Seismic performance of dual steel Concentrically Braced frames

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SUMMARY:
The seismic design according to current seismic codes is based on the principles of hierarchy criteria. Hence, the non-dissipative members should be designed to resist the full plastic strength of the dissipative zones. The possibility to combine the High Strength Steel (HSS) with Mild Carbon Steel (MCS) appears to be an effective and rational method to achieve this overall behaviour, where HSS is used for non-dissipative members while MCS for dissipative elements. This design approach leads to the so-called "dual steel" structures. The aim of this study is to analyse the seismic behaviour of dual steel structures having dual and simple concentrically bracing systems. To this end, nonlinear static and dynamic analyses were carried out to determine behaviour factors, overstrength factors and performance parameters such as ductility demand and storey acceleration pattern.

Keywords: High strength steel, dual-system, concentrically braced frames, nonlinear analysis

1. INTRODUCTION

According to modern codes the seismic design of steel or composite buildings are based on the concept of dissipative structures, where specific zones of the structures should be able to develop plastic deformation, mainly on ductile member, in order to dissipate the seismic energy. On the contrary, the non-dissipative zones should behave elastically under seismic action. For this reason, these zones should be designed to resist the full plastic strength of the dissipative members. As a consequence, the large overstrength demands to non-dissipative zones leads high consumption of material and, sometimes huge size of members to fulfil the design requirements. Therefore, the combined use of HSS grades for non-dissipative parts and mild steel (MS) for those dissipative get easier the application of capacity design criteria, resulting in smaller sizes than those obtained using MS only. Structures designed combining both HSS and MS is termed dual-steel structure (Dubina, 2010).

Recent studies (Dubina *et al.*, 2006; Dubina *et al.*, 2008; Dubina, 2010) have highlighted the advantages of Dual steel concept, especially for what concerns the control of seismic response of multistorey buildings to achieve overall ductile mechanism.

These considerations motivated the study presented in this paper, which is aimed to investigate the seismic behaviour of inverted-V dual concentrically braced frames (D-CBF) and concentrically braced frames (CBF) designed according to dual-steel concept and compliant to EN 1998-1 (2005) and AISC 341 (2005) requirements.

2. ANALYZED FRAMES

2.1. Investigated parameters

In order to evaluate the seismic behaviour of dual steel D-CBFs and CBFs, a set of 8-storey frames were designed on this study changing in each typology the following parameters: Column typology (Fully encased (FE), partially encased (PE) and concrete filled tube (CFT) composite columns) having S460 steel grade have been investigated and type of design soil condition, stiff and soft soil, were investigated.

From the combination of these parameters a total number of twelve frames to be examined are obtained. In order to organize the work properly, a label code was set up as follows:

(Typology).(Type of soil).(Column type)

where, the number 1 corresponds a stiff soil and 2 a soft soil. Regarding to column type, the number 1, 2 and 3 are associated to FE, PE and CFT, respectively.

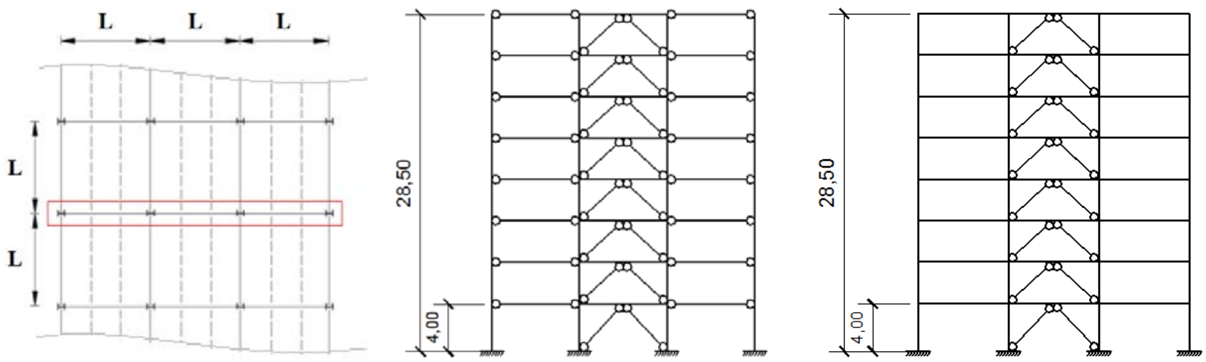
2.2. Structural system

The studied frames have a total frame height of 28.5 m with interstorey height equal to 3.5 m except for the first interstorey height that is 4.0 m.

