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Identification of the structural model and analysis of the global seismic behaviour of a RC damaged building

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ABSTRACT

The study of the structural behaviour of damaged RC buildings during ground motion is a fundamental topic in the modern earthquake engineering. Many studies have been carried out in order to better understand the real evolution of the damage in RC buildings during a seismic event. In this work, a damaged RC building has been intensively investigated in terms of materials property and stiffness evolution in order to interpreting the structural and nonstructural surveyed damage. The peculiarity of this building is its damage sequence during the 2002 Molise earthquake. In fact, the town of Bonefro suffered moderate damage (MCS intensity VII), with the exception of the investigated reinforced concrete building. The October 31, 2002 event ($M=5.4$) caused some structural damage to this building. The second event ($M=5.3$), on November 1, 2002, increased substantially the damage level (grade 4 according to the 1998 European Macroseismic Scale). It occurred just while, due to fortuitous circumstances, a 5 min. seismic velocimetric recording was being taken. The working group has performed some frequency analyses based on the recording. Several non linear models have been defined to understand the damage evolution of the building and the local and global damage patterns through for static analyses. Finally, linear and non linear models have been developed with the main goal of identifying the characteristics of a reliable undamaged structural model.

1. Introduction

The recent researches show that the real structural behaviour of RC buildings during earthquake is a fundamental topic in modern earthquake engineering. Many studies have been carried out in order to monitoring and understanding the evolution of the damage in RC buildings [1–7].

In this work, a RC moment resistant framed building has been intensively investigated in terms of materials property and stiffness evolution in order to interpreting the structural and non-structural damage due to a strong ground motion. In fact, the peculiarity of this building, located in Bonefro (CB, Italy), is its damage sequence during the 2002 Molise earthquake [8–10].

The city of Bonefro suffered moderate damage (IMCS=VII) in the seismic events of October 31 ($M=5.4$) and November 1, 2002 ($M=5.3$), which affected the Regions of Molise and Puglia in Southern Italy. The type and level of damage found on two close reinforced concrete (RC) buildings was particularly interesting. Adjacent RC buildings, are located outside the historical centre of Bonefro and they were designed and realised on 1982–83 by IACP (Istituto Autonomo Case Popolari). The two buildings, quite similar except for the number of storey, four and three respectively, were designed assuming a medium seismic design level, as defined by the Italian Code in force, although Bonefro was not classified as seismic area at that time. The two buildings suffered different damage. After the first shake, on October 31, the three storey building was only slightly damaged, while the four storey building suffered a much heavier damage ($d=3$, EMS98 [11]). The second shake increased the damage of the two structures to $d=2$ and $d=4$, respectively. Damaged building has, therefore, been assumed as case study and it has been widely investigated within the so-called BOB-CODE (BOnefro Building COmparison of Damage Estimates) Italian project [9,10].

Due to fortuitous circumstances, a velocity time-history was recorded at the top storey of the most damaged building. It permits to follow the evolution of the dynamic characteristics of the building, but being made after the first shake on October 31, on the already damaged structure, the measures cannot provide information on the dynamic characteristics of the undamaged structure. Moreover, some geophysical measurements were subsequently

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carried out near the building, in order to evaluate the influence of possible resonance effects between the more damaged building and the soil. A free field accelerometer was set up and some microtremor measures NASW [12] were carried out on the foundation soil, together with velocimetric measures in ambient vibrations on the less damaged three storey building, in order to obtain useful indications on the possible causes of damage [1,8,13].

In order to evaluate the seismic response of the studied building, accurate Non-Linear models were defined to study the evolution and distribution of the damage mechanisms. Recent studies have been highlighted that Static Non-Linear Analysis are the most appropriate procedure to predict the seismic damages distribution and evolution on structural and non-structural elements, at least for quite regular structures. Unfortunately, the difficulties in the definition of an accurate non linear modelling (such as displacement capacity of brittle elements) imply that there are still many problems on reliability of the previsions of the seismic response.

The main goal of the present work is to define an accurate non linear model of the building, calibrated on the results of velocimetric data analysis and that take into account of structural and non-structural properties directly measured on site, in order to rebuild a reliable model describing the undamaged condition as well as the damage evolution observed on the building.

