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Structural and non-structural intensity measures for the assessment of base-isolated structures subjected to pulse-like near-fault earthquakes

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ABSTRACT

The objective of the present work is to improve selection procedures for intensity measures (IM) to be used for scaling near-fault earthquakes, in order to minimize the variability in the prediction of engineering demand parameters (EDPs). To this end, the predictive capability of nine spectral IMs, among the most commonly used in the literature, is investigated with reference to three EDPs evaluated for base-isolated structures subjected to near-fault earthquakes, which can be characterized by pulse-type motions in the horizontal direction and significant vertical component. A six-storey reinforced concrete (r.c.) framed building originally designed as fixed-base, in compliance with a former Italian seismic code for a medium-risk zone, is retrofitted by means of friction pendulum bearings (FPBs), to attain performance levels imposed by the current Italian code in a high-risk seismic zone. The nonlinear dynamic analysis is carried out by a lumped plasticity model for the r.c. frame members, including a 26-flat surface axial load-biaxial bending moment elastic domain at the end cross-sections, and a nonlinear force-displacement law for the FPBs, that consider variable axial load combined with friction coefficient as function of the sliding velocity. Two sets of accelerograms are considered, consisting of near-fault records with significant horizontal pulses, selected by using a pulse index, and significant vertical component, selected by the ratio between the peak of the vertical and the horizontal ground acceleration.

1. Introduction

In the literature there are many ground motion intensity measures (IMs), tested on different types of buildings [1–5], which can be divided into two main categories: (a) structural IMs obtained from response spectral ordinates calculated at the fundamental vibration period of the structure or derived from integration over a suitable range of vibration periods; (b) non-structural IMs calculated from ground motion time histories (i.e. acceleration-, velocity- and displacement-related). Moreover, the optimal IM in terms of sufficiency, efficiency and practicality depends on both the type of structure and engineering demand parameters (EDPs) [5,6]. More specifically, efficiency refers to variability of the EDP for a given IM while sufficiency describes how much the IM is statistically independent of ground motion characteristics. However, there is no information in current seismic codes on how to scale near-fault ground motions appropriately when assessing, through nonlinear dynamic analysis, structures retrofitted with a base-isolation system. Current Italian seimic code (NTC08, [7]) prescribes that the response spectra of the selected earthquakes should be scaled so as to match the design spectrum in a suitable range of vibration periods. Similarly, European seismic code (EC8, [8]) requires the mean 5% damping elastic spectrum, calculated from all time histories, should not be less than 90% of the corresponding value of the 5% damping elastic response spectrum in the range of periods between $0.2T_1$ and $2T_1$, where T_1 is the fundamental period of the structure. In this way, both seismic codes take into account the lengthening of vibration periods as the structure undergoes inelastic deformations, also considering the effects of the higher vibration modes. On the other hand, the U.S.A. seismic code (ASCE/SEI7-10, [9]) considers pairs of horizontal ground motions scaled such that the average of the square root of the sum of the squares spectra from all horizontal component pairs does not fall below 1.3 times the corresponding ordinate of the design response spectrum. Finally, the U.S.A. seismic guidelines (USACE, [10]) provide limited information on the vertical component of ground motion, which is generally scaled by the same scaling factor used for the pair of horizontal ground motions.

Ground motions recorded at short site-to-source distances (i.e. less than 20 km) can be quite diverse from those recorded at greater distances, producing an amplification of the structural damage in the case of base-isolated structures [11,12]. However, NTC08 [7] and EC8 [8] do not consider specific scaling factors of near-fault ground motions, while ASCE/SEI7-10 [9] only ensures that horizontal compo-

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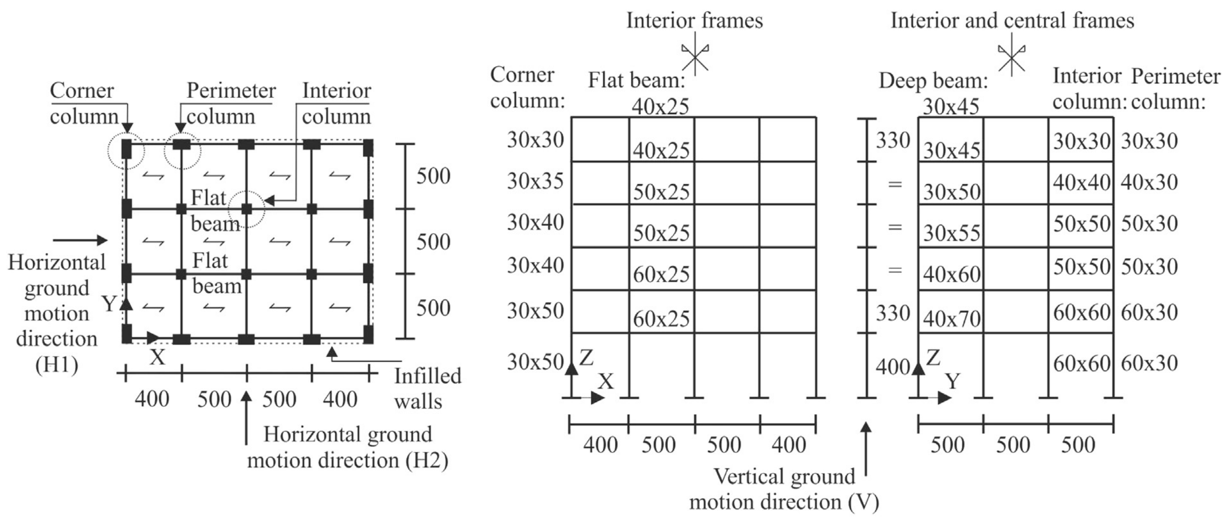

(a) Plan. (b) Elevations.

