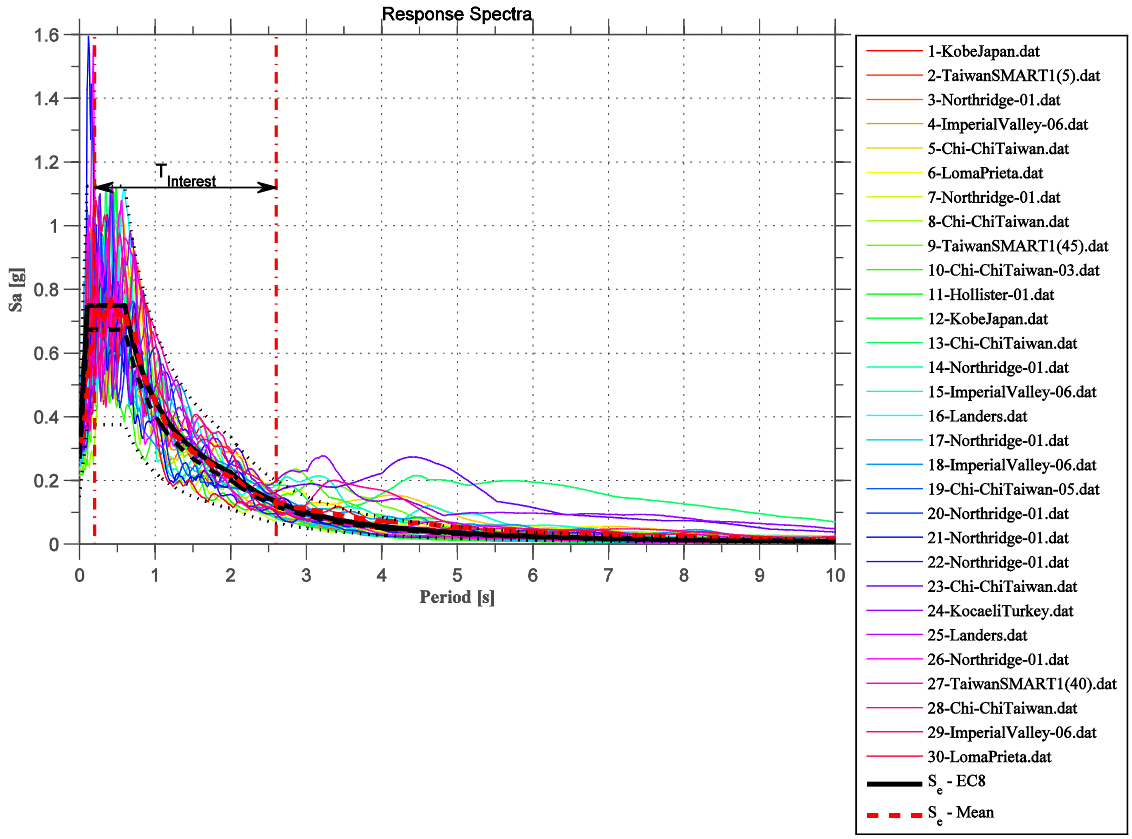


Fig. 9 – Response spectra of the selected ground motions

The Hunt-and-fill algorithm proposed by Vamvatsikos and Cornell [16] was also used, allowing for a reduction in the number of analysis per record required to obtain the IDA curves. Fig. 10 and Fig. 11 show the IDA curves of Cases 1 and 2, respectively, and Fig. 12 and Fig. 13 show the corresponding 16%, 50% and 84% fractile IDA curves. Additionally, Fig. 14 shows a comparison of the collapse fragility curves for both cases, simplistically assuming a collapse limit-state characterized by the flattening of the IDA curves, which was considered to occur if the slope of the IDA curve reduces to 10% of the initial value.

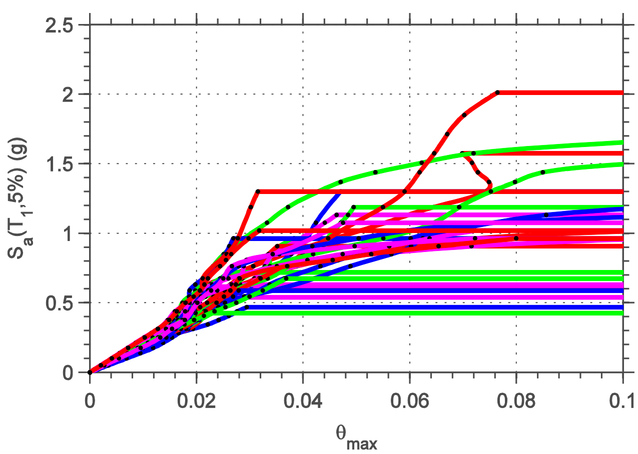


Fig. 10 – IDA curves of Case 1
(steel frame)

