Dynamic characterization of a full-scale 5-story R/C seismically damaged building specimen

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ABSTRACT

This paper discusses the system identification of a full-scale 5-story reinforced concrete building tested on the unidirectional NHERI-UCSD shake table. The purpose of the test program was to study the seismic response of the structure and nonstructural components and systems (NCSs) and their dynamic interaction at different levels of seismic excitation. The building specimen was tested under base-isolated and fixed-base configurations. In the fixed-based configuration, the building was subjected to a sequence of earthquake motion tests designed to progressively damage the structure and NCSs. Ambient vibration (AV) data was recorded continuously in the test structure for a period of about 15 days. Additionally, low-amplitude white noise (WN) base excitation tests were conducted at key stages during the test protocol. Using the structural vibration data recorded during the seismic tests by 24 accelerometers, the deterministic-stochastic subspace identification method (DSI) is employed to estimate the variations of the modal properties of the building by employing a short-time windowing approach. In addition, modal properties of the fixed-base structure at different levels of structural and nonstructural damage with the AV and WN test data assuming a quasi-linear response of the system are identified using three output-only (SSI-DATA, NExT-ERA and EFDD) and two input-output (OKID-ERA and DSI) methods. The results form AV and WN test data show that modal properties obtained by different methods are in good agreement and that the effect of structural/nonstructural damage progression is clearly evidenced via changes induced on the estimated modal parameters of the building.

Keywords: Building, Modal properties, Seismic damage, Shake table, System Identification.

INTRODUCTION

System identification is an active field of research aiming to characterize the dynamic properties of large and complex civil structures by using input-output or output-only vibration data recorded by sensors installed in the structure of interest. In particular, the results obtained from system identification have been used for vibration-based damage identification purposes (e.g., [1]). The objective is to identify damage in the structure by analyzing the variation in the estimated dynamic characteristics from an initial (reference) state to a state after the structure has been subjected to potentially damage-inducing loading. The dynamic characterization of civil structures usually comprises the estimation of natural frequencies, damping ratios, and mode shapes from the recorded data.

Vibration data recorded during earthquakes events on civil structures that have suffered damage is extremely scarce and large-scale shake tables have provided important data from structures tested at different states of damage and using different sources of dynamic excitation, including ambient vibrations (AV), white-noise (WN) base excitations, and seismic excitations (e.g., [2-4]). Most of previous studies have focused on the identification of the modal properties of structures using low-amplitude vibration data and a just a few researches have investigated the variations of the modal properties during strong motion earthquake excitations (e.g., [5,6]). In this paper, the modal properties of a full-scale 5-story reinforced concrete (RC) building specimen, referred to as BNCS building hereafter, tested on the large-scale UC San Diego shake table [7-9] are identified using the acceleration response data recorded during seismic tests. The time-variant modal properties of the BNCS building are identified by using the deterministic-stochastic subspace identification method (DSI) applied to a moving short-time window of input-output acceleration data [5,6,10,11]. These modal properties are compared to those previously identified before and after each seismic test from AV and WN base excitation test data.

BUILDING DESCRIPTION

The test specimen was a full-scale five-story RC structure fully furnished with nonstructural components and contents (Fig. 1a). The height of the building was 21.34 m, distributed in five stories of 4.27 m high (Fig. 1b). The building had plan dimensions of 6.6 × 11.0 m in the transverse and longitudinal directions (Fig. 1c), respectively, the latter coinciding with the direction of the movement of the uni-directional shaking table. In the longitudinal direction, the lateral resisting system of the building consisted of a pair of one-bay special moment resisting frames. The main structural components of the building included 0.30 × 0.71 m beams, 0.66 × 0.46 m columns, 0.20 m thick RC floor slabs, and two 0.15 m thick RC shear walls to accommodate an operating elevator. More information about the specimen can be found in [7] and [8].

