Analysis of change in dynamic properties of a frame-resistant test building

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Abstract

This paper introduces and employs a damage detection methodology applied specifically to frame-resistant buildings designed according to the capacity criterion. The proposed technique assesses the effectiveness of the ductile resistant mechanism, following the occurrence of an earthquake. The method is based on the observation that, for ductile frames, the damage resulting from a strong earthquake should be limited to the plastic hinge zones, typically distributed throughout the structure, according to the weak-beam/strong-column principle. This damage distribution results in negligible changes in damping and global mode shapes, but large changes in frequencies and local response. From the operational point of view, the procedure makes use of a modal characterization of the building, utilizing well-established experimental techniques, focused on both the overall behaviour of the building as well as on the local behaviour of single joints, where major damage might be expected to occur. Damage evaluation is based on measured local changes in the mode shapes. Two sets of damage indexes are calculated: the first is related to the overall behaviour of the structure, while the other is related to the effectiveness of the local ductile mechanisms. In this paper, application of this methodology to a 60 per cent scale, 5-storey precast concrete test building (9.14 m by 9.14 m in plan, 11.43 m high) is presented. This building was subjected to dynamic testing before and after the application of pseudo-dynamic loads, simulating a 1.5 g PGA earthquake. The damage evaluation indicated a significant loss in stiffness of the whole structural system (of the order of 70%) and shed light on the effectiveness of the ductile resistant mechanisms. These results are consistent with the data recorded during the pseudo-dynamic test and the observed visual damage.

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1. Introduction

Most of the modern seismic design codes, such as [1,2], have acknowledged the principles of capacity design, and allow for the design of structures adopting high force reduction factors, up to 6 or more. In recent years, many buildings have been constructed in accordance with this criterion. Following a medium-strong earthquake, damage evaluation and estimation of remaining capacity should be conducted with special attention paid to these buildings, in view of the low yielding force assumed in the design and the necessity of assessing whether the structure actually responded to the seismic action according to the predicted design mechanism. At present, the evaluation of the damage is commonly based on visual inspection (as far as the structure is accessible) and on qualitative considerations. Vibrational tests potentially represent a convenient alternate evaluation tool, which allows for making quantitative judgments even when visual inspection is not possible.

This paper reports the outcomes of a Non-Destructive Evaluation (NDE) effort, based on vibrational tests, conducted on an experimental 5-storey precast concrete building, tested under high intensity, simulated seismic loading. Construction and the seismic testing of this building were carried out at the University of California, San Diego (UCSD), and are part of a more comprehensive Precast Seismic Structural System (PRESSS) research project [3–5]. This project has been developed throughout the last decade, with the goal of improving the understanding of seismic performance of precast structures, taking full advantage of the potential of this construction technology. Jointly sponsored by American

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institutions and companies, the programme has coordinated the efforts of over a dozen different research teams from across the United States, and included, as significant steps, (a) the numerical and (b) experimental study of different types of ‘ductile dry connections’ (where the inelastic deformation is concentrated at the beam–column interface), as well as (c) verification of effectiveness of the whole precast structural system utilizing these connections. The NDE experimentation on this reduced-scale model was executed during the final phase of this project.

Detailed information concerning the design criteria of the model and of the corresponding prototype can be found in [3], while in Refs. [4,5] the results of the seismic experiment are presented. In order to better understand the choices involved in the ND evaluation reported in this paper, the next Section briefly summarizes this information, concerning the model construction and seismic testing. Section 3 explains the motivation and gives details of the dynamic experiments carried out before and after the pseudodynamic test. In Section 4 we present and discuss the results of the evaluation. Finally, a brief summary is provided at the end of the paper.

2. The PRESSS 5-story test building

2.1. Building configuration

The experimental model (Fig. 1) is based on a 5-storey prototype building, with dimensions 30.50 m by 61.00 m in plan, with 3.81 m interstorey heights, and 7.62 m bay lengths in each direction. Specifically, it represents a 60 per cent reduced-scale two bay by two bay portion of the prototype, thus having 9.14 m by 9.14 m dimensions in the plan view (Fig. 2). The floor system used in the first three levels is pretopped double tees, and the top two levels consist of topped hollow-core slabs. Resistance to the lateral seismic loads is provided, in one direction, by two different parallel frames, and in the orthogonal direction by a shear wall.

As the NDE has been carried out by comparing the situations both before and after the seismic test in the frame direction only, special consideration will be given in the following to the description of the frame resistant mechanism and to the damage resulting from the seismic experiment:

The frame members were configured according to the capacity-based design philosophy. In the capacity-based design of structures, the location of distinct elements, referred to as plastic hinges, is selected and appropriately designed and detailed for energy dissipation under severe deformation [6]. Specifically, capacity-based design philosophy applied to a frame-resistant multistorey building requires that, under seismic action, plastic hinges form in the beams rather than in the columns (weak beam/strong column mechanism), and that the shear strength of the members exceeds the shear corresponding to the flexural strength.

The frame resistant structure is comprised of four different types of ductile connections: Tension–Compression Yielding (TCY) gap connection, TCY connection, Hybrid connection and Pre-tensioned connection. TCY connections and TCY gap connections are implemented on the top two floors and the bottom three floors of the frame. This is referred to as the TCY frame (Fig. 3(a)). The hybrid connection and pre-tensioned connection are used in one seismic frame, referred to as the Pre-tensioned Frame (Fig. 3(b)), at the lower three and upper two levels respectively.

