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Wind and earthquake dynamic responses of fire-exposed steel framed structures

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ABSTRACT

There is a lack of knowledge on the wind and seismic responses of steel framed structures in the case of fire and an amplification of the structural response is expected in the case of existing structures exposed to fire: i.e. acceleration and deformability thresholds, under wind loads, and damage and buckling thresholds, under seismic loads, can be exceeded at the serviceability and ultimate limit states, respectively. To evaluate the wind and earthquake responses following a fire, a numerical investigation is carried out with reference to the steel framed structure of a 10-storey office building, which was designed for a low-risk zone under the former Italian seismic code and in line with Eurocodes 1 and 3. More specifically, the dynamic response of the test structure in a no fire situation, along the in-plan principal directions, is compared with what would happen in the event of fire, at 500 °C, 550 °C and 600 °C fire temperatures, hypothesising a reduction of stiffness and strength due to fire. Four fire scenarios have been considered on the assumption that the fire compartment is confined to the area of the first level (i.e. F1), the first two (i.e. F1/2) and the upper (i.e. F*i*, $i=5$, 10) levels, with the parametric temperature–time fire curve evaluated in accordance with Eurocode 1. Dynamic analyses are carried out in the time domain using a step-by-step initial stress-like iterative procedure. Along-wind loads are considered assuming, at each level, time histories of the wind velocity based on an equivalent spectrum technique. Real accelerograms, whose response spectra match those adopted by Italian seismic code for a medium-risk seismic zone, are considered to simulate the seismic loads.

1. Introduction

Stiffness and strength properties of steel degrade at high temperatures and this deterioration needs to be properly accounted for in the design of steel structures in the event of fire [1,2]. More specifically, the conventional design of steel framed buildings subjected to wind loads aims to provide stiffness and strength properties of the structural elements to control floor displacements and accelerations, such that an efficient use of the structure and suitable living comfort are guaranteed. On the other hand, the reduction of the buckling load in steel columns needs to be considered at high fire temperatures [3]. In the aseismic design it is commonly accepted that the structural steel members can undergo inelastic deformations under strong ground motions, provided that be kept within an acceptable threshold, while, damage limitation requirements and buckling thresholds are imposed at the serviceability and ultimate limit states, respectively [4–6]. The occurrence of fire following earthquake has gained attention of many recent numerical and experimental studies into the nonlinear response of steel [7–10] and composite steel–concrete [11] structures. Nevertheless, knowledge on the wind and earthquake dynamic responses of steel framed buildings damaged by fire is lacking and an amplification of the structural response for the serviceability and ultimate limit states is to be expected in such cases. Modern techniques can be used to enhance the seismic and/or wind performances of a fire-damaged steel structure through energy dissipation systems (e.g, passive, hybrid, semi-active and active-type systems, which represent additional damping sources) or, in the case of wind, through aerodynamic modifications (e.g., the wind response of a tall building can be improved by means of: sectional shapes chamfered or slotted in plan and/or tapered in elevation, "ad hoc" openings, fins). For a general discussion about these techniques see the works [12–20].

In the present work, the wind and earthquake responses of steel framed structures in a no fire situation, along the in-plan principal directions, are compared with those in which fire occurs, at 500 °C, 550 °C and 600 °C temperatures, assuming stiffness and strength properties of the frame members in line with the reduction factors proposed by Eurocode 3 [21]. To this end, 10-storey steel office buildings are designed in line with the previous Italian seismic code [22] for a low-risk zone, as well as the

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provisions of Eurocode 3. Wind actions are evaluated in compliance with Eurocode 1 [23]. Four fire scenarios are hypothesised assuming the fire compartment confined to the area of the first level (i.e. F1), the first two (i.e. F1/2) and the upper (i.e. Fi, i=5, 10) levels, with the parametric temperature–time fire curve evaluated in line with Eurocode 1. In order to consider along-wind loads, at each storey, time histories of the wind velocity for two return periods (i.e. T_r=10 or 50 years) are assumed, in accordance with an equivalent spectrum technique [24]. Then, real accelerograms, whose response spectra match those adopted by the current Italian seismic code [25] for a medium-risk seismic zone and a medium subsoil class, are considered to simulate seismic loads at serviceability (i.e. operational) and ultimate (i.e. life-safety) limit states. Finally, dynamic analyses are carried out in the time domain through a step-by-step initial stress-like iterative procedure [19,26]. For this purpose, the frame members are idealised by a bilinear model, which allows the simulation of the nonlinear behaviour under seismic loads [27,28].