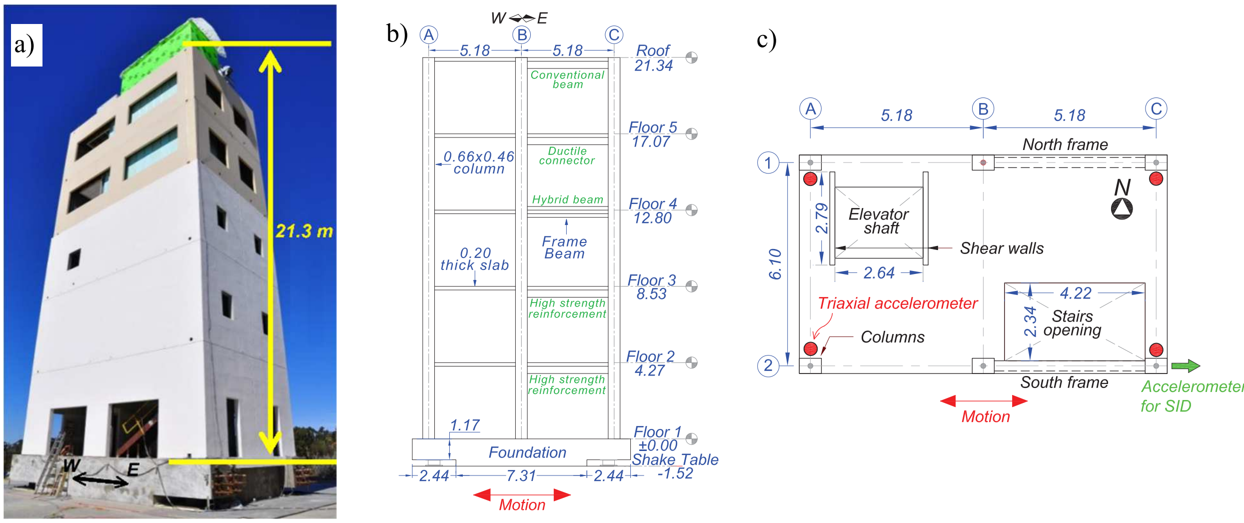

Figure 1. Instrumentation of a bridge: (a) elevation view, (b) section of the deck.

INSTRUMENTATION AND TEST PROTOCOL

More than 550 sensors, including displacement transducers, strain gauges, and accelerometers, were installed in the structural and nonstructural components of the building. In particular, four triaxial accelerometers were installed on the corners of each floor slab (see Fig. 1c) and two triaxial accelerometers were installed on the shake table platen. The accelerometers were Episensor with a frequency bandwidth DC–200 Hz, dynamic range of 155dB, and full-scale range ±4g. A band-pass infinite impulse response Butterworth filter of order 4 with cut-off frequencies at 0.04 and 25.0 Hz was used to filter the recorded raw acceleration data.

A suite of six seismic records was applied to the BNCS building with the purpose of progressively increase the seismic demand of the structure. Spectrally-matched and actual motions were considered in the seismic test phase. Table 1 shows the strong motions employed and their corresponding peak input acceleration (PIA), peak input velocity (PIV) and peak input displacement (PID) achieved on the shake table platen. The acceleration time-histories and their acceleration and displacement elastic response spectra for a damping ratio of 5% are shown in Fig. 2. In addition, Table 1 shows the ambient vibration (AV) test data and low-amplitude white-noise (WN) base excitation tests conducted. It is noted that for the WN tests, the nominal amplitude is shown as %g root-mean-square (RMS).

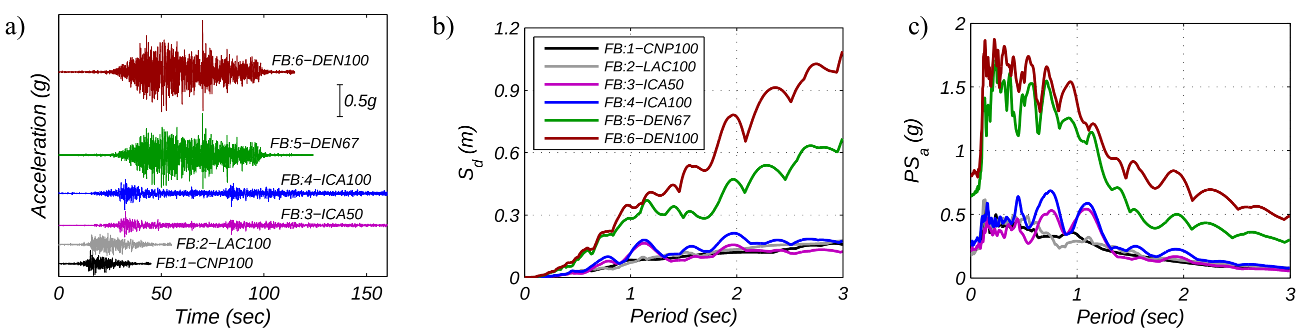

Figure 2. Input ground motions achieved on the shake table: (a) Acceleration time-histories; (b) Elastic displacement response spectra (ξ=5%); (c) Acceleration response spectra (ξ=5%).

Table 1. Description and nomenclature of tests and data applied to the BI-BNCS building.

