System identification of a full-scale five-story reinforced concrete building tested on the NEES-UCSD shake table

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ABSTRACT: This paper presents the identification of the modal properties of a full-scale 5-story reinforced concrete (R/C) building outfitted with a variety of nonstructural components and systems (NCSs) tested on the unidirectional NEES-UCSD shake table in April-May, 2012. The building specimen was tested under base-isolated (BI) and fixed-base (FB) configurations. The purpose of the test program was to study the seismic response of the structure and NCSs and their dynamic interaction at different levels of seismic excitation. For each base configuration (BI and FB), the building was subjected to a sequence of earthquake base motions carefully selected so as to progressively increase the seismic demand on the structure and NCSs. Before and after each seismic test, ambient vibration (AV) data were recorded, and additionally, low amplitude white noise (WN) base excitation tests were conducted at key stages during the test protocol. Because of the low intensity of the ambient and WN base excitations, a quasi-linear response of the system is assumed, and the modal parameters of an equivalent linear time-invariant (LTI) model are estimated. Using the structural vibration data recorded by 28 accelerometers, three system identification methods, including two output-only (SSI-DATA and NeXt-ERA) and one input-output (DSI), are used to estimate the modal properties of the BI and FB building between seismic tests. The results show that the modal properties obtained by different methods are in good agreement and that the estimated modal parameters of the building are affected by the amplitude of the excitation, environmental conditions, and structural/nonstructural damage in the structure.

KEY WORDS: Full-scale specimen; Shake table; System identification; Damage; Base isolation; Nonstructural components.

1 INTRODUCTION

Vibration-based damage identification has attracted attention in the field of earthquake engineering over the past thirty years. It potentially allows to detect and locate damage by studying the changes in the dynamic characteristics of a structure from an initial to a damaged state, when the structure has been excited by natural or human-made loads or when the structure has age-deteriorated. Experimental and operational modal analyses are the main techniques to identify the modal parameters (natural frequencies, damping ratios, and mode shapes) from recorded structural vibration data. These modal identification results can be further used to apply vibration-based damage identification techniques, comparing the modal properties of a structure at different damage states (e.g., [1], [2]).

Base isolation technology has been proved as an effective way to protect buildings and their nonstructural components and systems (NCSs) against earthquakes (e.g., [3], [4]). Between the different types of isolators, high damping rubber bearings have emerged as one of the most widely used in practice. Because the use of base isolation strategies has grown only recently after the 1995 Kobe earthquake in Japan, structural vibration data recorded in base-isolated buildings during large earthquakes is limited (e.g., [5], [6], [7]).

To overcome the scarcity of datasets collected from densely instrumented buildings during large earthquakes, shake table tests have produced important, high quality, and unique data to assess the dynamic properties of buildings at different states of damage (e.g., [8], [9]) and of buildings incorporating base isolation (e.g., [3]). However, due to the high costs associated with this type of experiments, their number is still very limited.

In 2012, a landmark project involved the testing of a full-scale five-story reinforced concrete (R/C) building fully outfitted with a large variety of nonstructural components and systems (NCSs). The building, in both its base-isolated (BI) and fixed-base (FB) configurations, was subjected to a suite of earthquake base motions on the NEES-UCSD unidirectional shake table. In this paper, the modal properties of the test specimen for both BI and FB configurations are identified using input-output and output-only methods with ambient vibration (AV) and low amplitude white noise (WN) base excitation test data.

2 DESCRIPTION OF THE BUILDING TEST SPECIMEN, INSTRUMENTATION, AND TEST PROTOCOL

2.1 Building test specimen and isolation system

The test building was a full-scale 5-story cast-in place R/C structure fully outfitted with a broad array of NCSs. The building had two bays in the longitudinal direction (direction of shaking) and one bay in the transverse direction, with plan dimensions of 11.0×6.6 m, respectively. The building had a floor-to-floor height of 4.27 m, a total height (measured from the top of the foundation to the top of the roof slab) of 21.34 m and an estimated total weight of 3010 kN for the bare structure and 4420 kN for the structure with all the NCSs, both excluding the foundation, which weighted 1870 kN. The seismic resisting system was provided by two identical one-bay special R/C moment resisting frames oriented east-west.
The beams had different details at different floors. Six 0.66×0.46 m columns were reinforced with 6 #6 and 4 #9 longitudinal bars and a prefabricated transverse reinforcement electro-welded grid. The floor system consisted of a 0.2 m thick concrete slab at all levels. Two 0.15 m thick transverse R/C shear walls provided the support for the elevator guiderails. Figure 1 shows the test specimen and elevation and plan views. For the first testing phase, the building was mounted on four high damping rubber isolators, located at the four corners of the foundation. The isolators had a rubber diameter of 0.65 m, a total height of 0.34 m, and a rubber height of 0.20 m (Figure 1d). Further information about the structural system, nonstructural components, and their design considerations can be found in [10].