The analysed frames were extracted from a reference building having virtually in plan an indefinite width, where the generic braced frame are placed alternately. Namely, the spacing of braced frames is equal to 2*L*, being *L* the span length in the transverse direction, as shown in Fig. 1.

The floor mesh is regular and orthogonal with columns spacing at L = 7.5 m. Fig. 1 shows the structural plan for both structural system and their respective geometric configurations.

Floors are made of composite steel decks simply rested on steel beams (primary and secondary), which are restrained to avoid flexural-torsional buckling. Considering the low out-of-plane stiffness of gusset plate connections, braces are pinned in both ends. Instead, owing to the high in-plane stiffness and the presence of flange stiffeners the beam-to-column joints of braced part for both typologies were assumed as rigid. Columns are fixed at the base and continuous through the height.



a) Plan b) CBF c) D-CBF

Figure 1. Mathematical models for non-linear response history analysis

2.3. Assumptions and design

The frames were basically designed according to EN 1998-1 (2005). Nevertheless, owing to the lack of specific requirements for D-CBFs in EN1998-1, it was introduced an additional design control to guarantee a minimum lateral strength to the moment resisting frames of D-CBF. In particular, according to AISC 341 (2005) it was designed the MRF part to provide the 25% of the total lateral strength of the frame.

The design peak ground acceleration was set equal to 0,32g. Both stiff and soft soil conditions have been assumed. The following gravity loads were considered in the structural design: G_k = 4.0kN/m^2

and Q_k = 3.0kN/m^2. It was assumed a behaviour factor equal to 2.5 and 4.8 for the concentrically and dual-concentrically braced frames, respectively. The dissipative elements (braces) were designed using S355 steel grade, while non-dissipative are made of S460, except to columns from 5th to 8th level where the MCS was applied in order to optimize the design efficiency (namely the ratio between the design forces and the design strength).

3. BEHAVIOUR FACTOR

In the present study two approaches were analyzed to quantify the behaviour factor: European and Aribert & Grecea (1997) approaches.

In European approach, the behaviour factor is obtained applying the Eqn. 3.1, being A_u the peak ground acceleration leading to accepted failure for the selected performance level, A_y is the peak ground acceleration corresponding to the yielding of the frame and finally, α is the overstrength factor from pushover analysis corresponding to ratio between maximum shear base and shear base when the first plastic hinge occurs. The values of A_y and A_u were obtained performing a set of nonlinear incremental dynamic analysis (IDA).

$$q = \alpha \times \frac{A_u}{A_y} \tag{3.1}$$

Aribert & Grecea (1997) have introduced a new formulation to define the q-factor based on the reduction of the shear base of building. Similarly to European approach, this method consists in performing a series of nonlinear dynamic analyses where each accelerogram is scaled by a multiplier factor "λ" step-by-step up to the specific criterion collapse termed as λ_u. Hence, the q-factor is given by ratio between the elastic theoretical base shear force, $V^{(e,th)}$, and the real inelastic base shear force, $V^{(inel)}$, as given by Eqn. 3.2.

$$\mathrm{q} = \frac{V^{(e,th)}}{V^{(inel)}} = \frac{V^{(e)} / \lambda_e}{V^{(inel)} / \lambda_u} = \frac{V^{(e)}}{V^{(inel)}} \times \frac{\lambda_u}{\lambda_e} \tag{3.2}$$

3.1. Performance criteria

In order to determine the q-factor for both approaches previously described, it is necessary to define damage levels to assess the seismic response of examined frames.

As given by EN 1998-3 (2005), the criteria associated to significant damage (SD) limit state were used. Therefore, the acceleration amplitude A_u and scalling factor λ_u were determined evaluating the following criteria:

$$A_u = \lambda_u = \min\,(A_M,\ A_{br}, A_{col})$$

where A_M corresponds to the attainment of the plastic bending moment in the middle of the braced beam; A_{br} corresponds to the maximum permitted brace deformation both in tension and in compression; A_{col} corresponds to the buckling of column.

