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Vibration control by damped braces of fire-damaged steel structures subjected to wind and seismic loads

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ABSTRACT

The aim of the present work is to evaluate the effectiveness of viscoelastic-damped braces (VEDBs) to improve the wind and earthquake responses of fire-damaged steel framed buildings, where a significant reduction of stiffness and strength properties of the structural elements following exposure to fire is highlighted. To this end, a ten-storey steel office building, designed for a low-risk zone under the former Italian seismic code and in line with Eurocodes 1 and 3, is considered as test structure. The dynamic response of the test structure in a no fire situation is compared with what would happen in the event of three fire scenarios, on the assumption that the fire compartment with a uniform temperature is confined to the area of the first (i.e. F1), fifth (i.e. F5) and tenth (i.e. F10) level, with the parametric temperature–time fire curve evaluated in line with Eurocode 1. Two retrofitting structural solutions are examined to upgrade the fire damaged test structures, by inserting diagonal steel braces with or without viscoelastic dampers. Frame members are idealized by a bilinear model, which allows the simulation of the nonlinear behavior under seismic loads, while an elastic linear law is considered for diagonal braces. Finally, viscoelastic dampers are idealized by means of a frequency-dependent model.

1. Introduction

In the assessment of fire damage to existing structures there is a high degree of uncertainty about the residual load capacity [1]. During fire, steel frame members of earthquake-damaged structures experience loss of load capacity and stiffness, due to high-temperature induced degradation in strength and Young's modulus of elasticity [2]. This behavior can induce geometrical and mechanical fire damage despite the fact that steel recovers much of its initial strength and stiffness after cooling. The main purpose of the present work is to evaluate the effectiveness of the passive energy dissipation devices, attached to the framed structure via a bracing system, for the fire retrofitting of medium-rise steel buildings later subjected to wind and seismic loads. Damped braces available in the literature differ according to the features of the supplementary damping devices [3]. Attention is focused on viscoelastic dampers (VEDs) which are displacement- and velocity-dependent, so ensuring a restoring force and the activation for vibrations of small amplitude [4].

A ten-storey steel office framed building, which was originally designed for a low-risk zone under the former Italian seismic code [5] and in line with the Eurocodes 1 [6] and 3 [7], is considered as test structure. A numerical fire investigation is preliminarily carried out in the event of three fire scenarios, on the assumption that the fire compartment with a uniform temperature is confined to the area of the first (i.e. F1), fifth (i.e. F5) and tenth (i.e. F10) level. The residual load capacity of the structural members after fire is evaluated considering the reduction factors of stiffness and strength proposed by EC3 [7], at the maximum temperature of the fire compartment (i.e. $T=600\,^{\circ}$C) evaluated by the EC1 time–temperature natural curves corresponding to the design fire load. For each fire scenario, two retrofitting structural solutions are examined to upgrade the test structure damaged by fire: additional diagonal braces; VEDs supported by the additional diagonal braces.

2. Test structure and modeling of the fire

A ten-storey office building with a symmetric plan (Fig. 1a), constituted of moment resisting steel frames (Fig. 1b) and steel-concrete composite deck with horizontal bracing, is assumed as test structure [2]. Three fire scenarios are also reported in Fig. 1b, assuming the fire compartment (Fig. 1a) confined to the area of the first (i.e. F1), fifth (i.e. F5) and tenth (i.e. F10) level, where a uniform temperature is considered as fire condition before wind or earthquake.

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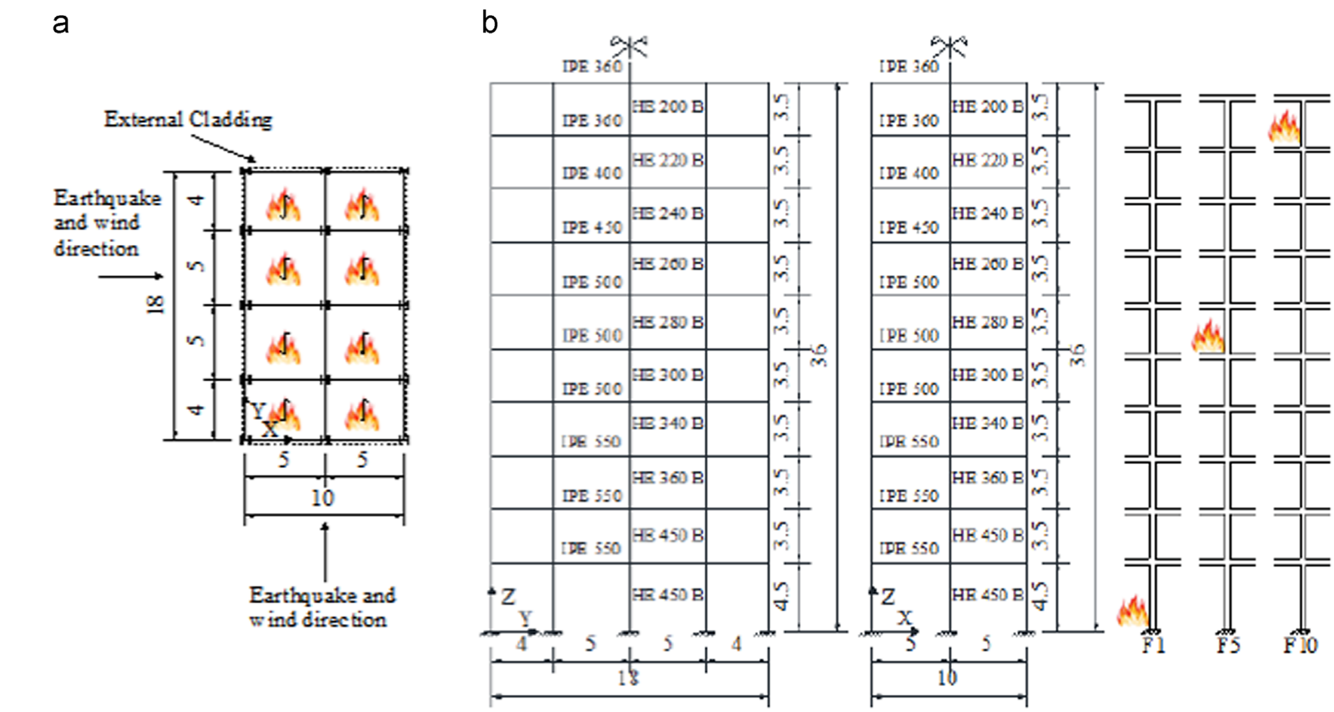