Fig. 1. Fixed-base structure (units in cm).

nents be rotated to the fault-normal and fault-parallel directions. Previous studies (e.g. [13]) have shown that for buildings where most of the mass participates in the first mode (i.e. regular fixed-base or base-isolated structures) spectral acceleration (S_a) may be the preferred IMs for ordinary far-fault ground motions. On the other hand, many IMs based on spectral acceleration are proposed for tall buildings or structures that undergo high nonlinearities: e.g. a two-parameter scalar intensity measure given as a combination of the S_a values at elastic and elongated first vibration period [14]; a vector of IMs including the S_a value evaluated at higher vibration periods [15]; a vector of IMs including an "epsilon" parameter to improve the prediction capability of S_a [16]; a vector of IMs combining the S_a values of the bi-directional (horizontal) ground motion [17]. Moreover, other studies have investigated the efficiency of alternative scalar and vector-valued IMs to predict the structural seismic performance of fixed-base buildings, also taking into account the pulse-like nature of near-fault earthquakes [4,18–20]. Finally, in the assessment of engineering demand parameters (EDPs) of two-dimensional models of base-isolated structures, the predictive capability of some IMs has been investigated in the literature: e.g. the velocity-related IMs are well correlated with the nonlinear response of the base-isolation system (e.g. the peak ground velocity, [21]) and superstructure (e.g. the relative input equivalent energy velocity spectrum intensity, [5]). However, few studies have focused on three-dimensional models of fixed-base buildings subjected to near-fault pulse-type ground motions [22] while information is lacking for base-isolated structures, where the efficiency of the IMs significantly depends on: the period ratio α_T, obtained from the ratio between the pulse-period (T_P) and the fundamental vibration period ($T_{I,H}$) of the base-isolated structure [23,24]; the acceleration ratio α_{PGA}, defined as the ratio between the peak values of vertical (PGA_V) and horizontal (PGA_H) ground acceleration [25].

The aim of the present work is to investigate the effectiveness of a number of the most frequently used IMs to predict EDPs in the base-isolation system and superstructure of reinforced concrete (r.c.) structures subjected to the horizontal (bi-directional) and vertical components of near-fault ground motions. Spatial (i.e. 3D) and planar (i.e. 2D) structural models of the test structure are subjected to the horizontal components of the selected earthquakes acting alone or in combination with the vertical one. Thus, the following cases are examined: i) 3D model subjected to bi-directional horizontal components; ii) 2D model, along both the in-plan X and Y principal directions, subjected to each of the horizontal components; iii) 3D model subjected to the tri-directional horizontal and vertical components. For this purpose, a six-storey r.c. framed building originally designed as fixed-base, in compliance with a former Italian seismic code for a medium-risk zone [26], is retrofitted with a friction pendulum (FP) system, to attain performance levels imposed by the current Italian code in a high-risk seismic zone [7]. The nonlinear analysis of the test structure is performed by means of a step-by-step procedure based on an initial-stress-like iterative procedure [27,28] and a two-parameter implicit integration scheme [29] so as to avoid the following drawbacks: numerical instability for time step relatively large, instability under roundoff error and beat phenomenon between spurious solutions. At each step of the analysis, plastic conditions are checked at the potential critical sections of beams and columns, where a 26-flat surface modelling of the axial load-biaxial bending moment elastic domain is adopted [30,31]. The response of a FP bearing is described by a nonlinear force-displacement law, with friction variability depending on sliding velocity and axial load [32]. A set of 43 near-fault records with significant horizontal pulses (SH) is selected from the Pacific Earthquake Engineering Research center database [33]. Then, a subset of 14 near-fault records with significant vertical components (SV) is selected in the SH set, considering accelerograms characterized by a value of the acceleration ratio α_{PGA} greater than 1.0 with reference to at least one of the horizontal components.

2. Fixed-base structure: layout, design and nonlinear modelling

A six-storey residential building with reinforced concrete (r.c.) framed structure, whose symmetric plan and elevations are shown in Fig. 1a and b, respectively, is considered as test structure for the numerical investigation. A simulated design of the fixed-base (unretrofitted) framed building is carried out in accordance with the former Italian seismic code [26], for a medium-risk seismic region and a medium-dense subsoil class. The gravity loads for the r.c. framed structure are represented by a dead load of 4.2 kN/m^2 on the top floor and 5.0 kN/m^2 on the other floors, and a live load of 2.0 kN/m^2 on all the floors; an average weight of about 2.7 kN/m^2 is considered for the masonry infill walls. Concrete cylindrical compressive strength of 25 N/mm^2 and steel reinforcement with yield strength of 375 N/mm^2 are considered. The geometric dimensions of beams and columns, whose design is carried out to comply with the ultimate limit states, are reported in Fig. 1b. Minimum reinforcements of the r.c. frame members are also imposed: for the beams, a tension reinforcement ratio nowhere less than 0.37% is provided and a compression reinforcement not less than half of the tension reinforcement is placed