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Effects of site condition in near-fault area on the nonlinear response of fire-damaged base-isolated structures

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ABSTRACT

Structural damage due to resonance, when the predominant vibration periods of both site and structure are close, is expected in the case of base-isolated structures subjected to near-fault earthquakes. Little is known, however, about the effects of different site conditions on the nonlinear seismic response of fire-weakened base-isolated structures. To analyze this aspect, eight five-storey reinforced concrete (r.c.) office buildings, base-isolated with High-Damping-Laminated-Rubber Bearings (HDLRBs), are designed in line with the Italian seismic code (NTC08) assuming four subsoil types, ranging from rock-site to soil-site, combined with relatively low and high values of the ratio between the vertical and horizontal stiffnesses of the HDLRBs. Three fire scenarios are considered at 45 (i.e. R45) and 60 (i.e. R60) minutes of fire resistance, with the fire compartment confined to the area of the first level (F1), the lower two levels (F1/2) and the fifth level (F5). A finite element thermal model to measure temperature distribution in the cross-section for columns, exposed to fire on one and four sides, and beams, exposed to fire on one and three sides, is considered in combination with the parametric time–temperature fire curve evaluated in line with Eurocode 1. Then, a nonlinear incremental dynamic analysis is carried out which takes into consideration the horizontal and vertical components of near-fault ground motions recorded at different site conditions from rock- to soil-site and normalized with respect to the NTC08 ones in accordance with the Modified Velocity Spectrum Intensity. Plastic conditions are assessed at the potential critical sections of the beams (i.e. end sections of the sub-elements in which a beam is discretized) and columns (i.e. end sections), whose reduced mechanical properties are evaluated in accordance with a thermal mapping and considering the 500 °C isotherm method proposed by Eurocode 2. Maximum ductility demand under the horizontal and vertical components of near-fault ground motions depends on the position of the fire compartment and increases moving from rock-site to soil-site.

1. Introduction

Forward-directivity and fling-step effects in near-fault areas can produce intense velocity pulses oriented in the fault-normal and fault-parallel directions, respectively. In particular, forward-directivity produces two-sided velocity pulses without permanent ground displacement while fling-step is the result of permanent ground displacement that generates one-sided velocity pulses [1]. An elongation of the pulse period at the soil-site rather than at the rock-site is expected for low values of magnitude, while this difference decreases for increasing values of magnitude [2]. On the other hand, analytical and field evidence [3,4] clearly highlights the fact that the high frequency vertical component of near-fault ground motions is characterized by site-dependent high values of vertical-to-horizontal peak ground acceleration and spectral acceleration ratios, contrary to the design values imposed by European [5] and Italian [6] seismic codes.

To evaluate the effects in near-fault areas on the nonlinear response of a structure, an intensity measure is necessary to provide scale factors of the recorded ground motions in order to predict the seismic structural response minimizing the record-to-record variations [7]. It is well known that non structure-specific intensity measures (e.g. peak ground acceleration and Arias intensity) are generally inefficient because they do not consider the properties of the structure in question. Consequently, a structure-specific intensity measure is required for structures responding significantly in their higher vibration modes or far into the inelastic range [8,9].

In the last two decades the seismic isolation technique has proved to be highly effective for the protection of reinforced concrete (r.c.) framed buildings located in seismic areas [10–16], with the assumption that the elastic behavior of the superstructure is

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guaranteed. Base-isolation with elastomeric bearings allows considerable reduction of the horizontal seismic loads transmitted to the superstructure, whose behavior is isolated or fixed-base along the vertical direction depending on the value, respectively low or high, of the ratio $\alpha_{K0}(=K_{V0}/K_{H0})$ between the vertical (K_{V0}) and horizontal (K_{H0}) nominal stiffnesses of the isolation system. However, structural damage due to a resonance effect, when the vibration periods of both site and structure are close, is expected in the case of base-isolated structures subjected to the horizontal components of near-fault earthquakes [17,18]. Moreover, plastic hinges are expected along the span of the beams due to the vertical component of these motions, especially at the upper storeys where the effects of the gravity loads generally prevail over those of the horizontal seismic loads and an amplification of the vertical motion is expected depending on the vertical stiffness of the isolators [18,19]. Aftershocks following fire may find seismic resistance considerably reduced, the properties of concrete and reinforcing steel bars being changed with heat, strain rate and temperature gradient [20–25]. In this case, the frequency content of the horizontal motion transmitted by the isolators to the superstructure can often become critical as soon as the strength level of a fire-weakened base-isolated structure is reduced and the ductility demand can increase at the lower levels much more rapidly for an ever-lower strength level, especially when the fire compartment is at these levels [26]. On the other hand, a marked amplification in global ductility demand at the fire-exposed upper levels of base-isolated structures, rather than base-isolated structures in the no-fire condition, is obtained for increasing values of the acceleration and spectral acceleration ratios [27].