Figure 1. Test specimen (a) completed building; (b) elevation view; (c) plan view; (d) view of isolator. (Dimensions in m).

2.2 Instrumentation array

The building was instrumented with a dense accelerometer array, consisting of four triaxial accelerometers per floor, one at each corner as represented by red circles in Figure 1c. Also, six triaxial accelerometers were placed on the North-East and South-West corners of the shake table platen. The accelerometers were force-balance Episensor, with a full-scale of ±4g, a frequency bandwidth DC–200 Hz, and a wide dynamic range of 155dB. The data acquisition system consisted of Quanterra Q330 digitizers from Kinemetrics Inc. In this study, the acceleration response of the building measured by 28 accelerometers (2 for each translational direction of each floor) and sampled at 200 Hz is used to identify the dynamic properties of the test specimen. Before the system identification process, the time series were detrended and filtered using a band-pass IIR Butterworth filter of order 4 with cut-off frequencies at 0.15 and 25 Hz, defining a frequency range containing all the vibration modes contributing noticeably to the building response.

2.3 Test protocol

The seismic tests on the building were conducted in April and May, 2012. In addition to the seismic base excitation tests, AV was continuously recorded from April 14 to May 18, and low-amplitude WN base excitation tests using the NEES-UCSD shake table were conducted at key stages during the test protocol. The seismic input motions were designed carefully and applied to the building to progressively increase the seismic demand on the structure and NCSs. Seven and six seismic input motions were applied to the BI and FB building, respectively. Table 1 summarizes the seismic test protocol and Figure 2 shows the acceleration time histories (THs) and the 5% damped elastic displacement response spectra (EDRS) of the seismic input motions used for the BI and FB building configurations.

Table 1. Seismic test protocol.

<table>
<thead>
<tr>
<th>Date</th>
<th>Seed motion (Earthquake)</th>
<th>Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>04-16-2012</td>
<td>Canoga Park (Northridge 1994)</td>
<td>BI1-CNP100</td>
</tr>
<tr>
<td>04-16-2012</td>
<td>LA City (Northridge 1994)</td>
<td>BI2-LAC100</td>
</tr>
<tr>
<td>04-17-2012</td>
<td>LA City (Northridge 1994)</td>
<td>BI3-LAC100</td>
</tr>
<tr>
<td>04-17-2012</td>
<td>San Pedro (Chile 2010)</td>
<td>BI4-SP100</td>
</tr>
<tr>
<td>04-26-2012</td>
<td>ICA 50% (Peru 2007)</td>
<td>BI5-ICA50</td>
</tr>
<tr>
<td>04-27-2012</td>
<td>ICA 100% (Peru 2007)</td>
<td>BI6-ICA100</td>
</tr>
<tr>
<td>04-27-2012</td>
<td>ICA 140% (Peru 2007)</td>
<td>BI7-ICA140</td>
</tr>
<tr>
<td>05-07-2012</td>
<td>Canoga Park (Northridge 1994)</td>
<td>FB1-CNP100</td>
</tr>
<tr>
<td>05-09-2012</td>
<td>LA City (Northridge 1994)</td>
<td>FB2-LAC100</td>
</tr>
<tr>
<td>05-09-2012</td>
<td>ICA 50% (Peru 2007)</td>
<td>FB3-ICA50</td>
</tr>
<tr>
<td>05-11-2012</td>
<td>ICA 100% (Peru 2007)</td>
<td>FB4-ICA100</td>
</tr>
<tr>
<td>05-15-2012</td>
<td>Taps Pump #9 67% (Denali 2002)</td>
<td>FB5-DEN67</td>
</tr>
<tr>
<td>05-15-2012</td>
<td>Taps Pump #9 100% (Denali 2002)</td>
<td>FB6-DEN100</td>
</tr>
</tbody>
</table>

Figure 2. Achieved seismic input motions: (a) acceleration THs for the BI building; (b) EDRS (ξ = 5%) for the BI building; (c) acceleration THs for the FB building; (b) EDRS (ξ = 5%) for the FB building.