2. General overview and experimental data of the building

The investigated building was built by IACP of the Province of Campobasso with reinforced concrete structure made of three frames along the longitudinal direction, X, and four frames in the transverse direction, Y (Fig. 1a). The RC staircase is eccentrically placed with respect to the longitudinal axis. The height of the first story is 2.9 m, while it is 3.1 m for other stories. The total height of the building is 13.85 m (Fig. 1b). The external frames have all deep beams, while the internal frame beam depth is equal to the slab thickness. All columns of the ground storey are 35×35 cm, with 6ϕ16 rebar, while the columns of the other storeys are 30×30, with 4ϕ16+2ϕ14 rebar at the 2nd story and 4ϕ16 at 3rd, 4th and roof level (Fig. 1c). The foundations are made of 75 cm wide, 60 cm deep beams. The tile-concrete floor slabs are 20 cm thick (16+4), while infill masonry panels are made of a double layered hollowed brick masonry.

The procedure presented in this paper aims substantially at solving an inverse problem, in which the numerical model adopted to simulate the damage evolution of the building has been suitably calibrated considering a significant collection of data directly recorded on the building. The data regard the overall structural dynamic behaviour before and after the second shake and the mechanical properties of material obtained by experimental tests in site.

The first parameter considered in the work is the fundamental modal frequency variations. The velocimeter placed at the last level of the staircase during the second shake recorded the following sequence: 1 min ambient vibration, 3 min transient seismic vibrations and 1 minute ambient vibrations. The first derivate of the records were before smoothed in order to obtain the accelerations and then analysed using the Short Time Fourier Transform (STFT), as shown in Fig. 2, to describe the variation in the time of the main frequencies of the building due to damage accumulation during the earthquake. The STFT is a special function that applies the Fourier Transform to a sub part of the whole signal through a sliding window technic (Windowing) [14]. In such way it is possible to follow in a 2D space (Time–Frequency) the variation in the time of the frequencies detected by the Fourier Transform in a single window.

The Fig. 2 shows that a sudden decrease of the main frequency occurred in both direction, in correspondence of the maximum acceleration values, due to the activation on nonlinear damage mechanisms. The initial and final part of the diagrams of Fig. 2 show, instead, the behaviour of the building in stationary conditions before and after the shake. For the numerical model calibration these last frequency values have been considered.

It is worth to note, however, that this value can be affected by the accuracy and the type of analysis used and even by a series of external parameters such as temperature, solar radiation, water content in the soil, etc.. The proper use of these data for the purpose of numerical models calibration would require, strictly speaking, an evaluation of the dynamic characteristics prolonged in time, in such a way that mediate the effect of various involved parameters. In any case, the approximation on the evaluation of the frequencies does not alter the effectiveness of the proposed method.

After the events, during the in site investigation, seven 44 mm diameter concrete micro-specimens were extracted from beams and columns and several ultrasonic tests were also carried out. The concrete strength was obtained with compression tests on core drilled horizontally at the centre of 10 columns. The mean cylindric strength was 14.02 MPa and agree well with the average wave velocity, $V=2700$ m/s. It has to be taken into account that there are some differences between the strength measured on core specimens and the actual in-situ strength. The outcomes of the compressive tests on the concrete specimens are reported in the following Table 1.

The relationship to convert the strength of a core specimen f_{core} into the equivalent in-situ value f_c is given in [15,16]:

$$f_c = (C_{H/D} C_{dia} C_a C_d) f_{core}$$

where $C_{H/D}$ is correction factor for height/diameter ratio H/D, C_{dia} is correction factor for diameter of core D, C_a is correction factor for the presence of reinforcing bars, and C_d is correction factor for damage due to drilling. The mean value of the cylindrical resistance (f_{cm}) and its standard deviation were evaluated to be 16.70 N/mm^2 and 3.07 N/mm^2 respectively. The elastic modulus of the concrete E_{cm} was determined starting from the average of the cylindrical resistance reappraised (f_{cm}) through the expression due to the EC2 [17], that furnish the mean value of the concrete elastic modulus after the shock on November 1, $E_{cm} = 9.5 \cdot \sqrt[3]{16.73} = 24300\ N/\text{mm}^2$.

About reinforcement, being it an industrial product and considering the operational difficulties to extract steel bars inside the damaged building, the mechanical properties have been defined referring to the original technical documentation. The destructive tests carried out at the time of the construction provided the yield stressf_y and the fracture stress f_t showed in Table 1, corresponding to a FeB44K Italian steel type.

3. Framework of the research

The availability of photo documentation before and after the November 1 event allows to follow-up the damage evolution in the most damaged column. According to the supposed type of damage, crushing of the compression zone on both faces of column with spalling of the concrete cover firstly occurred (after October 31 event). Subsequently crushing of the concrete core, accompanied by buckling of longitudinal bars and by stirrups failure (after November 1 event), have been observed. A three step procedure (Fig. 3 and Table 2) was set up to calibrate the overall mechanical characteristics of the initial undamaged model, which has been then used to trace the damage evolution due to the two seismic shakes.