4. NONLINEAR ANALYSIS

The nonlinear analyses were carried out using the computational platform of SeismoStruct – *Version 5.0.5*. Numerical models are based on fiber approach, in which the cross sections of structural elements are discretized into small regions, named fiber.

The concrete constituting the composite column was modelled in order to take into account the confinement effect due to the steel section and the reinforcement rebars, applying the model suggested by Martinez-Rueda and Elnashai (1997). The hysteretic model proposed by Menegotto & Pinto (1973) was considered in steel members to take into account the elastic, yielding and hardening branches and Bauschinger effect.

A physical-theory model was implemented to simulate the brace hysteretic behaviour, using two linear elastic beam-column elements connected together by a generalized plastic hinge. More details on this model can be found in Landolfo *et al*. 2010 and D'Aniello *et al*. 2012.

In order to take into account P-delta effects, a learning column was employed in these analyses where the seismic masses that are not tributary on frames were vertically applied at each floor. The Rayleigh damping with tangent stiffness-proportional damping matrix was used for nonlinear dynamic analysis, assuming 2% as damping ratios for first and second modes.

4.1. Pushover

According to EN1998-1 (4.3.3.4.2.4) two lateral load distributions are used for pushover analyses: i) 1st mode force distribution; ii) Uniform pattern (proportional to masses).

The performance parameters investigated by Pushover analysis are shown in Fig. 2, where the parameter V_y is the yield strength of the structure, V_{1y} is the base corresponding to first plastic hinge and V_d corresponds to the design base shear; δ_1, δ_y and δ_u are displacements at formation of the first plastic hinge, yield and ultimate displacement, respectively.

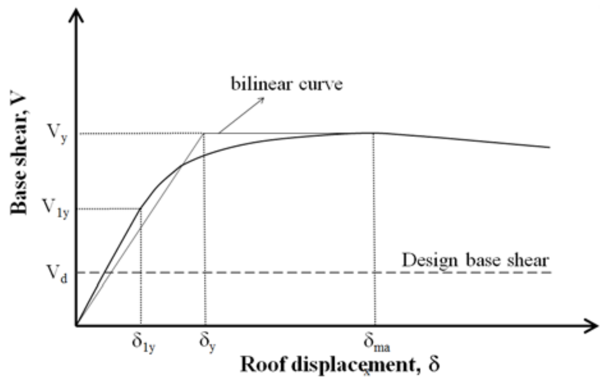


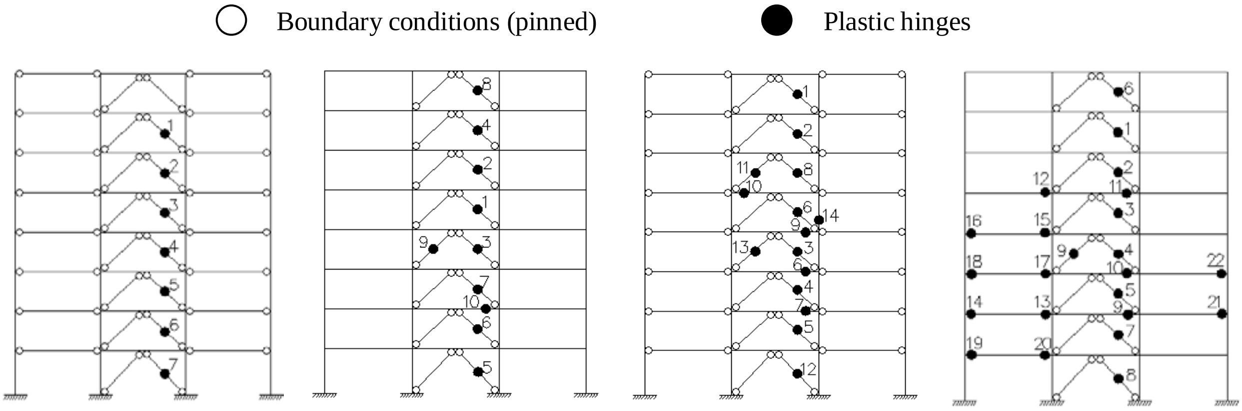
Figure 2. Capacity curves from pushover analysis

Fig. 3 illustrates the plastic hinge pattern at the target displacement obtained according to N2 method implemented by EN 1998-1 (2005) of four frames only because this damage distribution was similar to another columns type. As expected the first nonlinear events were the brace buckling which propagated from upper to lower levels of the examined frames, thus highlighting an overall cantilever behaviour of the frames. Subsequently, the braces in tension yielded, in general at the mid- height of the frames.