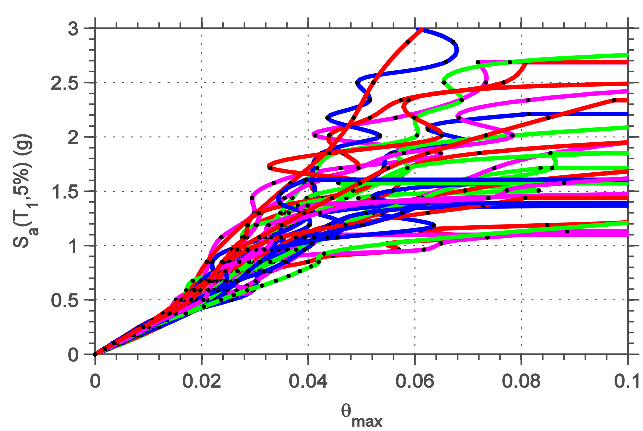


Fig. 11 – IDA curves of Case 2
(composite frame with circular CFSTs)

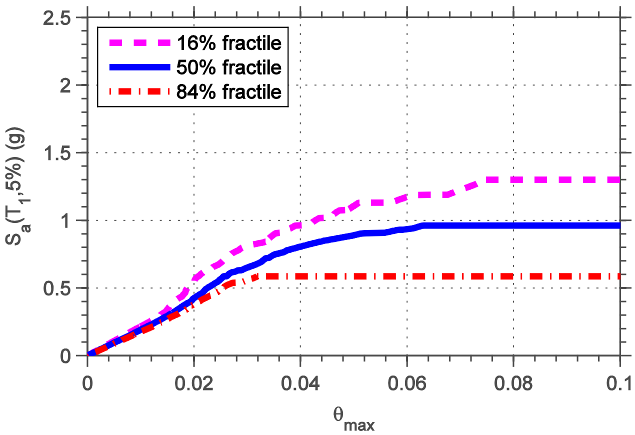


Fig. 12 – Fractile IDA curves of Case 1
(steel frame)

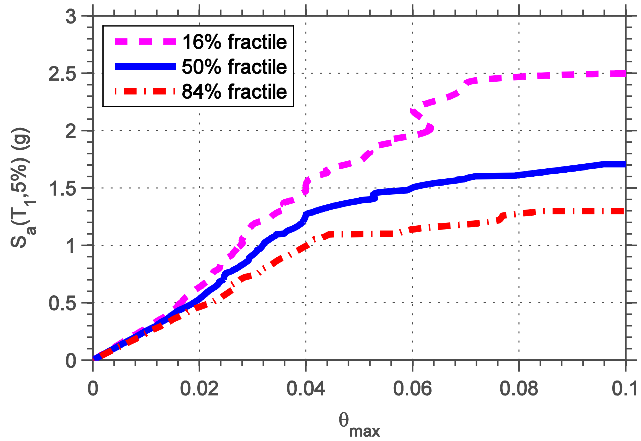


Fig. 13 – Fractile IDA curves of Case 2
(composite frame with circular CFSTs)