The TCY [7] frame connection attempts to reproduce a traditional tension/compression yielding connection, similar to what is used in cast-in-place construction. However, rather than distributed yielding over a finite plastic hinge length, yielding is concentrated at the connection and the short sleeves of the rebar ensure that the beam reinforcement that provides moment strength and energy dissipation does not fracture prematurely at this concentrated yielding location.

In the TCY gap frame [7], the beams are erected between the columns leaving a small gap between the end of the beam and the face of the column, while only the bottom portion of this
gap is grouted with fibre-reinforced mortar in order to provide contact between the beam and column. Centre on this bottom grouted region, post-tensioning bars clamp the frame together. At the top of the beam, mild steel reinforcement is grouted into sleeves that extend along the length of the beam and through the column. The reinforcing steel is debonded for a specified length at the gap, so that it can yield alternately in tension and compression without fracture. Since the gap opens on one side of the column as it closes on the other side by an equal amount, the length of the frame does not change, even as the connection yields.

The hybrid connection, developed at the National Institute of Standards and Technology (NIST) [8], is an attempt to combine the advantages of using mild reinforcement (which allows for high hysteretic dissipation) with those of using pre-tensioned reinforcement (which guarantees low residual deformation). The beams are connected to multistorey columns by unbonded post-tensioning strands that run through the columns, while mild steel reinforcement is grouted in place in the ducts at the top and bottom of the beam through the column. It yields alternately in tension and compression and provides energy dissipation. The amount of mild steel reinforcement and post tensioning steel are balanced so that the frame re-centres itself after a major seismic event.

Unlike the other three connection systems, the pre-tensioned connection uses multiple beams and single-story columns and is intended to be used for construction where the most economical method consists of using single-storey columns with multi-span beams. Long, multi-span beams are cast in normal pre-tensioned casting beds, with specified lengths of the pre-tensioning strand left unbonded. These beams are then set on single-storey columns with the column reinforcing steel extending through ducts in the beam. As the frame displaces laterally, the debonded strand remains elastic. The system is characterized by low energy dissipation (relative to other systems), but the pre-tensioned reinforcement allows the structure to re-centre after a major seismic event [3].

2.2. Loading protocol and experimental response

The seismic experiment was carried out utilizing pseudodynamic techniques. This is a testing procedure which combines numerical computation, online control, and experimental measurement for the purpose of simulating the seismic response of the structure (see for instance [9]). The test is conducted in a quasi-static manner, utilizing numerical models for the calculation of inertia forces and damping, and measuring experimentally the restoring force at the actuators. The model was subjected to a sequence of pseudodynamic tests, alternated with cyclic testing under a force vector in the form of an inverted triangle, to a roof-level displacement equal to the peak displacement achieved during the previous pseudodynamic sequence. Four different seismic levels were chosen, corresponding to 33, 50, 100 and 150 per cent of the design level earthquake, defined according to the SEAOC recommendations [10] for zone 4, and representing a maximum spectral acceleration equal to $1.0\, g$, with 5% damping and subsoil condition $Sc$.

During the last load test, the structure experienced a peak displacement equal to 457 mm, corresponding to a 4.5% drift. With respect to the base moment/roof-level displacement response, it was noted that the peak moments sustained by the two frames were very similar (about 7000 kN m) but that the hysteretic responses of the frames were very different, with the non-prestressed frame exhibiting more energy dissipation and more residual displacement.

Each connection type experienced different types and levels of damage, as explained below. Only minor damage in the form of cover spalling, and some spalling and incipient breakdown of the fibre grouted pads between the beams and columns was present in the hybrid prestressed connections. Joint cracking was also minor, with maximum crack widths of about 0.12 mm. Very little cracking developed in the beam, except at the connection to the column. Similarly, excellent behaviour was exhibited by the pre-tensioned connections provided at the upper two levels of the prestressed frame. Damage in these
connections was also limited to superficial cover spalling at, or adjacent to, the beam–column interface.

TCY-gap connections showed significant spalling from the soffit of the beams immediately adjacent to the connection. There was also some crushing and deterioration of the fibergrouted pad at the base of the gap. In addition, it was observed during testing to this level that there was upwards sliding of the beams on the grouted pads during the maximum response level. As a consequence of the high tension strains resulting from the seismic response coupled with the dowel bending caused by the interface sliding, a few of the mild steel reinforcing bars crossing the interfaces fractured in the latter stage of testing. The condition of the upper TCY connections was good, despite the observation that sliding occurred at the interface. Some bond loss between the top-level reinforcing bars and the grouted ducts was noted at the exterior connections. Damage to the column bases was minimal, with minimal spalling being observed at some, but not all, of the seismic columns [4].

3. Dynamic ND evaluation

3.1. Overview and scopes of the dynamic characterization

The purpose of the ND experiment reported in this paper is to highlight the differences in dynamic response of the model during the intact and damaged situations, in terms of changes in modal parameters, and appearance of anomalies. This comparison aims to acquire additional information on the seismic performance of the innovative connections and of the structural concept developed in the PRESSS programme. In addition, it seeks to recognize those aspects of dynamic response that better characterize the presence of damage, in view of their employment in a generalized ND diagnostic methodology for the post-earthquake assessment of multi-storey buildings.

The idea of utilizing vibrational tests for the purpose of recognizing, quantifying and localizing the presence of structural damage, dates back to the early 1970s, and has undergone significant development in the last decade. This is confirmed by the recent increase in the scientific literature dealing with testing techniques, signal analysis, and damage identification methods, as summarized in many state of the art papers (see for example [11]). The concept underlying most of the proposed methods is the assumption that the occurrence of damage will change the structural properties (mass, stiffness or damping), which leads to further changes in dynamic characteristics. These dynamic characteristics should be interpreted in the broader sense; however, reference is typically made to the classical modal parameters: frequencies, mode shapes and damping ratios.

