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Seismic performance of dual-steel moment resisting frames

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ABSTRACT

In seismic design of steel structures, the "dual-steel" concept concerns the combined use of High Strength Steel (HSS) in non-dissipative members and Mild Carbon Steel (MCS) in dissipative zones, in order to control the global frame behavior into a ductile overall failure mode. In this paper, a comprehensive parametric study devoted to investigate the seismic design and performance of Eurocode 8 compliant dual-steel Moment-Resisting Frames (MRF) is presented and discussed. The overall seismic performance has been analysed through static and dynamic nonlinear analyses against three limit states: damage limitation (DL), severe damage (SL) and near collapse (NC). The investigated parameters cover both geometric and mechanical variables, as the type columns, span length, number of storeys and spectral shape. The analyses showed that the use of HSS in Eurocode 8 compliant MRFs is effective to provide overall ductile mechanism, but it may lead to inefficient and uneconomical structures characterized by limited plastic demand due to the large design overstrength. The comparison between dual steel MRFs with those entirely made of MCS showed that: i) in order to fulfil the codified drift requirements and to limit the stability coefficients, the same shapes for members should be used for both structures in the most of cases; ii) a similar performance can be recognized in both dual steel and single grade steel structures under both damage limitation and significant damage limit state; and iii) dual steel frames guarantee a better control of plastic mechanism than single grade steel frames at near collapse limit state.

1. Introduction

In recent years, significant development in steel processing occurred. Indeed, the improvements in industrial processes by the combination of rolling practices and cooling rates allowed obtaining high strength steel (HSS) with very attractive properties. Owing to the high performance, the use of HSS has a number of benefits in terms of economic, architectural, environmental and safety aspects where the increase of strength allows a size reduction of the structural members enabling potential benefits also in terms of environment impact by saving energy and reduction of gas emissions [1]. Nowadays, in Japan and USA, HSSs are widely used for bridge and building construction in seismic zones. On the contrary, in Europe, there is still a limited number of applications on buildings, and even less in seismic regions, although some examples exist of use of HSS in bridge structures [2].

Seismic applications potentially represent the rational field to exploit the high performance of HSS. Indeed, 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 and members should behave elastically under seismic action in order to avoid the brittle collapse of the building. For this reason, these zones should be designed to resist the full plastic strength of the dissipative members. Consequently, the large overstrength demands to non-dissipative zones lead to high material consumption, and sometimes, huge size of members to fulfil this design requirement.

The combined use of HSS for non-dissipative members and of mild carbon steel (MCS) for dissipative members may allow an easier application of capacity design criteria. The expected design improvement would be obtained in terms of smaller member sizes than those obtained when using MCS only. Structures designed using the combination of HSS and MCS are termed "dual-steel" structures.

Recent studies [3–5] have highlighted the advantages of dual-steel concept, especially for what concerns the control of seismic response of multi-storey buildings to achieve overall ductile mechanism. In particular, Dubina et al. [5] showed the potential benefits given by using HSS in full strength moment-resisting steel beam-to-column joints, in order to guarantee the formation of plastic hinge in the beam and preserving both the connection and the column in moment resisting frames (MRFs).

On the other hand, notwithstanding the fulfilment of codified rules such those given by EN1998-1 [8], the design of MRFs with HSS columns may lead to obtain structures with lateral stiffness smaller than those designed with a single steel grade. Hence, some problems may arise

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Nomenclature

a_{gR}	reference peak ground acceleration
k	coefficient depending on seismicity (generally assumed equal to 3)
q	behavior factor
s	beam flexural overstrength
A	acceleration from IDA
A_d	design acceleration
A_y	acceleration corresponding to the yielding of the frame
A_θ	acceleration corresponding to the maximum permitted interstorey drift ratio
A_c	acceleration corresponding to the column plastification
A_R	acceleration corresponding to the maximum permitted local rotation
A_u	acceleration related to the failure
H	height of building
L	bay length
M_u	peak bending moment experienced by the beam
M_p	plastic bending moment on the beam
M_{max}	maximum bending moment on the beam
$M_{j,bi,Ed}$	bending moment in the beam at the intersection of the member centerlines into the joints on the side "i".
T_L	return period
T_{LR}	reference return period for which the reference seismic action may be computed
V_y	base shear from of the yield strength of the structure
V_{1y}	base shear at the formation of the first plastic hinge
V_d	design base shear
α_u	multiplier of horizontal seismic design action at formation of global plastic hinge in the system
α_1	multiplier of horizontal seismic design action at formation of first plastic hinge in the system
β	transformation factor according to EN1993:1–8
δ_{1y}	displacements corresponding to the formation of the first plastic hinge
δ_{max}	roof displacement corresponding to the achievement of an interstorey drift ratio equal to 3%
γ_I	importance factor
γ_{ov}	material overstrength factor
$\gamma_{material}$	density of material total mount
μ	ductility factor
ν	reduction effect accounting the lower return period of the seismic action
θ_y	beam yield rotation
Ω	overstrength factor