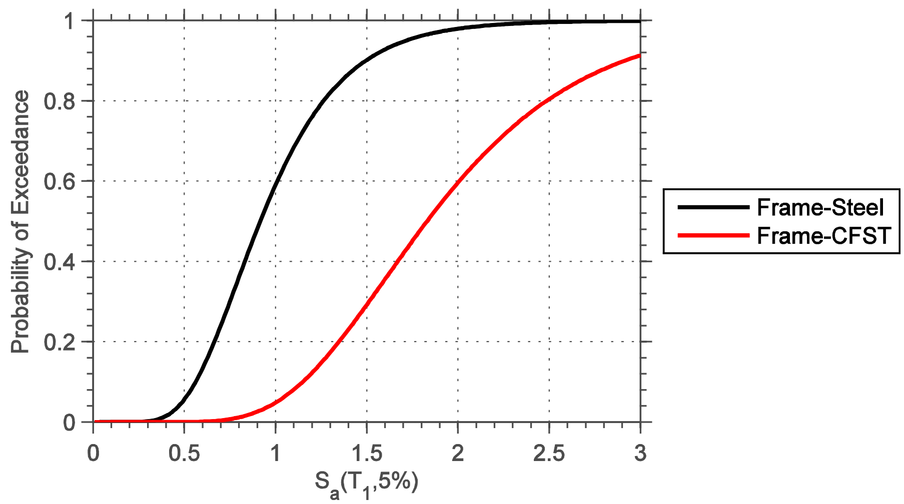


Fig. 14 – Collapse fragility curves of Cases 1 and 2

It is important to recall that both local and global ductility criteria defined in EC8 were considered in the seismic design of the cases. A more detailed look into the design process shows that the strength ratios evaluated at every joint are very similar between cases, pointing therefore to the potential development of similar collapse modes. As such, the collapse mechanism developed in the frames is expected to be largely dominated by stable weak beam-strong column mechanisms. This observation is further supported by non-linear static pushover analysis performed for both Case 1 and Case 2, in which this collapse mechanism was observed.

It is also important to note that the design of moment-resisting multi-storey frames according to EC8 allows the development of plastic hinges at beam ends (weak beam-strong column criterion), at the bottom ends of the columns located at the base level of the frame, and at the top ends of the columns located on the top storey. Considering that the only structural members that differ between cases are the columns, it becomes evident that the formation of a plastic hinge at the base of the structure will undoubtedly explore the behaviour of these elements. As such, the difference between the behaviour of CFST and steel members at the base of the frames can significantly influence the behaviour of the frame, and thus the collapse fragility curves of the cases.

By analysing the obtained fragility curves, one is able to conclude that the composite moment-resisting frame exhibits better seismic performance, in comparison to the steel frame. As an example, for a 1.0g value of $S_a(T_1, 5\%)$, the probability of exceedance of the defined collapse limit state is around 60% for the steel frame, whereas a value of 5% value is observed for the composite frame. This conclusion can be explained by the improved ductility properties of CFST members in comparison to bare steel elements. A comparison of the hysteretic behaviour of the interior base column of Case 1, namely steel section HEB340, and Case 2, namely circular CFST 404.6x12, shows that cyclic degradation takes place at higher levels of rotation for the composite element, as shown in Fig. 15. Therefore, for the same level of seismic demand of a given moment-resisting frame, the improved ductility properties of the CFST columns allows for lower probability of exceedance of the collapse limit state for a given IM value. This is further supported by analysing the IDA curves, where the flattening of the curves in Case 2 starts to occur for higher levels of the EDP considered.

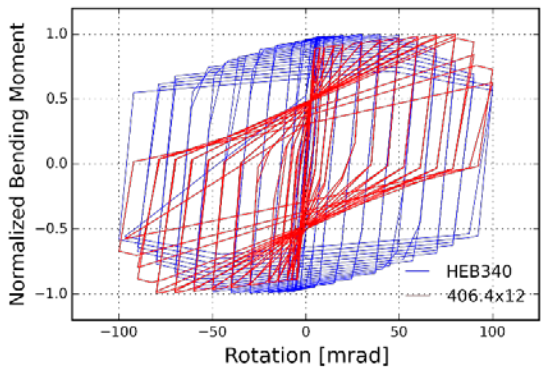


Fig. 15 – Hysteretic behaviour of a bare steel and circular CFST members

Overall, it is possible to conclude that the analysed composite moment-resisting frame shows increased seismic performance in comparison to the steel frame. This was achieved with the use of concrete filled steel tubular columns, which also allowed for a reduction of steel weight of the design solution.

5. Conclusions

In this paper, an evaluation of the seismic performance of moment-resisting frames with sustainable CFST members was carried out. The following conclusions can be withdrawn:

- Eurocode 8 has very restrictive cross-section slenderness requirements for dissipative elements for square and rectangular CFST members, in comparison to circular members;
- The use of circular CFST columns is beneficial to the design of composite moment-resisting frames, as square and rectangular solutions tend to require the use of more quantities of steel;
- The consideration of a fixed value of the behaviour factor according to EC8 in the design may lead to oversized design solutions, particularly for cases governed by stiffness requirements;
- If the Improved Force-Based Design (IFBD) is adopted, the designer is able to reach a solution that is different and adjusted for every seismic scenario, with reductions in steel weight averaging 20% in the 48 archetypes.
- Seismic design of moment-resisting frames using CFSTs instead of steel columns may lead to savings in the steel weight of beams and columns;
- The seismic performance assessment of Cases 1 and 2 (steel frame and composite frame with circular CFSTs, respectively) shows evidence of an increased seismic performance of composite moment-resisting frames using CFST columns, in comparison to equivalent steel only structural solutions.