A crucial issue is in the practical application of these methods to damage assessment of full-scale civil engineering structures, such as bridges or multi-storey buildings. Even if these methods have been generally developed for the purpose of being utilized in practical problems, their experimental validation is provided by laboratory tests on models reproducing idealized structural systems, typically simply supported or cantilevered beams. Only a few recent experiments have allowed for a direct evaluation of the actual effects of damage on the dynamic characteristics of a large-scale structure in real conditions, through the comparison of dynamic responses at different levels of damage. For instance, in the 1980’s Farrar et al. [12] dynamically characterized the I-40 Bridge on Rio Grande near Albuquerque by introducing four different levels of damage at the middle span; more recently, De Roeck’s group [13] carried out a similar experimentation on the Z24-Bridge in Switzerland. In general, the direct verification of the performances of vibration-based assessment method on real-life structures has led to the following observations:

- frequencies are usually quite insensitive to damage, and changes in frequencies due to damage are often negligible with respect to the changes due to other causes (temperature variations, boundary conditions . . . );
- mode shapes are more sensitive to localized damage, if expressed in the form of modal curvature [14] for bending resistant systems, or in general in terms of strain-mode-shapes [15];
- changes in damping and occurrence of anomalies (such as nonlinearity) are often related to the occurrence of damage, but the correlation between the two phenomena is not straightforward, and few convincing techniques have been proposed for utilizing this information in a quantitative damage evaluation.

It should be noted that most of these observations were derived from studies on single-span, simply-supported bridges, where the damage is minor and highly localized. Conversely, relatively little attention has been given, until today, to the practical application of this methodology to problems of post-earthquake safety assessment of frame concrete buildings, where visual inspection and localized diagnostic tests still represent the only methods employed in the practice.

It should be noted that while a good number of experimental studies have been conducted on reduced-scale test models (e.g. [16,17]), little experimental data concerning real structures is available to allow for a direct comparison between before and after the earthquake situations (among these, one of the earliest and fundamental works is that of Iemura and Jennings [18], relating to the Millikan Library at Caltech). All of these works attempt to utilize the structural response acquired during the seismic event, thus presuming that at least two recording accelerometers are permanently installed on the building; no consideration was given to the possibility of acquiring information on the safety level of the structure from subsequent low-intensity vibrational tests.

In view of these concerns, the experiment presented here appears particularly significant for the development of a modal based ND testing methodology for post-earthquake assessment of civil engineering structures. In fact, the limited scaling level adopted in construction of the PRESSS building assures a mechanical behaviour similar to that of the full-scale prototype, and the damage induced by the pseudodynamic tests is definitely representative of that caused on a frame building by a severe seismic action. At the same time, testing the
building in a laboratory, rather than in the field, has actually allowed for better control and documentation of the damage induced, and for the execution of the experimental work in the most favourable conditions. In the following, the conducted dynamic experiment is reported, along with the results of the modal extraction and the synthetic damage evaluation. Because of page limitations, the employment of these results in the structural identification of a multi-degree-of-freedom model will be the subject of a separate publication.

3.2. Experimental program and transducer locations

Dynamic characterization of the test building was repeated before (phase 1) and after (phase 2) the execution of the pseudodynamic test sequence in the frame resistant direction, corresponding to the X-direction with respect to Fig. 2. Because execution of the vibrational tests required having all of the hydraulic actuators disconnected from the reaction wall, it was unfeasible to repeat the characterization at different stages of the pseudodynamic experiment.

In each testing phase, the building was instrumented in four different configurations, each comprised of fifteen recording accelerometers. The first configuration (A) aimed to detect the overall behaviour of the building, and included three accelerometers on every level, as shown in Figs. 2 and 3, thus assuming a rigid behaviour of the floors within their plan. Each additional configuration focused on the dynamic response of an individual beam–column connection, specifically: The hybrid connection at the second floor of the prestressed frame (configuration B); the TCY-gap connection at the second floor of the TCY frame (configuration C); and the TCY connection at the fourth floor of the same frame (configuration D). In these configurations, instruments were closely clustered along the beam and column, as shown in Fig. 4. Special attention was given to measurement of the relative displacements and rotations at the beam/column interfaces.

The data acquisition setup consisted of the fifteen piezoelectric accelerometers, hardwired to a central data acquisition unit provided with a signal conditioner and a 16-bit A/D converter. Operation of the converter was driven by a Windows-based workstation, which also stored the acquired signals and performed the basic preprocessing.

3.3. Vibrational measurement techniques

The experiment was carried out utilizing well-established vibrational measurement techniques. Stepped-Sine Tests (SST), Shock Tests (SHT) and Ambient Vibration Tests (AVT), were repeated in each phase for each instrument position, as further described. These methods for modal testing are today quite extensively used both in mechanical and civil engineering. Comprehensive references regarding the underlying theory include the textbooks of Inman [19] and Ewins [20].

It is worth remembering that the dynamic behaviour of a linear system can be represented in the form of Frequency Response Functions (FRFs), defined, for each pair of forcing and measuring positions, as the ratio, generally complex, between the amplitude of the response $x_j$ of the structure and the amplitude of the applied harmonic force $F_k$. Measurement of these functions is the principal goal of a dynamic characterization, as they represent the basis for the derivation of parameters that characterize the modal model of the structure.