It was observed that frames designed under stiff soil conditions experienced a uniform damage pattern along the building height. Moreover, it was recognized that both D-CBFs and CBFs showed very similar damage distribution.

Appreciably differences in the plastic hinge pattern were observed for the frames designed under soft soil condition. In particular, the dual action was evident after brace buckling allowing a more uniform distribution of plastic engagement along building height. This result may be due to the higher lateral stiffness of the MRF in the frames design for soft soil than those for stiff soil. Indeed, the former are designed for higher lateral forces, resulting in a larger strength and stiffness than those obtained in the second cases.

Although beams were designed to resist the vertical forces due to contemporary brace yielding in tension and post-buckling in compression, it is worth noting that the most of beams belonging to the braced bays experienced the formation of plastic hinges.



a) CBF_1.1 b) D-CBF_1.1 c) CBF_2.1 d) D-CBF_2.1

Figure 3. Capacity curves from pushover analysis

4.1.1 Overstrength

The overstrength is defined by ratio between overall yield strength of the structure and the design based shear. In order to investigate on the influence this parameter, the structure overstrength ratio could be decomposed on follows:

$$\Omega = \frac{V_y}{V_d} = \frac{V_y}{V_{1y}} \times \frac{V_{1y}}{V_d} \tag{4.1}$$

The first term (V_y / V_{1y}) is the overstrength factor defined in EN 1998-1 (2005), which depends on the frame configuration, formation of the collapse mechanism, internal force redistribution and gravity loading. The second term (V_{1y} / V_d) is associated to criteria assumed in seismic design process, such as, member oversizing, serviceability requirements and/or gravity load.

Table 1 reports the overstrength factors obtained for all frames, showing the differences between the examined D-CBFs and CBFs. The former presented higher overstrength factors. This result is ascribable to the presence of MRF part, which contributes to lateral capacity. The CBFs exhibited lower overstrength. In general, the minimum overstrength introduced by EN 1998-1 (2005) corresponded to uniform pattern pushover analyses, while for overstrength associated to design process the minimum matched to 1st Mode pattern.

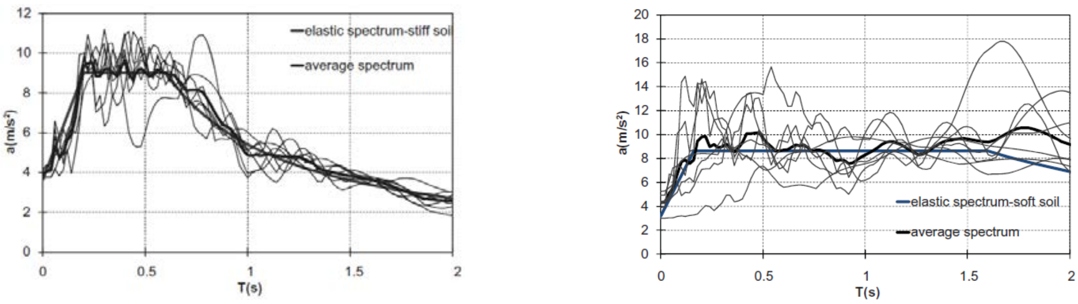
4.2. Dynamic analysis

4.2.1 Accelerograms

In order to perform the nonlinear dynamic analyses, two set of seven natural and generated records were selected to be compatible to the stiff and soft soil elastic spectra. The artificial records were obtained using SIMQKE software, while the natural records were selected from the European Strong Motion Data – ESMD. The 5% damped response spectra of the records used to perform the dynamic analyses for both stiff and soft soil conditions are showed in Fig. 4.