6. Acknowledgements

All the authors would like to acknowledge FCT for the financial support through the research project “Recycling & Seismic Protection: Sustainable High-Performance CFST Columns for Seismic Areas” (PTDC/ECM/117774/2010). Also acknowledged is the support of FERPINTA, by providing all the steel tubes for the experimental campaign, and PRESDOURO, for providing the resources for the casting of the concrete of the test campaign specimens.

7. References

[1] Schneider S (1998): Axially loaded concrete-filled steel tubes. *Journal of Structural Engineering*, **124** (10), 1125-1138.

[2] Han L.H (2002): Tests on stub columns of concrete-filled RHS sections. *Journal of Constructional Steel Research*, **58** (3), 353-372.

[3] Sakino K, Nakahara H, Morino S, Nishiyama I (2004): Behavior of centrally loaded concrete-filled steel-tube short columns. *Journal of Structural Engineering*, **130** (2), 180-188.

[4] Ellobody E, Young B, Lam D (2006): Behaviour of normal and high strength concrete-filled compact steel tube circular stub columns. *Journal of Constructional Steel Research*, **62** (7), 706-715.

[5] Elchalakani M, Zhao XL, Grzebieta R (2001): Concrete-filled circular steel tubes subjected to pure bending. *Journal of Constructional Steel Research*, **57** (11), 1141-1168.

[6] Varma A, Ricles J, Sause R, Lu LW (2002): Experimental behavior of high strength square Concrete-Filled Steel Tube beam-columns. *Journal of Structural Engineering*, **128** (3), 309-318.

[7] Varma A, Ricles J, Sause R, Lu LW (2002): Seismic behavior and modeling of high-strength composite concrete-filled steel tube (CFT) beam-columns. *Journal of Constructional Steel Research*, **58** (5-8), 725-758.

[8] Han LH, Lu H, Yao GH, Liao FY (2006): Further study on the flexural behaviour of concrete-filled steel tubes. *Journal of Constructional Steel Research*, **62** (6), 554-565.

[9] Duarte APC, Silva BA, Silvestre N, de Brito J, Júlio E, Castro JM (2016): Tests and design of short steel tubes filled with rubberized concrete. *Engineering Structures*, **112**, 274-286.

[10] Duarte APC, Silva BA, Silvestre N, de Brito J, Júlio E, Castro JM (2016): Experimental study on short rubberized concrete-filled steel tubes under cyclic loading. *Composite Structures*, **136**, 394-404.

[11] Silva A, Jiang Y, Castro JM, Silvestre N, Monteiro R (2016): Experimental assessment of the flexural behaviour of circular rubberized concrete-filled steel tubes. *Journal of Constructional Steel Research*, **122**, 557-570.

[12] CEN (2004): EN 1994-1-1 Eurocode 4: Design of composite steel and concrete structures. Part 1-1, General rules and rules for buildings. *European Commitee for Standardization*, Brussels, Belgium.

[13] CEN (2004): EN 1998-1 Eurocode 8: Design of structures for earthquake resistance. Part 1, General rules, seismic actions and rules for buildings. *European Commitee for Standardization*, Brussels, Belgium.

[14] FEMA (2009): FEMA P-695: Quantification of Building Seismic Performance Factors, Federal Emergency Management Agency, Washington, DC, USA.

[15] Villani A, Castro JM, Elghazouli AY (2009): Improved seismic design procedure for steel moment frames. *STESSA 2009: Behaviour of Steel Structures in Seismic Areas*, Philadelphia, USA.

[16] Vamvatsikos D, Cornell CA (2002): Applied incremental dynamic analysis. *Earthquake Spectra*, **20** (2), 523-553.

[17] PEER (2006): OpenSees: Open system for earthquake engineering simulation. *Pacific Earthquake Engineering Research Center, University of California*, Berkeley, CA, USA.

[18] Araújo M, Macedo L, Castro JM (2015): Calibration of strength and stiffness deterioration hysteretic models using optimization algorithms. *Proceedings of the 8th International Conference on Behaviour of Steel Structures in Seismic Areas*, Shanghai, China.

[19] Lignos DG, Krawinkler H (2011): Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading. *Journal of Structural Engineering*, **137** (11), 1291-1302.

[20] ANSYS (2009): ANSYS Structural Analysis Guide. *ANSYS, Inc.*, Canonsburg, PA, USA.

[21] ABAQUS (2011): ABAQUS Documentation. *Dassault Systèmes Simulia Corp.*, Providence, RI, USA.

[22] Araújo M, Macedo L, Marques M, Castro JM (2016): Code-based record selection methods for seismic performance assessment of buildings. *Earthquake Engineering and Structural Dynamics*, **45** (1), 129-148.