Stepped Sine Tests (SST) represents the most straightforward approach for carrying out an experimental FRF, and consists of applying a harmonic steady-state force to the structure, at a number of fixed frequencies, recording the corresponding amplitude of the response at each step. In the testing practice, such a harmonic force is generated utilizing a shaker. In the PRESSS experiment, the large-size UCSD counter-rotating mass-shaker, shown in Fig. 5, was utilized.

This device was specifically designed and constructed for exciting large-scale civil engineering structures, such as
buildings and bridges, where large amplitude forces and low frequencies are required. The basic structure of this exciter consists of an enclosure containing two counter-rotating steel buckets, controlled by an electromechanical motor. Depending on the mass loaded in the buckets, the shaker is capable of generating forces up to 25 kN in a 0 to 8 Hz frequency range. During the experiment, the shaker was mounted on the southwest corner of the top floor of the PRESSS building (Fig. 2), in such a way as to apply the harmonic force in the X direction. This direction corresponds approximately to the direction of the acceleration measured by channel 1 in configuration A.

Shock Tests (SHTs) represent an alternate testing technique for achieving the same FRFs. The fundamental concept of this method lies in the observation that an infinitely short impulsive force contains a full range frequency content (i.e. the Fourier Transform is a flat function). Therefore, the FRF can be calculated as the ratio between the FFT of the response signal $X_j(\omega)$ and of the FFT of the forcing impulse $F_k(\omega)$. Since real impulses occur over a finite duration, the resulting frequency content may be approximated as flat only up to a certain cutoff frequency $f_c$. Assuming that the impulse consists of a semi-sinusoidal profile (which is true when non-linear phenomena do not occur in the shock mechanics) the cutoff frequency is related to the duration $\tau$ through the following expression:

$$f_c = \frac{3}{2\tau}.$$  \hspace{1cm} (1)

In turn, $\tau$ depends on the mechanical properties of the tested structure and impactor. When, as in this case, the flexibility of the tested structure is negligible, it is demonstrated that $\tau$ depends on the stiffness $k_H$ and the mass $m_H$ of the impactor only, according to

$$\tau = \frac{1}{4\pi} \sqrt{\frac{k_H}{m_H}}.$$  \hspace{1cm} (2)

In this experiment, a commercial pulse-hammer was utilized, consisting of a regular sledge hammer, instrumented with a piezoelectric load cell between the tip and the body, that is capable of measuring impact forces up to 4.5 kN. The mass of the hammerhead is 6 kg, while the stiffness is adjustable by exchanging the tip. According to Eq. (2), hard tips should be chosen when wide frequency content is desired, with limited energy content; vice versa, soft tips concentrate the energy of the impulse around low frequencies. Given the dimensions of the building, frequencies in a range much lower than 100 Hz were expected, and the optimal impulse profile, with a 150 Hz cut-off frequency, was obtained by gluing additional layers of foam to the softest tip available.

Since they are easy to carry out and provide reliable results in a relatively wide frequency range, shock tests were extensively applied in this experiment. The test procedure consists of a sequence of eight shocks: following each shock, an eight second long response record is acquired, and the correspondent FRF calculated; the resulting FRFs finally averaged in order to reduce noise. To ensure that each of the first three fundamental modes (X-direction, Y-direction, and torsional) were excited, two forcing positions were selected for each transducer configuration, corresponding to the measurement directions A3 and A1 (Fig. 2), the latter coinciding with the forcing direction of the shaker.

Theoretically, stepped-sine tests allow for higher accuracy measurement of an experimental FRF, making use of stationary forcing, according to the definition of the FRF. In addition, step-by-step control can improve the experimental FRF resolution around the resonance peaks, as needed. However, in the present case, the FRFs obtained by SST exhibited an amplitude-dependence, which revealed nonlinearity in the structural response. As an example, Fig. 6 represents the FRF measured at point A1, compared with the corresponding FRF achieved through shock tests, at a relatively low level of excitation. The difference between the two curves demonstrates the force amplitude-dependent behaviour of the structure. The softening nature of the nonlinearity is highlighted by a reduction of the resonance peak frequency; in detail, note that the first resonance shifts from 2.69 Hz to 2.47 Hz, while the second from 3.38 Hz to 3.21 Hz.

This amplitude-dependent effect is not evident in the SHT-based FRFs, and is clearly owed to the relatively high vibration amplitude produced by the shaker on the building around the resonant frequencies. For this reason, SST-based FRFs will not be considered in the remainder of this paper.

Fig. 5. View of the 25 kN force shaker.

Fig. 6. FRF achieved at point A1 through stepped-sine tests in the intact situations (single plots), compared with the corresponding FRF obtained through shock tests (continuous line).
Fig. 7 compares some of the FRFs achieved through shock tests, in the intact and damaged conditions. Specifically, reference is made to the signals recorded at the three top floor accelerometers, and to hammer shocks at positions A1 and A3. Representation is limited to the 0–10 Hz frequency range.

In summary, given the dimensions of the PRESSS building, the shock tests represented in this case a valuable and convenient alternative to using the shaker, which appeared in this case to be oversized. However, it is worth remarking that the test building is only 60 per cent scale, compared to the prototype. It is evident that in the problems of ND evaluation of full-scale large buildings, utilizing a large shaker will likely be the recourse of choice.