Date	Description	Name	PIA (g)	PIV (mm/s)	PID (mm)	Damage state
May 7, 2012	AV1	AMB1	-	-	-	DS0
	6 min WN – 1.5%g RMS	WN1A	-	-	-	DS0
	6 min WN – 3.0%g RMS	WN1B	-	-	-	DS0
	6 min WN – 3.5%g RMS	WN1C	-	-	-	DS0
	Canoga Park (1994 Northridge EQ.)	**FB1-CNP100**	0.21	235.0	87.8	-
	AV2	AMB2	-	-	-	DS1
May 9, 2012	**LA City Terrace (1994 Northridge EQ.)**	**FB2-LAC100**	0.18	230.5	93.1	-
	AV3	AMB3	-	-	-	DS2
	ICA 50% (2007 Pisco EQ.)	**FB3-ICA50**	0.21	262.2	58.3	-
May 11, 2012	AV4	AMB4	-	-	-	DS3
	ICA 100% (2007 Pisco EQ.)	**FB4-ICA100**	0.26	284.9	73.2	-
	6 min WN – 1.5%g RMS	WN2A	-	-	-	DS4
	AV5	AMB5	-	-	-	DS4
May 15, 2012	4 min WN – 3.0%g RMS	WN2B	-	-	-	DS4
	TAPS Pump St. 67% (2002 Denali EQ.)	**FB5-DEN67**	0.64	637.4	200.6	-
	6 min WN - 1.5%g RMS	WN3A	-	-	-	DS5
	AV6	AMB6	-	-	-	DS5
	4 min WN – 3.5%g RMS	WN3B	-	-	-	DS5
	TAPS Pump St. 100% (2002 Denali EQ.)	**FB6-DEN100**	0.80	835.7	336.2	-
	AV7	AMB7	-	-	-	DS6

SYSTEM IDENTIFICATION

The modal properties of the BNCS building are identified using the DSI method with the recorded input-output acceleration data. A short-time windowing approach is employed in order to track the variation of the dynamic characteristics of the building during the seismic tests.

DSI method

The following equations describe a discrete-time linear time-invariant (LTI) state-space model [13]:

$$\mathbf{x}_{k+1} = \mathbf{A_d}\mathbf{x}_k + \mathbf{B_d}\mathbf{u}_k + \mathbf{w}_k , \tag{1a}$$

$$\mathbf{y}_k = \mathbf{C_d}\mathbf{x}_k + \mathbf{D_d}\mathbf{u}_k + \mathbf{v}_k , \tag{1b}$$

with

$$E\left[\begin{pmatrix}\mathbf{w}_p \\ \mathbf{v}_p\end{pmatrix}\begin{pmatrix}\mathbf{w}_q^T & \mathbf{v}_q^T\end{pmatrix}\right] = \begin{pmatrix}\mathbf{Q} & \mathbf{S} \\ \mathbf{S}^T & \mathbf{R}\end{pmatrix}\delta_{pq} \geq 0 , \tag{2}$$

where $\mathbf{x}_k, \mathbf{y}_k, \mathbf{u}_k$ = state, output, and input vectors, respectively; $\mathbf{A_d}, \mathbf{B_d}, \mathbf{C_d}, \mathbf{D_d}$ = state, input, output, and direct feed-through matrices, respectively; $\mathbf{w}_k, \mathbf{v}_k$ are the process and measurement noises with associated covariance matrices $\mathbf{Q}, \mathbf{R}$ and $\mathbf{S}$; δ_{pq} = Kronecker delta; and k = discrete time step. To determine the modal frequencies (f_r), modal damping ratios (ξ_r), and mode shapes ($\boldsymbol{\phi}_r$) of the dynamic system, it is required to estimate the state and output matrices ($\mathbf{A_d}$ and $\mathbf{C_d}$) given s samples the input and output data ($\mathbf{u}_j$ and $\mathbf{y}_j$ with $j = 0,\ldots,s-1$), because they are related trough the following expressions:

$$f_r = \frac{\sqrt{\lambda_r\,\lambda_r^*}}{2\pi} , \tag{3}$$

$$\xi_r = \frac{-\mathrm{Re}(\lambda_r)}{|\lambda_r|} , \tag{4}$$

$$\boldsymbol{\Phi} = \mathbf{C_d}\,\boldsymbol{\Psi} = \left[\boldsymbol{\phi}_1,\ldots,\boldsymbol{\phi}_r\right] , \tag{5}$$

where are the eigenvalues of the continuous-time state matrix $\mathbf{A}_c$ (with $\mathbf{A_d} = e^{\mathbf{A_c}\Delta t}$ and Δt = sampling time), $\mathbf{\Psi}$ are the eigenvectors of $\mathbf{A_d}$, and * and $|\cdot|$ denote complex conjugate and magnitude, respectively. In this article the DSI method is used to identify the modal properties of the BNCS building. Fig. 3 presents a flowchart of the DSI method [12].