such as the nonfulfillment of damage limitation requirements and overall stability problems due to P-Delta effects. These considerations clearly show the need to investigate the effectiveness of dual-steel concept in capacity design of MRFs. To this aim, in this paper a wide parametric numerical study has been carried out to evaluate the benefits of dual-steel concept on the performance based seismic design [6,7] of MRF compliant to EN1998-1 [8]. Furthermore, the overall seismic performance has been analysed through static and dynamic nonlinear analyses against three limit states as defined in EN1998-3 [9]: damage limitation (DL), severe damage (SL) and near collapse (NC). After describing the results of numerical analyses, the discussion looks for insights at the following issues: (i) quantification of performance parameters for each limit states, (ii) characterization of the behavior factors at each limit states, (iii) comparison between dual steel MRFs and single grade steel MRFs, and (iv) assessment of economic efficiency.

2. Framework of the study

2.1. Investigated parameters

Two sets of twenty-four MRFs were designed in accordance with EN1998-1 [8]. The first set of frames covers structures designed using S460 (HSS) for columns and S355 (MCS) for beams. The second set consists of frames made of S355 grade for both beams and columns. The comparison between these two sets allows examining the potential benefit of dual steel frames as respect to those made of single steel grade. For both ensembles of structures the investigated design parameters are shown as follows:

- number of storeys: 4-storey and 8-storey frames, as shown in Fig. 1, where the height of first floor is 4.0 m and all other are 3.5 m;
- span length: 5.0 m and 7.5 m.
- composite steel–concrete column typologies (see Fig. 2): fully encased (FE), partially encased (PE) and concrete filled tube (CFT);
- corner period of the design spectra: two types of soil conditions have been examined. The former representative of soil type C according to EN1998-1 [8] (hereinafter identified as "stiff soil") and the latter representative of very soft soil conditions with corner period of 1.6 s, which are representative of specific soil condition in Bucharest (hereinafter identified as "soft soil");

In order to identify each structure, a label code has been given to the frames as follows:

MRF_(Storey) · (Span) · (Soil) · (Column)

where:

- (Storey) = 1 for eight storeys; 2 for four storeys.
- (Span) = 1 for 5.0 m span; 2 for 7.5 m span.
- (Soil) = 1 for stiff soil; 2 for soft soil.
- (Column) = 1 for fully encased sections; 2 for partially encased sections; 3 for concrete filled tube sections.

The frames integrate the structural system of buildings (Fig. 2) that comply with the regularity conditions in plan and elevations, according to EN1998-1 [8]. As shown in Fig. 3 the building is intended as braced in the transverse direction. Floors consist of orthogonal steel beams (primary and secondary) and steel–concrete composite columns. Primary beams are considered braced to avoid lateral–torsional buckling in order to allow for plastic deformation in bending. All beam–column joints were assumed as full strength and full rigid. The columns were considered to be fixed at the base and continuous through the height. In plan, columns are spaced at L = 7.5 m or L = 5.0 m in both directions, as depicted in Fig. 2.

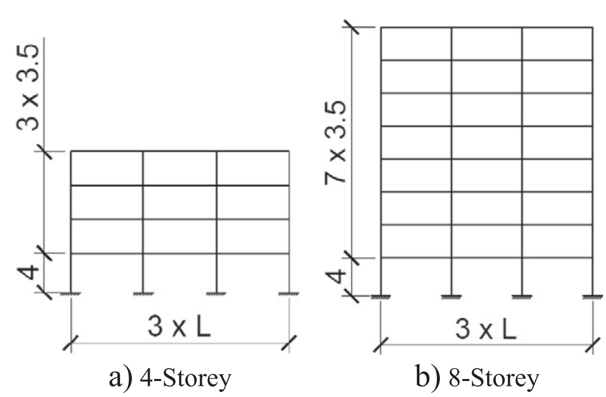

Fig. 1. Structural scheme of study cases.