In addition to SST and SHT, Ambient Vibration Tests (AVT) were also carried out. Rather than the calculation of the modal parameters, these tests were basically conducted to determine, in a preliminary manner, the natural frequencies of the building, in order to better plan the subsequent forced tests. For these purposes, the acquired data was utilized in the calculation of the Power Spectral Density (PSD). Fig. 8 represents, for instance, a typical PSD, obtained by averaging 100 PSD each 8 s long. The presence of noise, unavoidable in a laboratory (as is likely in a real-scale building), and the fact that the input is unknown, only allow for obtaining qualitative information.

One cannot fail to mention how many recent studies have been carried out for the purpose of exploiting ambient vibration signals, which have obvious advantages over the execution of forced tests. However, it is the opinion of the authors that, when technically feasible, the extra effort required in the execution of forced vibration tests is largely offset by the higher quality of the records and by the clarity of the results. Moreover, when the input is unknown, there is no way to obtain certain salient dynamic characteristics (such as the mass-normalized mode-shape amplitude).
shows the outcome of the MAC cross calculation compares the resulting frequencies, obtained in between two sets of modes is the Modal Assurance Criterion which is the shear wall-braced direction. significant change was measured for the Y 3.38 Hz to 1.94 Hz respectively. Indeed, as expected, no high highly significant shifts, from 2.69 Hz to 1.38 Hz and from 3.38 Hz to 1.94 Hz respectively. shock tests. Based on a preliminary analysis of the FRFs, considerations. It is remarkable that the frequencies associated with the second and the third vibrational modes occurred, resulting from the damage. It should also be noted that, while for the first three mode shapes the correspondence is unequivocal, as shown by MAC values close to one, higher modes have MAC values, which are not negligible even for noncorresponding modes. This appears more evident as the frequencies increase. In a qualitative manner, as the modal frequencies increase, it is harder to identify a correspondence between modes in the before and after situations. This is consistent with the fact that higher modes are more sensitive to local damage mechanisms, which have less influence on the lower vibrational modes. Because they are more sensitive to localized damage, higher modes are hard to recognize experimentally and predict analytically, therefore they may be impractical for the problem of localization. For this reason, in the following we will focus our attention on the three lower mode shapes.

Considering the amplitude-dependent response obtained using the shaker, it was decided to extract the modal parameters using the high quality FRFs obtained from the shock tests. Based on a preliminary analysis of the FRFs, curve fitting was limited to the 0–15 Hz frequency range, thus identifying within this range five natural frequencies. The range restriction is mainly related to the scarce interest in the higher frequencies for recognizing global instability problems in the structure. Frequencies, mode shapes and damping ratios were extracted by fitting the experimental FRFs curves with the theoretical expression for Multi-Degree-of-Freedom (MDOF) linear, viscously damped systems, using a classical Multi-Input-Multi-Output (MIMO) optimization technique. Again, the reader is referred to the specific literature [19,20] for the details of these extraction methods.

Table 1 compares the resulting frequencies, obtained in phase 1 and phase 2; frequencies are correlated on the basis of a modal recognition, which makes use of qualitative observations. It is remarkable that the frequencies associated with the X-displacement and torsional modes, experienced highly significant shifts, from 2.69 Hz to 1.38 Hz and from 3.38 Hz to 1.94 Hz respectively. Indeed, as expected, no significant change was measured for the Y-direction mode, which is the shear wall-braced direction.

A quantitative criterion for defining the correspondence between two sets of modes is the Modal Assurance Criterion (MAC), first introduced by Allemang and Brown [21] and defined as

\[
\text{MAC} \left( \psi_r^I, \psi_s^II \right) = \frac{|\left( \psi_r^I \psi_s^II \right)^T |^2}{(\psi_r^I \psi_r^I) (\psi_s^II \psi_s^II)}
\]  

where \( \psi_r^I \) and \( \psi_s^II \) are the \( r \)-th and \( s \)-th mode shapes measured in the intact and damaged situations. From Eq. (3), a MAC value close to 1 indicates a correspondence between two modes.

Table 2 shows the outcome of the MAC cross calculation for the before and after situations, utilizing the results of the modal extraction relative to configuration A. It is apparent that an exchange in the order of the frequencies associated with the first mode, the lateral flexibility of the frames is considerably higher than the deformability of a floor in-plane. The deformation of a floor in its plane, or the torsion of a frame beam requires higher deformation energy, that is associated with a higher mode shapes.

As each floor is assumed rigid in its plane, its motion As each floor is assumed rigid in its plane, its motion

\[
\begin{array}{cccccc}
\text{Before} & \text{After} \\
\hline
2.69 & 3.13 & 3.38 & 8.75 & 11.20 & 13.10 \\
1.38 & 1.38 & 1.94 & 5.31 & 7.38 & 13.10 \\
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Table 1
Natural frequencies and modal recognition

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<tr>
<th>Frequency (Hz)</th>
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<td>Before</td>
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4. Results

4.1. Modal extraction and mode recognition

Frequencies, mode shapes and damping ratios were extracted by fitting the experimental FRFs curves with the theoretical expression for Multi-Degree-of-Freedom (MDOF) linear, viscously damped systems, using a classical Multi-Input-Multi-Output (MIMO) optimization technique. Again, the reader is referred to the specific literature [19,20] for the details of these extraction methods.

