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Seismic vulnerability and retrofitting by damped braces of fire-damaged r.c. framed buildings

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ABSTRACT

There is a lack of knowledge regarding the seismic response of fire damaged reinforced concrete (r.c.) buildings; moreover, an amplification of the damage is to be expected for such structures in the event of an earthquake. To evaluate the nonlinear seismic response following a fire, a numerical investigation is carried out with reference to a five-storey r.c. framed building, which, designed according to the previous Italian seismic code for a medium risk zone now, needs to be considered as in a high-risk seismic zone in line with the current Italian seismic code. More specifically, the nonlinear seismic response of the test structure in a no fire situation is compared with that in the event of fire, at 45 (i.e. R45) and 60 (i.e. R60) minutes of fire resistance, assuming damaged (i.e. DS) and repaired (i.e. RS) stiffness in combination with damaged strength conditions. Then, the fire-damaged test structures are retrofitted by the insertion of hysteretic damped braces (HYDBs), placed only at the storey where the fire compartment is hypothesized. Five fire scenarios have been considered on the assumption that the fire compartment is confined to the first level (i.e. F1), the first two (i.e. F1/2) and the upper (i.e. Fi, $i = 3–5$) levels, with the parametric temperature–time fire curve in accordance with Eurocode 1. The nonlinear dynamic analysis is performed through a step-by-step procedure based on a two-parameter implicit integration scheme and an initial-stress-like iterative procedure. At each step of the analysis, plastic conditions are checked at the critical (end) sections of the girders and columns, where a thermal mapping with reduced mechanical properties is evaluated in accordance with the 500 °C isotherm method proposed by Eurocode 2, while the behaviour of a HYDB is idealized through the use of a bilinear law provided that buckling is prevented.

1. Introduction

Assessment of the seismic vulnerability and retrofitting of existing structures where fire safety is neglected represent a far-reaching problem especially in the case of reinforced concrete (r.c.) framed buildings designed for vertical loads only or with erroneous seismic zone classifications and code provisions. Traditional retrofitting techniques are based on increasing strength and stiffness, by adding new structural elements to the system (e.g. r.c. shear wall or steel brace) and/or enlarging the existing members (e.g. steel encasing or concrete jacketing). However, the increase of seismic resistance capacity is generally combined with an increase of the seismic demand and it is possible that the retrofitted structure will be less safe than in the original condition [1]. On the other hand, it is well known that, even after a strong earthquake, greater seismic protection can be obtained by using supplementary damping devices supported by steel braces [2,3]. At present, a wide variety of damped braces is available [4–12]: displacement-dependent (e.g. friction damper, FRD; hysteretic damper, HYD), velocity-dependent (e.g. viscoelastic damper, VED; viscous damper, VSD) or self-centring (e.g. shape memory alloys, SMA). In the present work the attention is focused on metallic yielding HYDs, which are characterized by a stable hysteretic behaviour independent on temperature and velocity of motion. These devices are generally manufactured from traditional materials and require little maintenance, representing a low cost and reliable solution for energy dissipation.

In the conventional aseismic design it is accepted that structures can withstand strong ground motions by undergoing inelastic deformations. Consequently, fire can be a serious problem for a structure that has been partially damaged in a prior seismic event, because fire resistance will decrease [13]. More specifically, the fire response of r.c. frame members depends on the thermal (i.e. thermal conductivity, specific heat, thermal diffusivity and mass loss), mechanical (i.e. compressive and tensile strength, modulus of elasticity and stress–strain law) and deformation (i.e. thermal

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expansion and creep) properties of concrete and reinforcing steel bars, changing substantially with heating rate, strain rate and temperature gradient [14]. The combined effect of earthquake and fire has gained attention of recent studies into the nonlinear response of r.c. framed [15] and frame-wall [16] structures.

On the other hand, earthquakes following fires may find structures whose seismic resistance is considerably reduced. Numerical [17–19] and experimental [20,21] studies have been carried out on the assessment of the residual seismic load capacity of r.c. structures, in terms of stiffness, strength and ductility after fire. In most cases of fire, structures experience degradation of material properties, due to high temperature, and damage to the structural members, from thermal expansion. In addition, fire induces spalling of concrete that can play a significant role in the seismic performance of r.c. frame members. However, despite the lack of knowledge on the seismic vulnerability of existing framed structures in the event of fire we can expect an amplification of the structural damage in the case of existing structures that have been exposed to fire.