Table 1. Overstrength factors

Frames	V_y / V_{1y}		V_{1y} / V_d	
	Min.	Pattern	Min.	Pattern
CBF_1.1	1.41	1st Mode	1.11	1st Mode
CBF_1.2	1.44	Uniform	1.10	1st Mode
CBF_1.3	1.34	1st Mode	1.12	1st Mode
CBF_2.1	1.21	Uniform	1.06	1st Mode
CBF_2.2	1.22	Uniform	1.07	1st Mode
CBF_2.3	1.19	Uniform	1.06	1st Mode
D-CBF_1.1	1.64	Uniform	1.66	1st Mode
D-CBF_1.2	1.68	Uniform	1.70	1st Mode
D-CBF_1.3	1.62	Uniform	1.65	1st Mode
D-CBF_2.1	1.25	Uniform	1.40	1st Mode
D-CBF_2.2	1.33	Uniform	1.39	1st Mode
D-CBF_2.3	1.45	1st Mode	1.33	1st Mode



a) Stiff soil b) Soft soil

Figure 4. 5% - damped response spectra

The average incremental dynamic and pushover response curves plotted in terms of ratio base shear and design based shear (V / V_d) versus maximum drift at the top of the structure are compared in Fig. 5. The plots show that the average incremental dynamic curves were closer to the 1st Mode pattern than uniform pattern response curves, mainly for the frames designed for stiff soil condition. However, at large scaled acceleration the difference between IDA and both pushover response curves is less noticeable.

4.2.2 Interstorey drift ratios

Fig. 6 shows the distribution of interstorey drift ratios obtained from pushover at target displacement and those from nonlinear dynamic analyses corresponding to maximum value found in each floor for design P.G.A.

As it can be observed the pushover analysis tends to overestimate the interstorey drifts. In particular, it was observed that the 1st Mode pattern from pushover analyses provide results closer to those given by dynamic analyses than uniform pushovers.

Concerning to type of columns, it is possible to observe that the frames with CFT columns showed larger drifts in comparison another types where PE columns were responsible for the less values.

In general, the frames located on soft soil showed higher interstorey drift for both typologies in view of the two types of nonlinear analyses. Dynamic analyses showed that the D-CBFs experienced the highest drift demand. Moreover, there was no substantially difference regarding to pushover analyses between CBFs and D-CBFs.

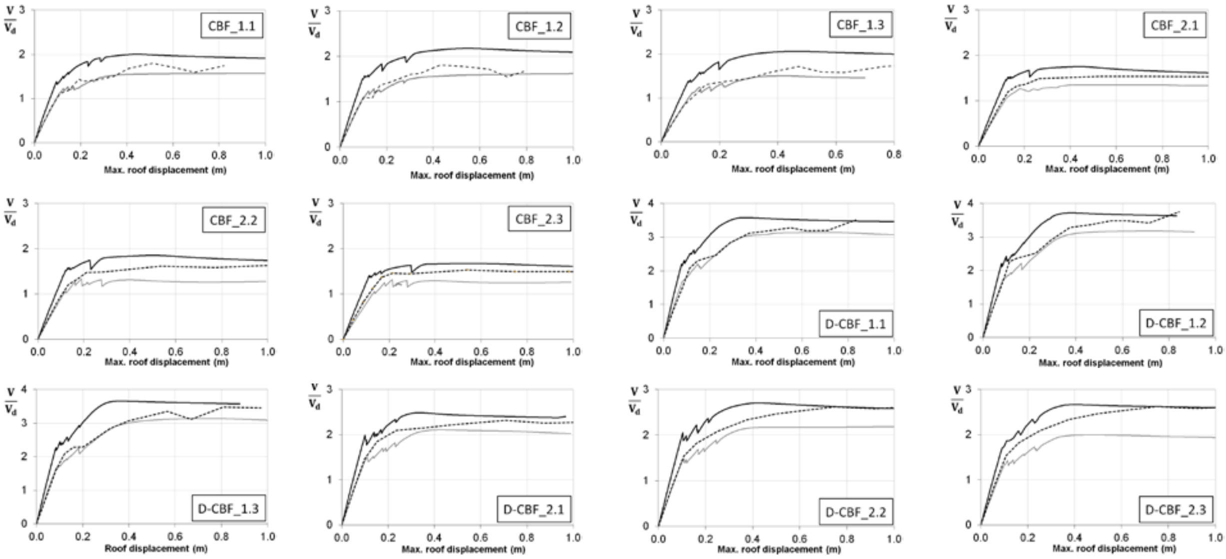


Figure 5. Capacity curves from static and dynamic analyses

(pushover_uniform 1^{st} Mode   Average IDAs)