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\]  

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Table 2 shows the outcome of the MAC cross calculation for the before and after situations, utilizing the results of the modal extraction relative to configuration A. It is apparent that an exchange in the order of the frequencies associated with the second and the third vibrational modes occurred, resulting from the damage. It should also be noted that, while for the first three mode shapes the correspondence is unequivocal, as shown by MAC values close to one, higher modes have MAC values, which are not negligible even for noncorresponding modes. This appears more evident as the frequencies increase. In a qualitative manner, as the modal frequencies increase, it is harder to identify a correspondence between modes in the before and after situations. This is consistent with the fact that higher modes are more sensitive to local damage mechanisms, which have less influence on the lower vibrational modes. Because they are more sensitive to localized damage, higher modes are hard to recognize experimentally and predict analytically, therefore they may be impractical for the problem of localization. For this reason, in the following we will focus our attention on the three lower mode shapes.

In this case, it appears appropriate to describe the behaviour of the building with a simplified model, assuming that: (i) each floor can be considered rigid in-plane; (ii) axial flexibility of frame columns is negligible; (iii) resistant mechanism of frames and wall act in-plane. In general, these hypotheses are sufficiently accurate for the first three mode shapes, which are characterized by low deformation energy. For example, for the first mode, the lateral flexibility of the frames is considerably higher than the deformability of a floor in-plane. The deformation of a floor in its plane, or the torsion of a frame beam requires higher deformation energy, that is associated with a higher mode shapes.

As each floor is assumed rigid in its plane, its motion can be modelled by three degrees of freedom (DOF): x displacement, y displacement, and rotation \( \theta_z \) with respect to the geometrical centre. Therefore a 15-DOF model is sufficient for describing the overall behaviour of the 5-storey building. Based on this geometrical model, the experimental

\[
\begin{array}{cccccc}
\text{After} & 1.38 & 3.13 & 1.94 & 5.31 & 7.38 & 13.1 \\
2.69 & 0.98 & 0.10 & 0.20 & 0.14 & 0.08 & 0.24 \\
3.13 & 0.00 & 0.48 & 0.98 & 0.23 & 0.14 & 0.31 \\
3.38 & 0.02 & 0.99 & 0.30 & 0.17 & 0.31 & 0.11 \\
8.75 & 0.06 & 0.01 & 0.21 & 0.67 & 0.50 & 0.49 \\
11.2 & 0.35 & 0.30 & 0.22 & 0.38 & 0.61 & 0.34 \\
13.1 & 0.12 & 0.28 & 0.01 & 0.29 & 0.53 & 0.87 \\
\hline
\end{array}
\]
Table 3
Modal components of first three mode shapes

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<tr>
<th>I mode shape</th>
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<th>A1 (kg/√s)</th>
<th>A2 (kg/√s)</th>
<th>A3 (kg/√s)</th>
<th>By coordinate x (kg/√s)</th>
<th>y (kg/√s)</th>
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<th>θz (rad/√s)</th>
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<table>
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<th>Story</th>
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<th>y (kg/√s)</th>
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<td>6.29E-04</td>
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</table>

modal components obtained for the first three mode shapes are reported numerically in Table 3, and graphically in Fig. 9, where the before and after conditions are compared. Also, Fig. 10 shows a 3-dimensional representation of the same three modes in the undamaged condition.

4.2. Global structure behaviour

It has already been mentioned how most of the recent publications on damage identification deals with single- or multi-span structures, such as bridges, where the damage introduced (experimentally or numerically) is limited and localized. As for these types of problems and structural configuration, there is a general agreement on the fact that frequencies are usually a parameter insensitive to damage. However, we cannot look at these findings as a general rule. Indeed, the PRESSS experiment highlights that, in the case of a frame-resistant structure subject to seismic action, the variations in frequency can be very significant, regardless of the actual visual impact of the damage produced by the earthquake. In fact, the test building experienced a significant loss in stiffness which resulted in changes on the order of 55% for the first frequency, and 45% for the torsional, which represent the modes primarily involved in the frame direction pseudo-dynamic tests.

More quantitative information on the mass and stiffness characteristics of the building can be derived from the analysis of the lower mode shapes. To this aim, one can theoretically use a refined Finite Element Model. In this paper, in order to clarify the essence of the general procedure, we prefer to keep the model as simple as possible, assuming equal distribution of mass and stiffness at all floor levels. This apparently sharp simplification of the model is in fact perfectly consistent with the linearity of the mode shapes observed both before and after the damage episode, and does not seriously affect the accuracy of the results.
Fig. 9. Graphical representation of first three mode shapes in the intact (light line) and damaged (bold line) situations.

Fig. 10. 3-dimensional representation of first three mode shapes in the intact situation.
Fig. 9 shows how the first mode shape, which is principally a lateral displacement mode in direction $X$, also contains a significant torsional component; in the same way, the torsional component also contains a lateral deformation component. This asymmetry is due to the difference between the mass centre and the stiffness centre. An eccentricity of the centre of stiffness, with respect to the geometrical centre of the floor, can be attributed to a difference in the stiffness between the two frames. Additionally, as a consequence of the presence of the actuators on the west side of the floor during the tests, an eccentricity in the mass distribution was expected. The actual mass distribution can be calculated by remembering that for each vibrational mode, the following equation is valid:

$$\psi^T \mathbf{M} \psi = 1 \quad (4)$$

where $\mathbf{M}$ is the mass matrix, and $\psi$, the $r$-th mass-normalized mode shape. As a first approximation, we can assume that: (i) each floor has the same mass $m$ and moment of inertia $I_G$, calculated with respect to the centre of mass; (ii) distribution of the stiffness of each floor is such as to guarantee, for each mode shape, the same instantaneous rotational centre $C_r$. With these assumptions, Eq. (4) simplifies to:

$$I_G \sum_{k=1}^{n} \theta_k^2 = 1 \quad (5)$$

where $\theta_k$ is now the rotational component of the $r$-th mode, measured at the $k$-th story, and $I_G$ is the moment of inertia with respect to the instantaneous rotational centre. When, as for the second mode, the rotational component is negligible (i.e. the centre of rotation tends to infinity), it is more accurate to utilize the following:

$$m \sum_{k=1}^{n} \left( x_k^2 + y_k^2 \right) = 1 \quad (6)$$

where $x_k$ and $y_k$ are the translational components of the $r$-th mode, measured at the $k$-th story. Eq. (5) applied to the first and third modes, and Eq. (6) applied to the second one, yield an estimation of the equivalent values of mass, moment of inertia, and mass eccentricity of a building floor of $m = 58331$ kg, $I_G = 992000$ kg m$^2$ and $e_G = 0.78$ m.