In the present work, the nonlinear seismic response of r.c. framed structures in a no fire situation is compared with that in which fire has occurred, at 45 (i.e. R45) and 60 (i.e. R60) minutes of fire resistance, assuming damaged (i.e. DS) and repaired (i.e. RS) stiffness of the frame members in combination with damaged strength conditions. To this end, five-storey r.c. office buildings are designed in line with the previous Italian seismic code [22] for a medium risk zone. A numerical fire investigation is preliminarily carried out considering a thermal–mechanical mapping analysis, with reduced mechanical properties evaluated in accordance with the 500 °C isotherm method proposed by Eurocode 2 [23], followed by a sequentially uncoupled nonlinear dynamic analysis [24,25]. Then, the fire-damaged test structures are retrofitted by the insertion of hysteretic damped braces (HYDBs), placed only at the storey where the fire compartment is hypothesized. In order to study the seismic response of the r.c. framed building damaged from fire, real ground motions corresponding to a high-risk zone and two topographic conditions are considered in line with the current Italian seismic code [26]. Five fire scenarios are hypothesized 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, $i = 3–5$) levels, with the parametric temperature–time fire curve evaluated in accordance with Eurocode 1 [27].

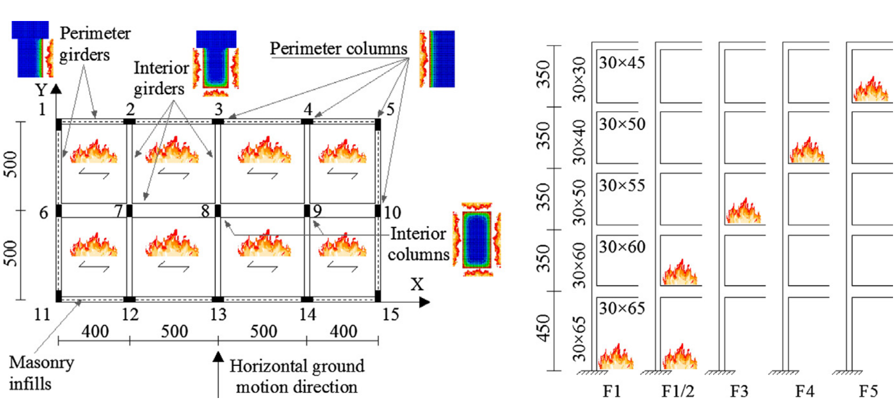
2. Test structure: Design and fire modelling

A typical five-storey office building, with a r.c. framed structure shown in Fig. 1a, is considered as test structure. Non-structural elements regularly distributed in elevation, such as perimeter masonry infills, are considered. For the sake of simplicity, the plane frames orientated along the horizontal ground motion direction (Y), perpendicular to the floor slab direction (X) shown in Fig. 1a, are considered as a reference scheme. The dimensions of the cross sections assumed for the columns and the girders, equal at a given level and regularly tapering in elevation, are reported in Fig. 1b. A simulated design of the test structure is carried out in line with the previous Italian seismic code [22], for a medium-risk seismic region (degree of seismicity $S = 9$, which corresponds to a coefficient of seismic intensity $C = 0.07$) and a typical subsoil class (subsoil parameter $\varepsilon = 1$). The gravity loads are represented by a dead load of 4.48 kN/m^2 on the top floor and 5.18 kN/m^2 on the other floors, and a live load of 3.0 kN/m^2 on all the floors; infill walls, regularly distributed in elevation along the perimeter are assumed to weigh, on average, about 2.7 kN/m^2. A cylindrical compressive strength of 20 N/mm^2 for the concrete and a yield strength of 375 N/mm^2 for the steel are assumed for the r.c. frame members. The design complies with the ultimate limit states satisfying minimum conditions for the longitudinal bars of the girders and columns: at least two 12 mm bars are provided both at the top and bottom throughout the entire length of the frame members; for the girders, a tension reinforcement ratio not less than 0.37% (for the assumed yield strength) is provided and, at their end sections, a compression reinforcement not less than half of the tension reinforcement is placed; minimum steel geometric ratio is 1% for the symmetrically-reinforced section of each column. The dynamic properties of the six main vibration modes are reported in Table 1: i.e. vibration period (T_i); effective masses in the X ($m_{E,X}$) and Y ($m_{E,Y}$) directions, expressed as percentage of the total mass (m_{tot}).

Five fire scenarios are reported in Fig. 1a and b, assuming 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. Fi, $i = 3–5$) levels. It is worth noting that F1/2 fire scenario is obtained from F1 and F2, which occur simultaneously. The geometric properties of the fire compartment are reported in Table 2, for the first and the upper

Table 1
Dynamic properties of the test structure ($m_{tot} = 9.35$ kN s^2/cm).

Mode	T_i (s)	$m_{E,X}$ (%m_{tot})	$m_{E,Y}$ (%m_{tot})
1	0.808	81.6	0
2	0.749	0	80.4
3	0.326	13.0	0
4	0.302	0	13.5
5	0.195	3.4	0
6	0.183	0	3.8



(a) Plan and fire compartment.

(b) Elevation and fire scenarios.

Fig. 1. R.c. test structure (dimensions in cm).