3 SYSTEM IDENTIFICATION

3.1 Methods

To estimate the modal properties of the building, two state-of-the-art output-only system identification methods were used for the AV data: Data-Driven Stochastic Subspace Identification (SSI-DATA) [11] and Multiple-Reference...
Natural Excitation Technique combined with Eigensystem Realization Algorithm (NExT-ERA) [12]. For the low-amplitude WN base excitation test data, in addition to the two abovementioned output-only methods, one input-output method was employed: Deterministic-Stochastic Subspace Identification (DSI) [11].

The three methods used in this paper work with the state-space (SS) formulation of the equation of motion (state equation), which, together with the measurement equation, define the SS model

\[
x_{k+1} = A_d x_k + B_d u_k + w_k
\]

(1)

\[
y_k = C_d x_k + D_d u_k + v_k
\]

(2)

where \( x_k, u_k, y_k \) = state, input, and output vectors at time \( t_k \); \( w_k \) = process and measurement noise vectors at time \( t_k \); \( A_d \) = discrete state matrix (dynamical system matrix); \( B_d \) = discrete input matrix; \( C_d \) = discrete output matrix; and \( D_d \) = discrete direct feed-through matrix.

From the relationship between the discrete and continuous state matrices \( A_d = \exp(A_d \Delta t) \) where \( \Delta t \) is the sampling time, it can be proven that their eigenvectors (\( \Psi \)) are identical, while the continuous and discrete eigenvalues, \( \lambda_c \) and \( \lambda_d \) respectively, are related as

\[
\lambda_d = \frac{\ln(\lambda_c)}{\Delta t}
\]

(3)

Then, from the eigenvalues and eigenvectors of \( A_d \), the modal frequencies (\( f_c \)), modal damping ratios (\( \xi_c \)) and mode shapes (\( \Phi_c \)) of the system can be obtained as

\[
f_c = \frac{\omega_c}{2\pi}; \quad \xi_c = \frac{\text{Re}(\lambda_c)}{\|\Phi_c\|}; \quad \Phi_c = C_d \Psi
\]

(4)

SSI-DATA and NExT-ERA are output-only system identification methods, i.e., \( u_k = 0 \) in Eq. (1), while DSI is an input-output method. The three methods assume that \( w_k \) and \( v_k \) are zero-mean Gaussian white processes. The order of the identified SS model is defined by using the stabilization diagram, which summarizes the identified modal parameters versus model order (e.g., [13], [14]), with criteria

\[
\left| f_c - f_j \right| \leq 1\% ; \quad \left| \xi_c - \xi_j \right| \leq 5\% ; \quad \left(1 - \text{MAC}_{f,i} \right) \leq 2\%
\]

(5)

where \( f_i, \xi_i \) and \( f_j, \xi_j \) are the identified natural frequencies and damping ratios for models of successive orders \( i \) and \( j \) = \( i+2 \), and \( \text{MAC}_{f,i} \) is the modal assurance criterion [15] of a pair of corresponding mode shapes identified for models of successive orders. Figure 3 shows the stabilization diagram obtained using SSI-DATA for the AV data recorded on the fixed-base building on May 5, 2012, between 17:30 and 17:40 PST. The eleven mode shapes identified using the method proposed in [16]. The eleven mode shapes identified from the first data set are shown in Figure 5. They correspond to the first four longitudinal modes (1-L, 2-L, 3-L, and 4-L), the first three coupled transverse-torsional modes (1-T+To, 2-T+To, and 3-T+To), and the first four torsional modes (1-To, 2-To, 3-To, and 4-To). The modes 1-T+To, 1-L, and 1-To correspond mainly to the deformation of the isolation system and are identified as isolation modes, while the modes 2-T+To to 4-To mostly represent the deformation of the superstructure and are referred to as structural modes.
Polar plots are used to investigate the level of non-proportional or non-classical damping in the identified mode shapes. Figure 6 shows that the identified mode shapes are almost purely classically damped, since all the vectors shown in the polar plots are almost collinear. The most likely reason is that the level of structural vibration is very low for AVs; consequently, the level of shear deformation in the isolators is low and the damping effect of the isolation system is minor.