1. Define Hankel matrices:

$$\mathbf{U}_{0|2i-1} = \begin{pmatrix} \mathbf{U}_{0|i-1} \\ \mathbf{U}_{i|2i-1} \end{pmatrix} = \begin{pmatrix} \mathbf{U}_p \\ \mathbf{U}_f \end{pmatrix} \quad ; \quad \mathbf{Y}_{0|2i-1} = \begin{pmatrix} \mathbf{Y}_{0|i-1} \\ \mathbf{Y}_{i|2i-1} \end{pmatrix} = \begin{pmatrix} \mathbf{Y}_p \\ \mathbf{Y}_f \end{pmatrix}$$

2. Compute oblique projections:

$$\mathbf{O}_i = \mathbf{Y}_f /_{\mathbf{U}_f} \begin{pmatrix} \mathbf{U}_p \\ \mathbf{Y}_p \end{pmatrix} \quad ; \quad \mathbf{O}_{i+1} = \mathbf{Y}_f^- /_{\mathbf{U}_f^-} \begin{pmatrix} \mathbf{U}_p^+ \\ \mathbf{Y}_p^+ \end{pmatrix}$$

3. Determine order of the system using singular value decomposition:

$$\mathbf{W}_1 \mathbf{O}_i \mathbf{W}_2 = \begin{pmatrix} \mathbf{U}_1 & \mathbf{U}_2 \end{pmatrix} \begin{pmatrix} \mathbf{S}_1 & \mathbf{0} \\ \mathbf{0} & \mathbf{0} \end{pmatrix} \begin{pmatrix} \mathbf{V}_1^T \\ \mathbf{V}_2^T \end{pmatrix} = \mathbf{U}_1 \mathbf{S}_1 \mathbf{V}_1^T$$

$\boldsymbol{W}_1, \boldsymbol{W}_2$: weighting matrices

4. Compute the extended observability matrices $\mathbf{\Gamma}_i$ and $\mathbf{\Gamma}_{i-1}$:

$$\mathbf{\Gamma}_i = \mathbf{W}_1^{-1} \mathbf{U}_1 \mathbf{S}_1^{1/2}$$

$\mathbf{\Gamma}_{i-1}$ is defined by removing the last l rows of $\mathbf{\Gamma}_i$

5. Determine the state sequences $\tilde{\mathbf{X}}_i$ and $\tilde{\mathbf{X}}_{i+1}$:

$$\tilde{\mathbf{X}}_i = \mathbf{\Gamma}_i^{\dagger} \mathbf{O}_i \quad ; \quad \tilde{\mathbf{X}}_{i+1} = \mathbf{\Gamma}_{i-1}^{\dagger} \mathbf{O}_{i+1}$$

where $\bullet^{\dagger}$ denotes the Moore-Penrose pseudo-inverse of the matrix $\bullet$

6. Solve the set of linear equations for $\mathbf{A_d}, \mathbf{B_d}, \mathbf{C_d}$ and $\mathbf{D}_d$ using least-squares:

$$\begin{pmatrix} \tilde{\mathbf{X}}_{i+1} \\ \mathbf{Y}_{i|i} \end{pmatrix} = \begin{pmatrix} \mathbf{A_d} & \mathbf{B_d} \\ \mathbf{C_d} & \mathbf{D_d} \end{pmatrix} \begin{pmatrix} \tilde{\mathbf{X}}_i \\ \mathbf{U}_{i|i} \end{pmatrix} + \begin{pmatrix} \boldsymbol{\rho}_w \\ \boldsymbol{\rho}_v \end{pmatrix}$$

7. Compute eigenvalues μ and eigenvectores ($\mathbf{\Psi}$) of the state matrix $\mathbf{A_d}$.

8. Determine modal properties of the system:

$$f_r = \frac{\sqrt{\lambda_r \lambda_r^*}}{2\pi}$$

$$\xi_r = \frac{-\mathrm{Re}(\lambda_r)}{|\lambda_r|}$$

$$\mathbf{\Phi} = \mathbf{C_d} \, \mathbf{\Psi}$$

with

$$\lambda_r = \frac{\ln(\mu_r)}{\Delta t}$$

Figure 3. Schematic overview of the DSI system identification method (adapted from [12]).