2. Steel test structure: design and fire modelling

The symmetric steel structure, with a rectangular plan (Fig. 1a), of a 10-storey office building (Fig. 1c) is considered as test structure in this study. More precisely, moment resisting frames are placed to carry (horizontal) wind or seismic loads, while a grid of main and secondary girders support at the floor levels a composite deck with horizontal bracing. A simulated design of the test structure is carried out in line with the previous Italian seismic code (DM96, [22]), for a low-risk seismic region (degree of seismicity S=6, which corresponds to a coefficient of seismic intensity C=0.04), a medium subsoil class (subsoil parameter ε=1) and a coefficient of seismic protection equal to 1.2. Wind actions are evaluated in compliance with Eurocode 1 [23], assuming: flat terrain with a roughness length of 0.30 m; urban area (class B of terrain roughness) with a reference velocity of 28 m/s, which represents a mean value of those assumed for the nine zones of the Italian wind map; altitude of 500 m above sea level. Moreover, the test structure also satisfies the ultimate limit states for strength and buckling evaluated in

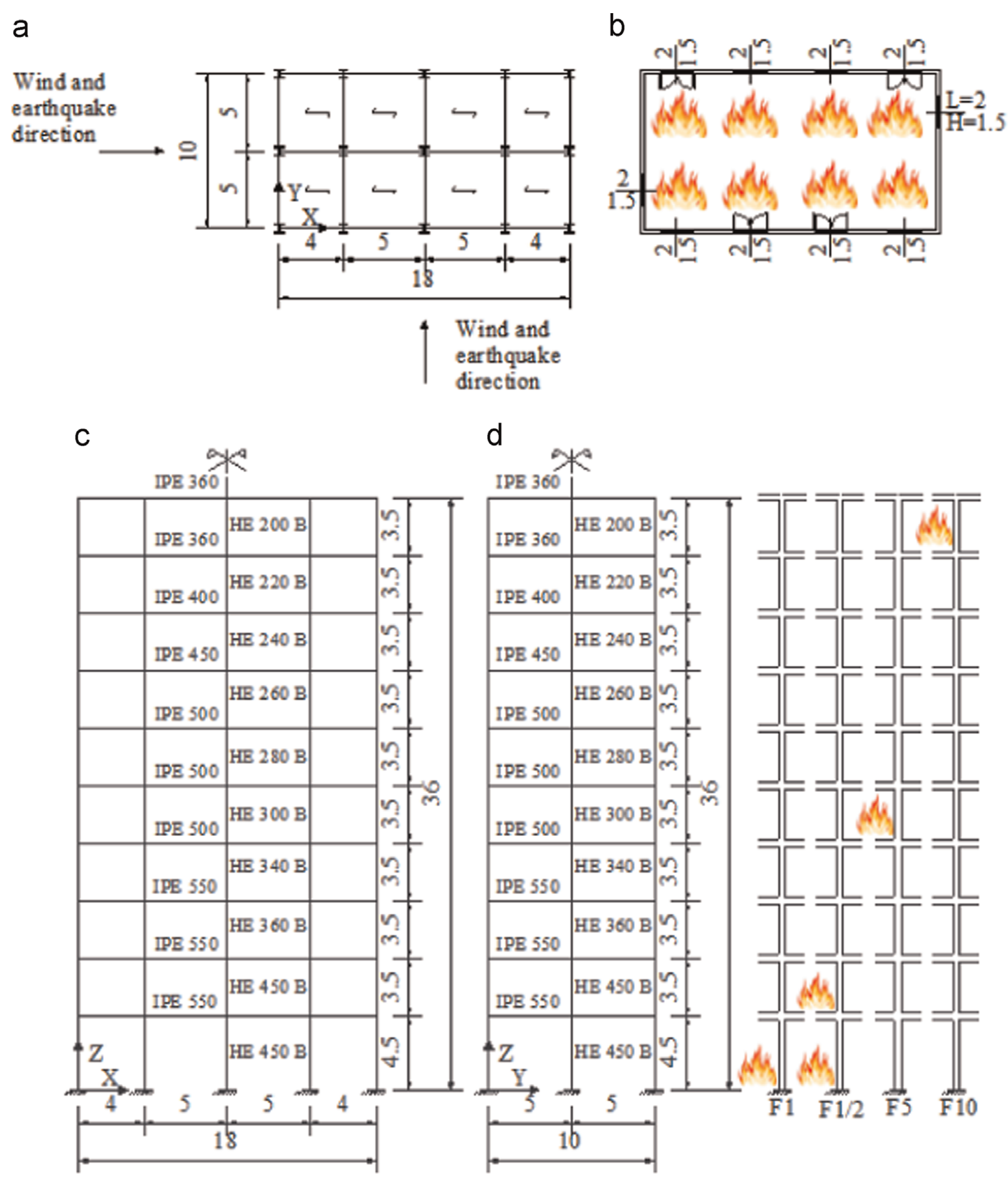

Fig. 1. Steel test structure (dimensions in m). (a) Plan. (b) Fire compartment. (c) Elevation. (d) Fire scenarios.