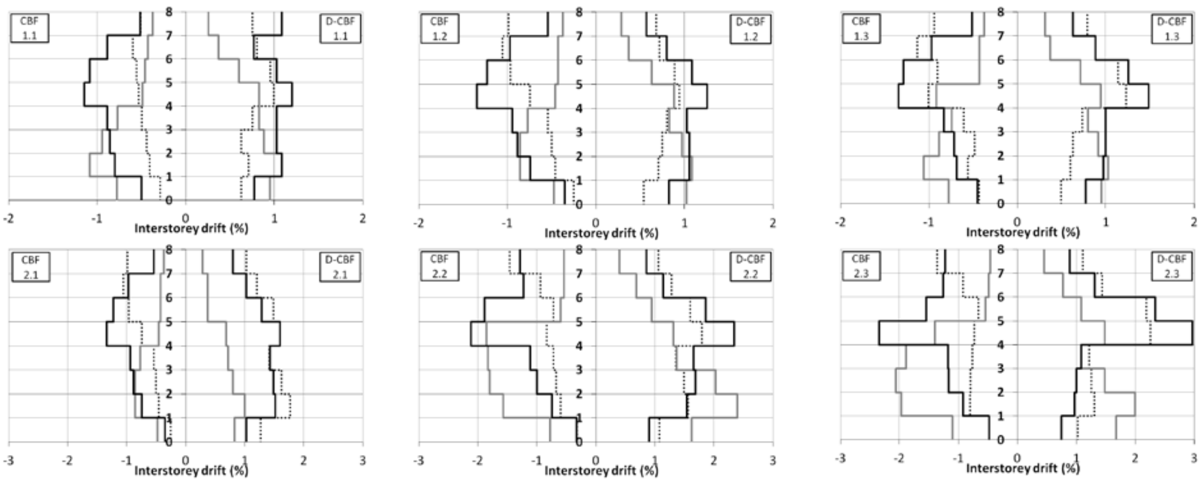


Figure 6. Interstorey drift demand

1^{st} Mode   Uniform   Average dynamic analyses)

4.2.3 Brace ductility

Figure 7 depicts the distribution along the building height of the average brace ductility demand (both in tension and in compression). As it can be observed, the compression ductility demands for D-CBFs were higher than CBF. On contrary, there were no substantial differences for tension braces where the deformation demand imposed was not adequate to yielding them. This result is mainly due to the flexural strength and stiffness of the braced beams which is smaller in D-CBFs than in the relevant CBFs. Indeed, the flexural deformation of the beam at mid-span is an additional source of compressive deformation for braces.

In comparison to the value recommended by EN 1998-3 (2005), the ductility of compression braces was larger to value recommended based in requirements for limit state of significant damage (SD).

In general terms, the ductility demand of braces in compression was not uniform along the height. Indeed, there was a large concentration of damage on the upper floor where the braces have slenderness.

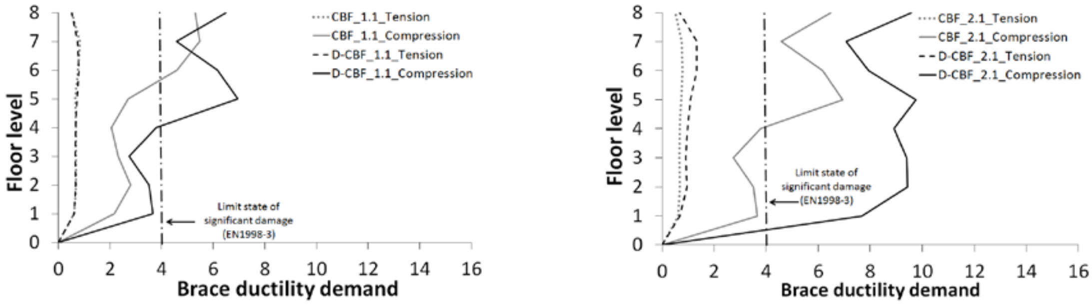


Figure 7. Interstorey drift demand

4.2.4 Storey acceleration

Since storey acceleration are related to the non-structural damage, it was monitored the maximum acceleration pattern along the building height.