Identification of modal parameters from seismic experimental data

The changes of the modal properties of the building during the seismic tests are estimated by using short-time windows of input-output data. The acceleration measured at the top of the foundation (floor 1) is considered as the input and the acceleration responses at the south-east corner of the upper floors as output (see Fig. 1c), i.e., m = 1 and l = 5 with m = number of inputs and l = number of outputs. A single acceleration per floor in the longitudinal direction of the building is considered for conducting the identification because the shake table imposed the excitation in that direction, therefore, small acceleration responses were measured in the transverse direction and the torsional effects were minor. The minimum length criteria (Eq. 6) was chosen to define the length of the windows of data [6].

$$s_{\min} = 2i(m + l + 1), \tag{1}$$

In the case of the BNCS building, a number of block rows of the Hankel matrices equal to $i = 101$ was used based on preliminary analysis of identification results conducted with data from the beginning, strong motion, and final parts of the FB1-CNP100 seismic test. Then, the number of data samples on each short-time window is $s_{\min} = 2 \times 101(1+5+1) = 1414$. Because the sampling frequency was 200 Hz, the minimum window length corresponds to $1414/200 = 7.07$ s. The identification process was conducted considering an overlapping of 90% between consecutive time windows.

The physical modes were chosen as those satisfying the following criteria:

1. For a given short-time window, modes satisfying $\left|f^{(100)} - f^{(99)}\right| / f^{(100)} \leq 15\%$ and $\text{MAC}_{\phi^{(100)},\phi^{(99)}} \geq 75\%$, where $f^{(n)}$ = natural frequency for model order *n*, and $\text{MAC}_{\phi^{(n)},\phi^{(m)}}$ is the modal assurance criterion (MAC) [14] between modes identified for model orders *n* and *m*.
2. For consecutive short-time windows $(i-1)$ and *i*, modes satisfying $\left|f_{(i-1)} - f_{(i)}\right| / f_{(i-1)} \leq 15\%$ and $\text{MAC}_{\phi_{(i-1)},\phi_{(i)}} \geq 90\%$, where $f_{(n)}$ = natural frequency for time window *n*, and $\text{MAC}_{\phi_{(n-1)},\phi_{(n)}}$ is the MAC value between corresponding modes identified for windows (*n*-1) and *n*.

Based on system identification analyses conducted for the BNCS building using AV and WN base excitation test data [14], modes with natural frequencies below 15 Hz were analyzed. It is noted that the frequency range from 0 to 15 Hz includes the first three longitudinal modes of the building (1-L, 2-L, and 3-L) [14]. Only longitudinal modes are identified with the seismic data because the shaking direction was imposed in that direction, therefore, the identification associated with transverse and torsional modes is not plausible because of their low contribution to the dynamic response of the building. The mode shapes identified for the first time window of the FB1-CNP100 test are shown in Fig. 4.

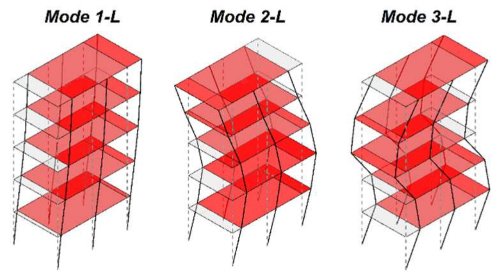

Figure 4. Identified mode shapes of the building.

The bottom panel of Fig. 5 depicts the input acceleration time-histories measured on top of the foundation (floor 1) and upper panels show the natural frequencies of the first three longitudinal modes of the building identified using the short-time window approach described above. In these plots, natural frequencies of the corresponding modes identified with AV and WN base excitation data reported in [14] are marked with green and red dashes, respectively, at the beginning and at the end of each seismic test. In addition, for the fundamental mode (1-L), black dashes during the strong motion phases report the identification results obtained in Chen et al. [15] by using an optimization approach based on the roof displacement measured from collocated accelerometers and GPS antennas.