The above considerations emphasize the need to investigate the nonlinear seismic behavior of fire-weakened base-isolated structures at different sites in a near-fault area where aftershocks are hypothesized, assuming that the horizontal and vertical components of the ground motion occur simultaneously. To this end, eight r.c. five-storey office buildings, base-isolated with fire-protected High-Damping-Laminated-Rubber Bearings (HDLRBs), are designed in accordance with the Italian seismic code (NTC08) [6], with four ground types (i.e. subsoil classes A, B, C and D) combined with two values, relatively low (i.e. 400) and high (i.e. 2400), of the nominal stiffness ratio α_{K0} of the HDLRBs. Three fire scenarios are considered at 45 (i.e. R45) and 60 (i.e. R60) minutes of fire resistance, with the fire compartment confined to the area of the first level (F1), the lower two levels (F1/2) and the fifth level (F5). A finite element thermal model of cross-section [28] for columns, exposed to fire on one and four sides, and beams, exposed to fire on one and three sides, is considered in combination with the parametric time–temperature fire curve evaluated in line with Eurocode 1 [29]. Then, reduced stiffness, strength and ductility properties of the superstructure are evaluated with the 500 °C isotherm method proposed by Eurocode 2 [30]. A nonlinear incremental dynamic analysis is carried out considering the horizontal and vertical components of near-fault ground motions recorded from rock- to soil-site and scaled in line with a Modified Velocity Spectrum Intensity measure [8]. Plastic conditions are checked at the potential critical sections of the beams (i.e. end sections of the sub-elements in which a beam is discretized) and columns (i.e. end sections), where reduced strength and ductility are evaluated. A viscoelastic model with variable stiffness in the horizontal and vertical directions, depending on the axial force and lateral deformation, simulates the response of an HDLRB.

2. Fire layout and seismic design of the base-isolated test structure

A five-storey office building with symmetric plan, whose r.c. framed structure is base-isolated by fifteen identical fire-protected HDLRBs, is considered in Fig. 1, where length and cross-sections of the frame members are also shown. Perimeter masonry infills, assumed as non-structural elements regularly distributed in elevation, are considered (Fig. 1a). For the sake of simplicity, the plane frames orientated along the y direction are assumed as reference scheme (Fig. 1a). In order to account for the plastic deformations along the beams, each is discretized into four sub-elements of the same length and lumped masses are considered at the end, quarter-span and mid-span sections (Fig. 1b). Eight cases of base-isolated structures in the no-fire condition, obtained by combining two values of the nominal stiffness ratio α_{K0} for the HDLRBs (i.e. 400 and 2400) and four site conditions (i.e. subsoil classes A, B, C and D, corresponding to rock-, stiff-, medium- and soil-site, respectively), are considered as test structures. Each base-isolated structure (BI) is labelled with two symbols: the first indicates the subsoil class, the second the value of α_{K0}. Moreover, the base-isolated structures in the no fire condition are compared with those in which fire occurs at 45 (i.e. R45) and 60 (i.e. R60) minutes of fire resistance. Three fire scenarios are reported in Fig. 1, assuming the fire compartment confined to the area of the first level (i.e. F1), the first two levels (i.e. F1/2) and the fifth level (i.e. F5). It is worth noting that the F1/2 fire scenario is obtained from the F1 and F2 ones, which are assumed to occur simultaneously.

The design of the base-isolated structures is performed in a high-risk seismic zone assuming, besides the gravity loads, the horizontal seismic loads acting in combination with the vertical ones and supposing elastic response of the superstructure (i.e. behavior factors for the horizontal and vertical seismic loads, $q_H = q_V = 1.0$). For each subsoil class provided by the Italian seismic code (NTC08), the main parameters of the horizontal and vertical elastic response spectra, for the ultimate life-safety (in the design of superstructure) and collapse (in the design of base-isolation system) limit states are reported in Table 1: subsoil parameters in the horizontal (S_H) and vertical (S_V) directions; peak ground acceleration in the horizontal (PGA_H) and vertical (PGA_V) directions; acceleration ratio $\alpha_{PGA}(=PGA_V/PGA_H)$. The gravity loads used in the design are represented by dead and live loads, equal respectively to: 4.4 kN/m^2 and 3 kN/m^2, for the top floor; 6.1 kN/m^2 and 3 kN/m^2, for the isolated floor; 5.7 kN/m^2 and 3 kN/m^2, for the other floors. The weight of the perimeter masonry infills is taken into account by considering a gravity load of 2.7 kN/m^2. A cylindrical compressive strength of 25 N/mm^2 for the concrete and a yield strength of 450 N/mm^2 for the steel are assumed for the r.c. frame members.

The design of the superstructure has been carried out so as to satisfy minimum conditions for the longitudinal bars of the beams and columns, according to the provisions for low ductility class imposed by the NTC08. The HDLRBs fulfill the collapse limit state verifications for the maximum shear strains: i.e. $\gamma_{tot} \leqslant 5$ and $\gamma_s \leqslant 2$, where γ_{tot} and γ_s represent the total shear strain and the shear strain of the elastomer due to seismic displacement, respectively. Moreover, the maximum compression axial load (P) does not exceed the critical load (P_{cr}) divided by a safety coefficient equal to 2.0. The minimum tensile stress (σ_{tu}) resulting from the seismic analysis is assumed as $2G(=0.7$ MPa, for a shear modulus of the elastomer $G = 0.35$ MPa). In Table 2, depending on the subsoil class and stiffness ratio α_{K0} considered in the analysis, the following geometric properties of the HDLRBs are reported: the diameter of the isolator (D); the total thickness of elastomer (t_e); primary (S_1) and secondary (S_2) shape factors; results of the verifications for the HDLRBs are also reported in Table 2. Note that the design of the isolators depends on the buckling control for $\alpha_{K0} = 400$, while the conditions imposed on the maximum values of γ_{tot} or γ_s prove to be more conservative for $\alpha_{K0} = 2400$. Finally, tensile forces are found in the isolators of the BID400 and