Figure 8 show the profile of maximum storey acceleration corresponding to the considered sets of accelerograms scaled at the design P.G.A.. As it can be seen, up to building mid-height the differences between CBFs and D-CBFs are negligible, although the latter showed slightly lesser accelerations. A larger scatter can be recognized at the higher storeys. This may be due to the presence of higher modes in nonlinear conditions.

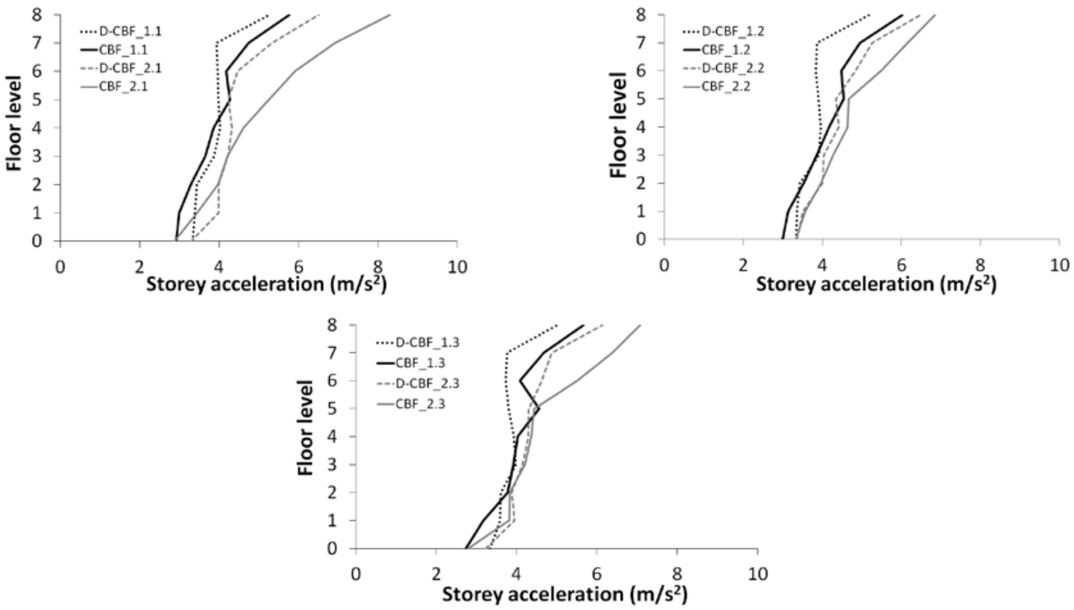


Figure 8. Storey acceleration pattern.

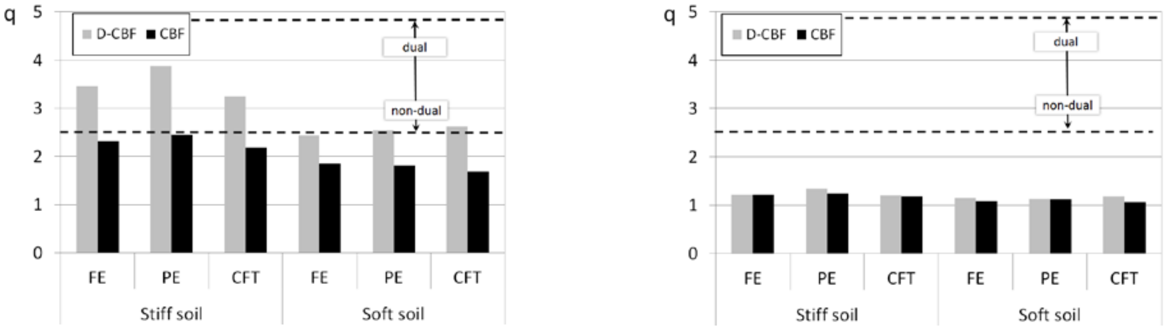
4.2.5 Behaviour factors

The behaviour factors obtained for the examined frames according to both European and Aribert & Grecea (1997) approaches are summarized in Figure 9. It can be easily recognized that the European approach leads to larger behaviour factors than those given by the Aribert & Grecea (1997) approach.