It is noted that at the beginning of each seismic test, the input motion exhibits low amplitudes and the identified natural frequencies match very well those identified using AV data (green dashes) and are higher than those identified using WN base excitation data (red dashes). As the amplitude of the base excitation increases, the identified natural frequencies decrease significantly, but they recover toward the end of each test, reaching a value lower than that identified at the beginning of the same seismic test but significantly higher than the lowest frequencies values identified for the corresponding test. This implies that the equivalent lateral stiffness of the building decreases significantly during the strong motion phase (because of nonstructural and structural damage) but then it increases when the amplitude of the excitation decreases at the end of the seismic test. This behavior suggests that cracks in the concrete are opened during the strong motion phase and then they close after the amplitude of the excitation becomes small. However, some permanent degradation of the stiffness is observed, because the frequencies identified at the end of each seismic test are lower than those identified at the beginning of the same test. The same pattern is observed for the three longitudinal models. For model 1-L, at the beginning of test FB1-CNP100 a natural frequency 1.67 Hz is identified, progressively decreasing at the end of each of the seismic tests, and reaching a value of 0.73 Hz at the end of the final test FB6-DEN1100. Similarly, modes 2-L and 3-L progress from 6.77 and 10.88 Hz at the beginning of test FB1-CNP100 to 3.7 and 6.78 Hz at the end of test FB6-DEN100, respectively. For the first four tests (FB1-CNP100 to FB4-ICA100), the natural frequency of mode 1-L identified in this work in the strong motion phase and that obtained by Chen et al. [16] are in very good agreement.

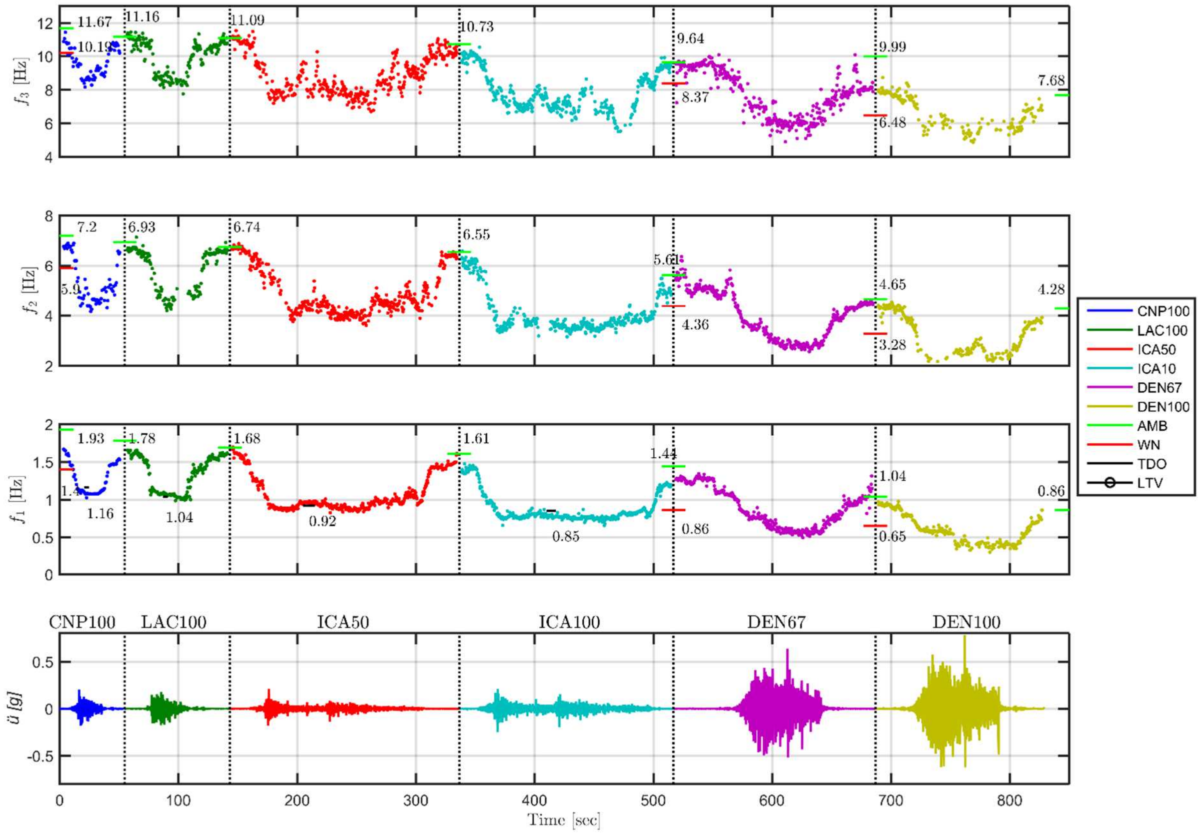

Figure 5. Temporal evolution of the natural frequencies identified for first three longitudinal modes.