Fig. 1. Test structure (units in m). (a) Plan and fire compartment. (b) Elevation and fire scenarios.

Table 1
Fire parameters in the EC1 time–temperature curves.

T (°C)	First level			Upper levels		
	$q_{f,d}$ (MJ/m^2)	b (J/m^2 s$^{1/2}$ K)	t^*_{max} (h)	$q_{f,d}$ (MJ/m^2)	b (J/m^2 s$^{1/2}$ K)	t^*_{max} (h)
600	186.21	1671	0.10	198.11	1696	0.10

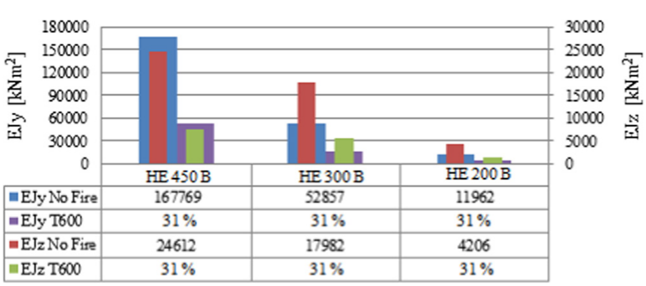

Fig. 2. Flexural stiffness of columns exposed to fire.

The EC1 parametric fire curve is used in the present study to simulate the time–temperature evolution during an actual fire [6], on the assumption that the fire load of the compartment is completely burnt out. Fire parameters in the EC1 time–temperature curves of the first and upper levels are reported in Table 1: i.e. T, maximum temperature; $q_{f,d}$, design fire load corresponding to the surface area of the floor; b, thermal absorptivity of surrounding surfaces of the compartment; t^*_{max}, time when the maximum temperature in the heating phase happens.

3. Fire effects and retrofitting of the test structure

The fire-damage effects on the residual capacity of the test structure are evaluated on the basis of the temperature distribution in the frame members, considering the EC1 time–temperature natural curve of the fire compartment. In accordance with the high thermal conductivity of steel and the thinness of the cross-sections, the assumption of a uniform temperature distribution of $T=600$ °C is admissible after 60 min for the F1 and F5 fire scenarios and after 45 min for the F10 fire scenario. Then, the residual load capacity of the structural members after fire is evaluated considering the reduction factors of effective yield strength and Young's modulus of elasticity of the steel proposed by EC3. In Fig. 2, flexural stiffness of columns is reported along the building height, assuming a direct correspondence between the examined level (i.e. HE450B, HE300B and HE200B) and the fire compartment (i.e. F1, F5 and F10). In particular, major (i.e. EI_y) and minor (i.e. EI_z) axes of bending are examined. Note that a decrease in stiffness of about 69% is obtained in comparison with the no-fire condition.

Plastic domains between axial load (N) and bending moment (M) of columns are shown in Fig. 3 along the major axis of bending. Fire compartments at the first (i.e. F1), fifth (i.e. F5) and tenth (i.e. F10) levels are compared with the no-fire condition at the ambient temperature $T=20$ °C. Moreover, the N–M domain is obtained in line with EC3, simply by replacing the plastic axial load under uniform compression with the buckling load. A marked narrowing of the N–M domain of about 59% is observed. Further details on the fire damage of girders can be found in [2].

For the purpose of retrofitting the fire-damaged structure, from a low- up to a high-risk seismic zone, and controlling the wind-induced vibrations, two structural solutions are examined for each fire scenario: the insertion of diagonal braces at the level where the fire compartment is hypothesized only (AB structure, with "Added Braces"); the insertion of VEDs supported by the additional