As expected, due to their ductile behaviour, for dual-concentrically braced frames the larger values of q-factors were obtained.

Generally speaking, the q-factors obtained using the European approach presented values smaller than those recommended by EN 1998-1 (2005) for both typologies.

It is also worth noting that the Aribert & Grecea (1997) approach led to similar q-factors for both D-CBFs and CBFs.



a) European approach b) Aribert & Grecea approach

Figure 9. Behaviour factors for D-CBFs and CBFs

As it was seen before, the decisions taken in seismic design may introduce overstrength to structures being it measured by ratio V_{1y}/V_d. Therefore, in order to taken into account the design oversize the behaviour factor regarding to European approach was multiplied by minimum overstrength associated to design process.

Table 2 shows the behaviour factors found for both actual and adapted European approach. As expected, the q-factors from D-CBFs suffered considerable increase. However, the difference between two approaches was no substantially for CBFs.

Table 2. Overstrength factors

Frames	European approach			European approach adapted			
	V_y/V_{1y}	q_o	q_{EC8}	V_y/V_{1y}	V_{1y}/V_d	q_o	q_{ad}
CBF_1.1	1.41	1.65	2.32	1.41	1.11	1.65	2.58
CBF_1.2	1.44	1.69	2.43	1.44	1.10	1.69	2.67
CBF_1.3	1.34	1.63	2.18	1.34	1.12	1.63	2.45
CBF_2.1	1.21	1.53	1.85	1.21	1.06	1.53	1.96
CBF_2.2	1.22	1.48	1.80	1.22	1.07	1.48	1.93
CBF_2.3	1.19	1.42	1.68	1.19	1.06	1.42	1.79
D-CBF_1.1	1.64	2.11	3.46	1.64	1.66	2.11	5.74
D-CBF_1.2	1.68	2.31	3.88	1.68	1.70	2.31	6.59
D-CBF_1.3	1.62	2.00	3.24	1.62	1.65	2.00	5.35
D-CBF_2.1	1.25	1.95	2.44	1.25	1.40	1.95	3.41
D-CBF_2.2	1.33	1.91	2.54	1.33	1.39	1.91	3.53
D-CBF_2.3	1.45	1.81	2.62	1.45	1.33	1.81	3.49

5. CONCLUSIONS

In this paper the seismic performance of EC8 compliant dual and non-dual concentrically braced frames designed according to the dual-steel concept was investigated. To this aim a parametric study was carried out investigating on the influence of type of columns and two types of soil condition.

In particular, the frames designed for stiff soil conditions experienced a uniform damage pattern along the building height. Moreover, among them both D-CBFs and CBFs showed very similar damage distribution. The D-CBFs and CBFs designed for soft soil conditions exhibited a significantly different damage distribution. This result is ascribable to the higher lateral stiffness of the MRF in the frames design for soft soil than those for stiff soil. Indeed, the former are designed for higher lateral forces, resulting in a larger strength and stiffness than those obtained in the second cases.

Although beams were designed to resist the vertical forces due to contemporary brace yielding in tension and post-buckling in compression, it is worth noting that the most of beams belonging to the braced bays experienced the formation of plastic hinges.

The D-CBFs revealed larger drift demand especially for dynamic analyses. On the other hand, there was no significant difference regarding to pushover analyses between CBFs and D-CBFs.

It is important to note that in all examined cases the combined use of MCS and HSS allowed to effectively control the collapse mechanism, avoiding failure of columns.

The ductility demands D-CBFs were higher than CBF. Moreover, the ductility demand of compression braces was larger than value recommended based in EN 1998-3 (2005) requirements.

The behaviour factors obtained by the European approach were substantially higher than those obtained by the Aribert & Grecea (1997) method and smaller than the recommended by EN 1998-1 (2005). However, D-CBFs showed higher q-factors due to their ductility behaviour. On the contrary, the q factors by the Aribert & Grecea (1997) method were similar for both CBFs and D-CBFs.

AKCNOWLEDGEMENT
The authors thank to Erasmus Mundus External Cooperation Window - ISAC and HSS-SERF (High Strength Steel in Seismic Resistant Buildings Frames – Grant N0 RFSR-CT-2009-00024) for financing this study.

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