Fig. 6 shows the identification results for the equivalent viscous damping ratios. In the top three plots, the identified damping ratios for modes 1-L, 2-L, and 3-L are shown. Consistently with Fig. 5, identification results from AV and WN test data [14] and those reported in [15] for the strong motion parts of the first four tests are included in the plots with dashed lines. Significant variations of the identified damping ratios are observed. In general, they tend to increase during the strong motion phase for all modes. In addition, the values identified at the beginning and end of the seismic tests are relatively close to those identified when AV data was employed. It is noted that identification results obtained using AV data are affected by environmental conditions, and, in particular, the effects on damping ratios are significant. As the seismic demand induced by the seismic excitation increases (from FB1-CNP100 to FB6-DEN100), the identified damping ratios also increase, suggesting that a larger amount of energy is dissipated when the excitation increases. It is worthy to note that the underlying mathematical model considered in the system identification method used, assumed that all the sources of energy dissipation are identified as equivalent viscous damping. For mode 1-L, the average value of the identified damping ratio in the strong motion part of the excitations is about 5-7% for the first four tests (FB1-CNP100 to FB4-ICA100) and increases to about 8 and 10% for tests FB5-DEN67 and FB6-DEN100, respectively. Similar values and trend are observed for the damping ratios identified for mode 2-L, while the damping ratio variation associated with mode 3-L is smaller.

The variation of the mode shapes during the seismic tests is also investigated in a similar fashion as the natural frequencies and damping ratios (Fig. 7). The MAC values are used to study the evolution of the mode shapes during the seismic test, taking the mode shapes identified at the beginning of test FB1-CNP100 as reference. For modes 1-L and 2-L, the MAC values only exhibit minor changes during the whole testing protocol (MAC>90% for most values), suggesting that damage does not affect significantly the mode shapes. For mode 3-L, some deviations from the unity are observed along few specific time windows; however, no logical pattern is observed, implying that this can be generated by artifacts related to the identification process.

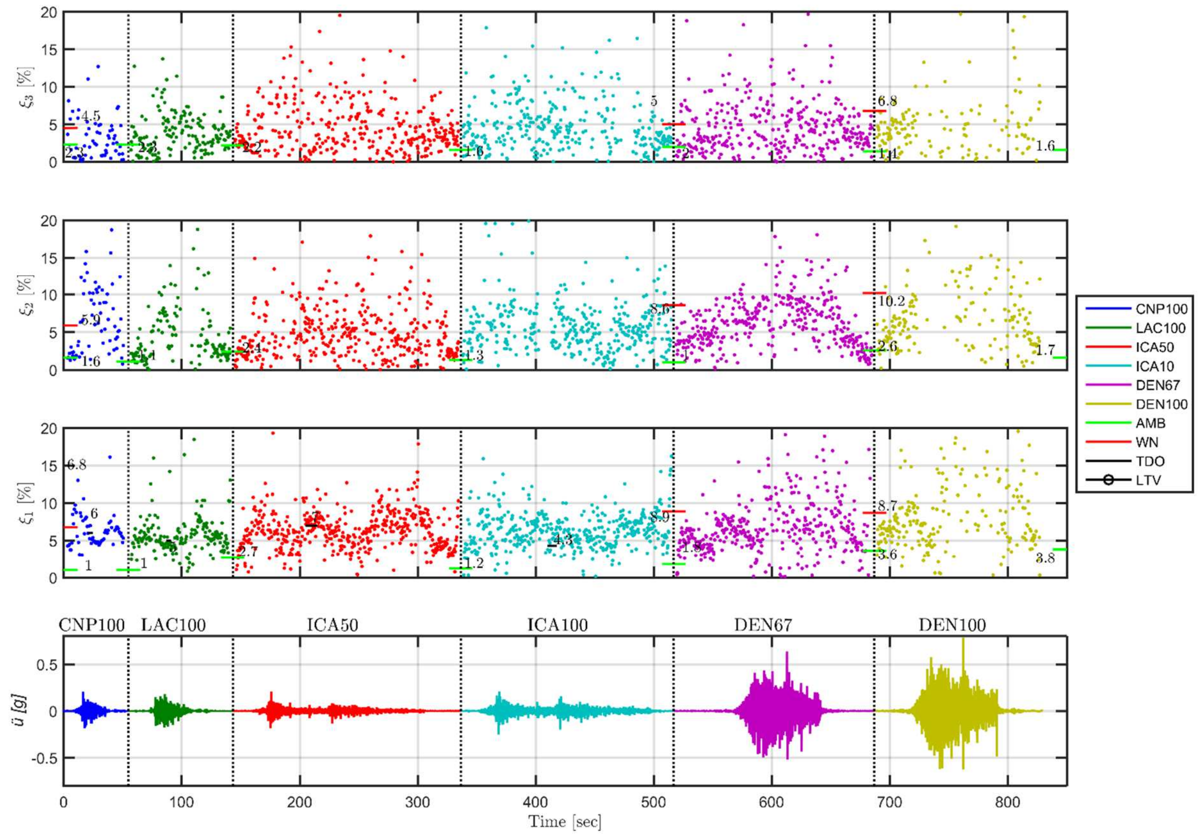

Figure 6. Temporal evolution of the damping ratios identified for first three longitudinal modes.