The natural frequencies obtained from the automated identification process are plotted as a function of time in Figure 7. In this plot, vertical dashed lines mark the times of the seven seismic tests conducted on the BI building. The natural frequencies of the modes corresponding to the deformation of the isolation system (1-T+To, 1-L, and 1-To) decrease after each seismic test, but recover in time before the next seismic test. Since the total mass of the building remained practically unchanged during the complete BI test phase, the decrease in the natural frequencies of the isolation modes is due to a reduction in the lateral (shear) stiffness of the isolators, which recovers in time while the building is not subjected to another seismic excitation. These modal identification results indicate that the BI building does not suffer any structural damage during the BI test phase, and only minor nonstructural damage was observed. It is noted that the peak interstory drift ratio remained below 0.4% during the BI test phase.

To analyze the overall trend of the identified modal damping ratios during the BI test phase, Figure 8 shows the evolution of the identified equivalent viscous damping ratios for modes 1-T+To, 1-L, 1-To, 2-T+To, 2-L, and 2-To. The damping ratios of the isolation modes (1-T+To, 1-L, and 1-To) are higher than those of the structural modes 2-T+To, 2-L, and 2-To. Cyclic variations in the identified natural frequencies and damping ratios due to changes in environmental conditions are observed, similarly to the case of the FB building [17].

System identification based on low amplitude white noise base excitation test data

Figure 9 and Figure 10 show the natural frequencies and equivalent damping ratios, respectively, identified using SSI, NiExT-ERA, and DSI from the measured response during the WN base excitation tests conducted before BI1 (1A and 1B), after BI4 (5A and 5B), and after BI7 (8A and 8B). It is noted that before each seismic test, a sequence of three WN base excitations, with nominal amplitude 1.5%g (A), 3.0%g (B), and 3.5%g (C) RMS base acceleration respectively, were applied to the BI building. The identified natural frequencies (Figure 9) show a good agreement between the different system ID methods for each WN test, with relative differences lower than 10%. Moreover, for the same level of base excitation (same RMS base acceleration), the identified natural frequencies show differences lower than 15% before and after the seismic tests. However, as the level of base excitation (RMS) increases, the identified natural frequencies of most of the modes decrease. This is due to the nonlinear behavior of the base isolators during the WN tests; their shear stiffness decreases with increasing shear deformation. Consequently, the natural frequencies of the identified equivalent LTI model decrease as the level of excitation increase from 1.5%g to 3.0%g RMS acceleration. The identified damping ratios (Figure 10) corresponding to the first three modes of vibration (1-L, 1-L+T, and 1-L+To) are...
high, ranging between 7 and 15%. These high damping ratios are most likely due to the contribution of the isolator bearings, (see Figure 11). On the other hand, the damping ratios of the modes associated with the deformation of the superstructure (modes 2-L to 4-L) are mostly in the range 1 to 5%.

Figure 9. Natural frequencies of the BI building identified before BI1 (1A and 1B), after BI4 (5A and 5B), and after BI7 (8A and 8B) using WN base excitation test data.

Figure 10. Equivalent damping ratios ($\zeta$) of the BI building identified before BI1 (1A and 1B), after BI4 (5A and 5B), and after BI7 (8A and 8B) using WN base excitation test data.

The mode shapes of the ten modes identified using data from the 1.5%g RMS WN base excitation test conducted before test BI1 are shown in Figure 11. These modes correspond to the first four longitudinal modes (1-L, 2-L, 3-L, and 4-L), the first three coupled longitudinal-transverse modes (1-L+T, 2-L+T, and 3-L+T), and the first three coupled longitudinal-torsional modes (1-L+To, 2-L+To, and 3-L+To). Figure 12 shows the polar plots of the mode shapes identified using DSI with data from the 1.5%g RMS WN base excitation test conducted before test BI1. These mode shapes are close to purely classically damped, most likely due to the low level of shear deformation induced by the WN base excitation in the isolation system in the case of the structural modes. However, the degree of non-proportional damping is higher than in the modes identified using AV data.

Figure 11. Mode shapes of the BI building identified using DSI with data from 1.5%g RMS WN base excitation test before test BI1.

Figure 12. Polar plot of the mode shapes of the BI building identified using DSI with data from 1.5%g RMS WN base excitation test before test BI1.

Figure 13 compares the absolute acceleration THs measured in the E-W direction at the roof of the building with their counterparts simulated using the SS model identified with DSI. Figure 13a shows the comparison for the 1.5%g RMS WN test conducted before the seismic test BI1, while Figure 13b shows the comparison for the 3.5%g RMS WN test conducted after the seismic test BI7. A very good match is observed between the measured and identified responses in both cases. Similar results were obtained for all the WN tests performed. The very good match obtained between the measured and predicted (using the identified model) acceleration responses for all WN tests confirms the accuracy of the identified equivalent (or effective) modal parameters.