In order to gain information on the stiffness distribution, the relation

$$\psi^T \mathbf{K} \psi = \omega_r^2 \quad (7)$$

can be utilized, where $\mathbf{K}$ is the stiffness matrix of the building. Let us define the stiffness of the shear wall, $k_W$, of the Tension Compression Yielding (TCY) frame $k_{TCY}$ and of the PreTensioned (PT) frame $k_{PT}$ as:

$$k_W = \frac{\psi^T \mathbf{W} \mathbf{K} \psi}{\psi^T \mathbf{W} \psi}, \quad k_{TCY} = \frac{\psi^T \mathbf{TCY} \mathbf{K} \psi_{TCY}}{\psi^T \mathbf{TCY} \psi_{TCY}}, \quad k_{PT} = \frac{\psi^T \mathbf{PT} \mathbf{K} \psi_{PT}}{\psi^T \mathbf{PT} \psi_{PT}} \quad (8)$$

where $\psi$, $\psi_{W}$, $\psi_{TCY}$ and $\psi_{PT}$ is a vector collecting the components of mode $r$ relevant to the in-plane wall displacement, and similarly for $\psi_{W}$, $\psi_{TCY}$ and $\psi_{PT}$. Eq. (8) implicitly assumes that the three stiffness values are not dependent on the vibrational modes. In fact, these stiffnesses can be seen as the ratio between the maximum value of an inverted triangular distribution of forces, each concentrated at one floor level, and the maximum displacement of the structure, when the resulting displacement of the building has an inverted triangular shape, as well. In this case, Eq. (7) can be written as

$$\left( \psi^T \mathbf{W} \psi \right) k_W + \left( \psi^T \mathbf{TCY} \psi_{TCY} \right) k_{TCY} + \left( \psi^T \mathbf{PT} \psi_{PT} \right) k_{PT} = \omega_r^2. \quad (9)$$

The set of three equations written for the first three vibrational modes, represents a linear system where the unknown quantities are the three stiffness $k_W$, $k_{TCY}$, $k_{PT}$.

Table 4 reports the results achieved by solving this system, with reference to the situations before and after damage. It can be observed that:

- even in the undamaged situation, the two frames exhibit different stiffness values;
- both of the frame systems, TCY and PT, experienced a significant loss in stiffness; however, the TCY frame appears heavily damaged with respect to the PT counterpart, indicating a loss in stiffness of 77% for the former and 61% for the latter;
- the shear wall did not experience any reduction in stiffness. Rather, it apparently exhibits an increase. This change, which is relatively small respect the changes observed for the frames, can be attributed to the resolution of the modal extraction process. This is an expected results, as the pseudodynamic loads were only applied in the frame resistant direction.

The significant changes in stiffness unequivocally denote the occurrence of severe damage. However, two other aspects of the global response of the structure apparently contradict this observation, and deserve to be further discussed.

First, the lower three mode conserved their shapes after being damaged. This is qualitatively evident from Fig. 9, as well as quantitatively from the MAC values reported in Table 2, that resulted in values very close to one for corresponding modes. This is a direct consequence of the capacity-based philosophy adopted in the design of the PRESSS building: during a strong earthquake, the weak-beam/strong-column mechanism allows for the simultaneous formation of plastic hinges at each storey-level. Since the damage is evenly distributed along the height,
the resulting plastic mechanism is, from a global point of view, very close to the deflected shape that appears in the initial elastic modes. This suggests an alternate interpretation of the capacity design criterion: the dissipating elements should be configured in order to allow for the mode primarily involved in the seismic response to maintain its shape in the inelastic-phase. In other words, the conservation of the shape appears to be the main discriminating factor in assessing the effectiveness of the choices made in the seismic design process. This requirement may be employed as a general rule in the post-earthquake assessment of buildings, and in the case of the PRESSS prototype has been fulfilled.

Second, only a very slight increase in damping was observed following the onset of damage. For the first three modes, the damping ratios in the before and after situations are \( \xi^I = 0.007 \) and \( \xi^{II} = 0.008 \), respectively, for the first three modes; moreover, these changes in damping are basically independent of the mode shape. This is apparently in contradiction with many experiences \([22–25]\) conducted in the past on RC or PRC structures, which have shown how damping is a very sensitive parameter to damage (damping variations on the order of 100% can be easily measured, following the opening of a simple crack). In the case of cracked RC or PRC structures, friction mechanisms will occur: (i) at the concrete interface, when the crack is subjected to the opening and closing actions, and (ii) at the bond area between the concrete of the tension zone and the tension reinforcement. In the case of the PRESSSS building, cracking is limited, and restricted to the beams’ edges. This is again a consequence of the capacity based design, in which dissipation is located at the plastic hinges, and members outside the plastic hinges are intended to remain elastic during the seismic event. Inside the plastic hinges zones, the presence of gaps and the reinforcement debonding preclude friction damping mechanisms when the amplitude of vibration is low.