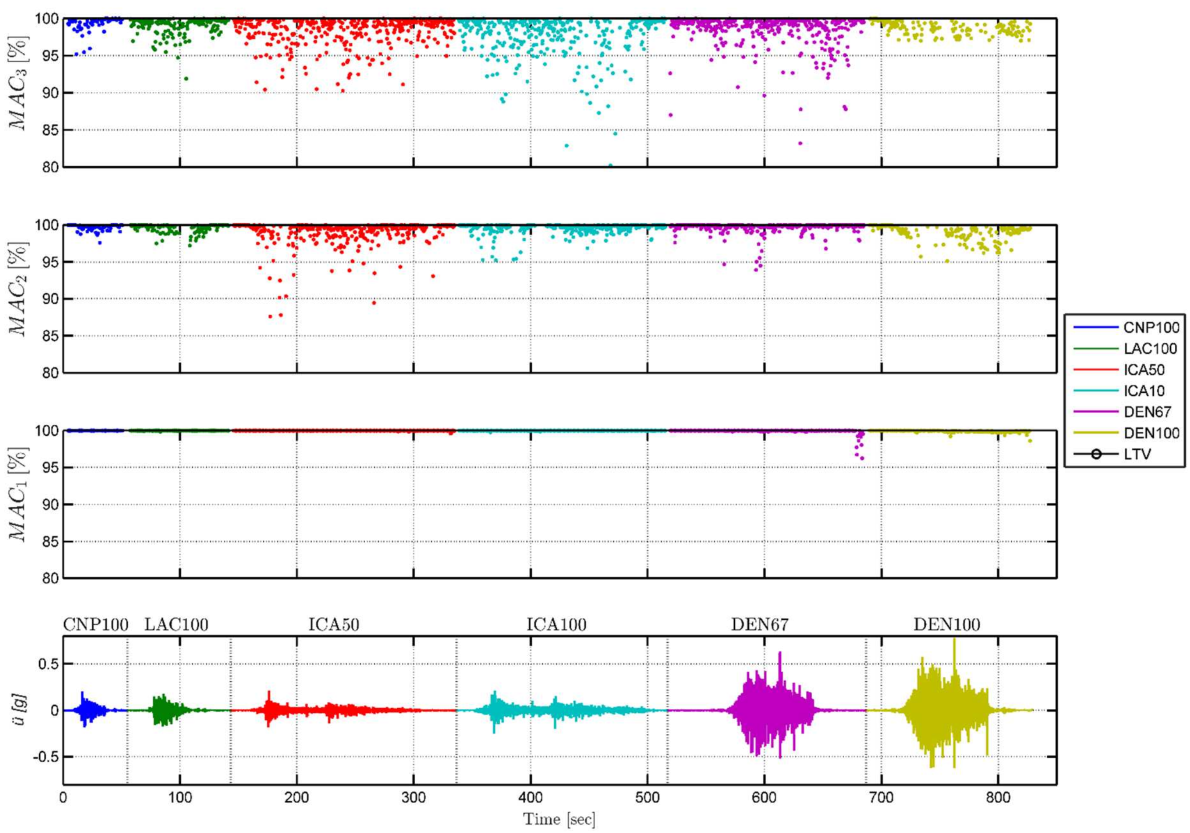

Figure 7. Temporal evolution of the MAC values corresponding to the first three longitudinal modes.

CONCLUSIONS

Experimental data collected from a series of six shake table tests conducted on a full-scale five-story reinforced concrete building specimen tested at the University of California, San Diego, was used to identified the modal properties of the building using seismic test data of varying intensities. The first three longitudinal modes of the building were identified using the deterministic-stochastic subspace identification method with input-output acceleration data recorded during the seismic tests. The time evolution of the modal parameters was tracked by using a short-time windowing approach. The nonlinear behavior of the building was observed from the temporal variation of the identified natural frequencies and equivalent damping ratios. The stiffness reduction in the building due to nonstructural and structural damage implied a reduction in the identified natural frequencies and an increment in the identified damping ratios. Natural frequencies and damping ratios identified using the data at the beginning of each seismic test were found in good agreement with those identified from ambient vibration data. The variation of the mode shapes was analyzed using the MAC, taking the mode shapes identified at the beginning of the first seismic test as reference. It was found that the mode shapes did not change significantly during the seismic tests, i.e., they were not affected by the damage.

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