Figure 13. Measured versus identified absolute acceleration responses at the roof of the BI building for WN tests: (a) 1.5%g RMS before BI1; (b) 3.5%g RMS after BI7.

3.3 System identification of the fixed-base building

Because the seismic base excitation tests for the FB building induced structural damage, different damage states were defined before and after each seismic test. These damage states range from DS0, which corresponds to the undamaged state before test FB1, to DS6, which corresponds to the damage state after test FB6 (see Table 1).
System identification based on ambient vibration test data

Figure 14 and Figure 15 show the natural frequencies and equivalent damping ratios (\(\zeta\)) obtained using SSI-DATA and NEXT-ERA from the AV data recorded at different damage states. The natural frequencies estimated by SSI and NEXT-ERA are in very good agreement for all the damage states. As the damage induced by the seismic tests increases, the natural frequencies decrease due to the stiffness degradation caused by the damage. The higher modes are less sensitive to low levels of damage than the lower modes, whose natural frequencies begin to decrease from the initial states of damage.

Figure 14. Natural frequencies of the FB building identified using AV data at different damage states.

The damping ratios (Figure 15) identified using the two identification methods are in good agreement. The agreement is better at the lower modes than at the higher modes; however, their variability of the two estimates of the damping ratios is larger than that of the natural frequencies. The estimated damping ratios do not show a clear trend as the damage progresses, but they are in the range 0 to 5%, which is in agreement with previous studies for similar structures (e.g., [8]).

In the undamaged state (DS0), the lowest frequency corresponds to the mode 1-T+To (first coupled transverse-torsional mode). Because the seismic base motions were applied to the building along its longitudinal (E-W) direction, and consequently most of the structural and nonstructural damage was produced by deformations of the building in this direction, from DS1 onwards, mode 1-L has the lowest natural frequency (see Figure 14); therefore, the building lateral stiffness degrades faster in the longitudinal than in the transverse. A similar observation can be made for the higher modes (2-T+To, 2-L, and 2-L+To).

The mode shapes identified using SSI-DATA from AV data collected at DS0 are presented in Figure 16. The first five longitudinal, the first three torsional, and two coupled translational-torsional modes can be identified using the AV data. From the polar plots (not shown here due to space limitation) of the mode shapes identified for the FB building using both AV and WN base excitation test data, it can be concluded that the identified modes are almost purely classically damped.

The MAC coefficient [15] is used to compare the mode shapes estimated by SSI and NEXT-ERA for AV test AMB1 (Figure 17) conducted at DS0. Most of the MAC values along the diagonal (corresponding modes) of the MAC coefficient matrix are close to one, indicating that the mode shapes identified using both methods are close to each other. A similar pattern exists at all states of damage. As for the damping ratios, the agreement between the mode shapes identified using the two methods is better for the lower modes than for the higher modes. This is due to the fact that the participation of the higher modes in the measured responses is lower than that of the lower modes and, consequently, the signal-to-noise-ratio (SNR) is lower for the higher modes.

The modal properties of the FB building were also identified using back-to-back ten-minute long time windows of AV data recorded continuously between May 5 and May 18, 2012. The identified modal parameters were analyzed statistically in order to investigate the variability and consistency between different system identification methods.

The MAC coefficient [15] is used to compare the mode shapes estimated by SSI and NEXT-ERA for AV test AMB1 (Figure 17) conducted at DS0. Most of the MAC values along the diagonal (corresponding modes) of the MAC coefficient matrix are close to one, indicating that the mode shapes identified using both methods are close to each other. A similar pattern exists at all states of damage. As for the damping ratios, the agreement between the mode shapes identified using the two methods is better for the lower modes than for the higher modes. This is due to the fact that the participation of the higher modes in the measured responses is lower than that of the lower modes and, consequently, the signal-to-noise-ratio (SNR) is lower for the higher modes.

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Figure 15. Equivalent damping ratios (\(\zeta\)) of the FB building identified using AV data at different damage states.

Figure 16. Mode shapes of the FB building identified using SSI-DATA from AV data at DS0.

Figure 17. MAC values between the mode shapes of the FB building identified from AV data at DS0.
In addition, the effects of environmental conditions on the identified modal parameters were investigated. Due to space limitation, these results are not presented here, but can be found in [17].