**4.3. Connections behaviour**

Further information on the nature of the inflicted damage can be obtained by analyzing in detail the dynamic behaviour of single connections. A modal extraction was carried out starting with the FRFs achieved through shock tests, for configurations B, C, and D. Fig. 11 shows the first mode shape in the intact and damaged situations for the three connections investigated, namely the hybrid (a), the TCY gap (b) and the TCY (c) connections.

Direct comparison allows some immediate observations:

- despite the global mode shapes remaining basically unchanged, significant differences are apparent at the local level;
- deformation at the gap is characterized by a relative rotation between the column and the beam edge, as expected;
- an unwanted sliding effect is also present;
- even in the undamaged situation, the behaviour of the connections is in some cases not symmetrical, and this asymmetry is more evident in the damaged state.

\[
\begin{align*}
\Delta \phi_L &= \phi_L - \phi_C \\
\Delta \phi_R &= \phi_C - \phi_R.
\end{align*}
\]

Table 5 summarizes the results of parameter calculations for the three connections investigated (hybrid, TCY gap and TCY), in the undamaged and damaged situations.

The significant aspects of this behaviour can be quantitatively described using the following strain-mode-shape features: the left and right beam edge slide \( \Delta z_L \) and \( \Delta z_R \); the left and right beam edge rotation \( \phi_L \) and \( \phi_R \); the column rotation \( \phi_C \); the opening of the left and right gaps \( \Delta \phi_L \) and \( \Delta \phi_R \), calculated as:

The hybrid prestressed connection suffered significant stiffness deterioration, the most severe among the three connection types, following the simulated earthquake. The main evidence of this fact is that the modal rotation of the beam edges, \( g \phi_L \) and \( g \phi_R \), is nearly zero in the damaged situation: implying that the beams remain almost horizontal during the vibrational motion. The change in the gap between the intact and damaged situations is also significant: variations of \( \Delta \phi_L \) and \( \Delta \phi_R \) have been measured as of the order of 125% and 97%, respectively. This stiffness deterioration is not unexpected, and reflects the behaviour experimentally evaluated in the preliminary tests, as reported in [8]. Unlike the other connection models, the sliding effect here is very slight in both before
Table 5
Changes in the behaviour of the joints according to the first mode shape in the intact and damaged situations. ∆z, and ∆R: left and right gaps sliding; φ, and φR: left and right beams edge rotation; φC column rotation; ΔφL and ΔφR: left and right gaps rotation

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<tr>
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<th>C — TCY gap joint</th>
<th>D — TCY joint</th>
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</table>

and after situations, and still consistent with the behaviour predicted. Also, there appears to be no correlation between slide amplitude and damage.

The TCY-gap connection monitored presents an unexpected asymmetrical vibrational behaviour. As clearly shown in Fig. 11, there is a significant upward modal slide ∆zL of the left beam edge, while the right side does not seem to show this phenomenon. Curiously, this asymmetrical behaviour, seen in both intact and damaged situations, is more pronounced in the intact state. Evidently, the phenomenon is not strictly related to damage. A possible explanation of this sliding effect can be found in the specific construction details of the joint. In fact, for this type of connection the shear stiffness is basically related to the effectiveness of the fibre-grouted mortar pad which fills the base of the gap. As reported in [4], during the pseudodynamic experiment in many cases we saw an unwanted slide of the beam edge, due to an insufficient clamping force in the post-tensioning of the connection. It is interesting to observe that on the right, where the fiber-grouted pad probably worked correctly, a significant decrease in the right beam edge rotation φR was measured. As predicted in [7], this is a consequence of the expected degradation of the TCY mechanism of the mild reinforcement.

Finally, the TCY connection is characterized by behaviour that changes completely between the intact and the damaged situations. In the intact state, low sliding and negligible gap opening were measured, as expected. Following damage, the joint exhibits a severe loss of stiffness that can be recognized in the opening of the joint gaps ΔφL and ΔφR, in high sliding values ∆zL and ∆zR, and a change in the column rotation φC of 93%. All these changes are obvious symptoms of severe degradation of the joint, in accordance with the design concept [7].

5. Conclusions

Most of the damage recognition methods proposed in the literature refer to single, localized, light damage occurring at undesirable locations. This results in negligible changes in frequency, sensible changes in the strain mode shape, and important changes in damping. Instead, for structures designed according to capacity based philosophy, the damage resulting from a strong earthquake is heavy, and globally distributed through the structure, in regions specifically designed to dissipate energy (plastic hinges). In this case, large changes in frequency, and negligible changes in mode shapes and damping should be expected. This observation suggests the adoption of a methodology whereby the problem of post-earthquake damage assessment, applied to a frame resistant building, requires the experimental measurement and analysis of both global mode shapes and local strain mode shapes.

In this paper, the application of this methodology to a case study has been presented. The modal extraction and damage evaluation showed a significant loss in stiffness of the whole structural system (on the order of 70%), and shed light on the effectiveness of the resistant mechanism. In fact:

- from a global standpoint, the building essentially conserved the same mode shapes, as quantitatively shown by the cross calculation of the MAC values;
- local changes in stiffness appeared to be concentrated at the designed plastic hinges, where changes in relative rotation on the order of 100% were measured;
- based on changes in modal damping, no unwanted dissipative mechanism occurred outside the plastic hinges zones.

These results are consistent with the data recorded during the pseudodynamic test and the observed visual damage.

Acknowledgements

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References