System identification based on low amplitude white noise base excitation test data

At damage states DS0, DS4, and DS5, six minutes of WN base excitation with a RMS acceleration of 1.5%g were applied to the test specimen. Figure 18 and Figure 19 show the natural frequencies and damping ratios identified using SSI-DATA, NExT-ERA, and DSI from the WN test data recorded at the three damage states. As in the case of AVs, the natural frequencies estimated by the different methods are in very good agreement and they decrease as the damage progresses in the system. However, the natural frequencies of the modes with components predominantly in the longitudinal direction exhibit greater reductions than the other modes, since, as explained earlier, most of the structural and nonstructural damage is associated to structural deformations in the direction of the excitation. It is noticed that for all the WN tests, the mode 1-L has the lowest natural frequency.

Figure 19. Equivalent damping ratios ($\xi$) of the FB building identified using 1.5%g RMS WN base excitation test data at different damage states.

By comparing the natural frequencies identified from AV and WN base excitation test data at the same damage states, it is observed that the natural frequencies decrease by a similar percentage from AV to WN test data.

The MAC coefficient is used to compare the modes shapes identified by the three different methods (Figure 20). Most of the MAC values along the diagonal of the MAC coefficient matrix are close to one, from which we can conclude that the corresponding mode shapes identified by the different methods are close to each other, with a better agreement for lower modes than higher modes for the same reason as previously explained for Figure 17. Similar results were obtained at all damage states.

The same comparison was carried out for all the WN tests, allowing to conclude that the matching between the measured and simulated (from identified SS models) building responses are in good agreement at all floor levels. However, the level of discrepancy between the recorded and simulated responses increases as the level of damage in the structure increases. This is due to the increasing nonlinear response behavior of the structure (even at low amplitude response level) as the damage increases, which cannot be captured accurately by the LTI model assumed by the system identification methods used herein.

Figure 21 compares the absolute acceleration THs measured in the E-W direction at the roof level of the FB building with their counterparts simulated from the SS model identified by DSI using data from WN base excitation tests with 1.5%g and 3.5%g RMS acceleration performed at damage states DS0 and DS5, respectively.

Figure 21. Measured versus identified absolute acceleration responses of the FB building at the roof level for WN tests: (a) 3.5%g RMS before FB1; (b) 3.5%g RMS after FB5.
4 CONCLUSIONS

A full-scale five-story R/C frame building fully outfitted with a variety of nonstructural components and systems (NCSs) was built and tested on the NEES-UCSD shake table while in a base-isolated (BI) and fixed-based (FB) configuration. The building was subjected to a suite of earthquake base motions of increasing intensity. Using the structural response measured by 28 accelerometers during ambient vibrations (AV) and low amplitude white noise (WN) base excitations, three system identification (SID) methods, including two output-only (SSI-DATA and NExT-ERA) and one input-output (DSI) method, were used to identify the modal parameters of the BI and FB building between individual seismic tests.

BI building

From the modal properties identified using AV data, it is observed that the natural frequencies of the modes associated with the deformation of the isolation system decrease after each seismic test, but recover in time before the next seismic test. The effect of environmental conditions on the identified natural frequencies and damping ratios is observed through the analysis of continuous-time recorded AV data. From the modal parameters identified using WN base excitation test data, it is concluded that the natural frequencies of the building did not change significantly before and after the seismic tests for the same level of WN base excitation (RMS). However, as the level (RMS) of the WN base excitation increases, the natural frequencies decrease due to the softening behavior of the base isolators as their shear deformation increases. The damping ratios corresponding to the first three identified modes of vibration are high ([7-15]%), because these modes correspond mostly to deformations of the isolation system (i.e., isolation modes). The damping ratios identified for the other modes (structural modes) which are mainly associated with deformations of the superstructure are significantly lower ([1-6]%).

FB building

A study of the modal properties identified using AV and WN base excitation test data shows that the natural frequencies decrease as the damage in the building increases. The reduction in natural frequency, due to the degradation of the stiffness of the structure caused by the damage, is more significant for modes corresponding to deformations of the building in the longitudinal direction (e.g., longitudinal modes) which is the direction of the seismic excitations. The identified damping ratios do not show a clear trend as a function of damage state, but longitudinal modes have higher damping ratios than the coupled translational-torsional and torsional modes. The correlation between the mode shapes identified using different methods was studied using the MAC coefficient, which shows a good agreement between the different SID methods at each state of damage.

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REFERENCES