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Damage Identification of a Three-Story Infilled RC Frame Tested on the UCSD-NEES Shake Table

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ABSTRACT

Reinforced concrete (RC) frames with masonry infill walls can be frequently found in areas of high seismic risk around the world. Such structures are often designed with older building codes and may experience catastrophic failures during earthquakes. A 2/3-scale, three-story, two-bay, infilled RC frame was tested on the UCSD-NEES shake table to investigate the seismic performance of this type of construction. The frame was designed according to the building practice in California in the 1920s. The shake table tests were designed so as to induce damage in the structure progressively through scaled earthquake records. At various levels of damage, low-amplitude white-noise base excitations were applied to the infilled RC frame, which responded as a quasi-linear system with modal parameters depending on the extent of the structural damage. In this study, the modal parameters identified from the white-noise test data acquired at various levels of damage are used to detect the existing damage. A sensitivity-based finite element model updating strategy is employed to detect, locate, and quantify damage at each damage state considered. This paper presents the results of the damage identification study, which shows that the method can accurately identify the location and severity of damage observed in the tests.

Introduction

In recent years, vibration-based structural health monitoring has attracted increasing attention and is of growing importance in the civil engineering research community. Vibration-based, non-destructive, damage identification uses changes in dynamic characteristics (e.g., modal parameters) of the structure to identify damage. Extensive literature reviews have been provided by Doebling et al. [1, 2] and Sohn et al. [3] on this subject. Sensitivity-based finite element (FE) model updating methods have been applied successfully for condition assessment of structures [4, 5]. These methods update the physical parameters of a FE model of the structure as damage evolves by minimizing an objective function that measures the discrepancy between FE predicted and experimentally identified structural dynamic properties, such as natural frequencies and mode shapes. Optimal solutions of the problem are reached through sensitivity-based constrained optimization algorithms.

A 2/3-scale infilled RC frame, designed according to the engineering practice in California in the 1920s, was tested on the UCSD-NEES shake table in November 2008 to assess the seismic performance of such older buildings as part of a collaboratory project involving researchers from Stanford University, the University of Colorado at Boulder, and the University of California at San Diego. This two-bay, three-story frame was the largest structure of this type ever tested on a shake table. The shake table tests were designed so as to induce damage in the building progressively through scaled earthquake records. Between the earthquake tests, low-amplitude white-noise base excitation tests were applied to the infilled RC frame, which responded as a quasi-linear system with modal parameters evolving as a function of damage. The deterministic-stochastic subspace identification (DSI) method, based on system input and output signals, was used to estimate the modal parameters (natural frequencies, damping ratios, and mode shapes) of the undamaged structure and of the structure in its various damage states [6]. In this study, a FE model updating strategy is applied for damage identification of the structure after it was subjected to earthquake excitations of increasing amplitude. The objective function used here for damage identification is defined as a combination of the discrepancies between

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the FE predicted and experimentally identified natural frequencies and mode shapes. The damage identification results are compared to the damage observed in the physical specimen.

Test Specimen, Test Setup, and Dynamic Experiments

Three-Story Infilled RC Frame

The three-story specimen shown in Figure 1 was tested on the outdoor shake table at the University of California, San Diego in November 2008. The prototype structure was designed by Stavridis and Shing [7] to represent structures built in California in the 1920s era. The specimen corresponds to the external frame of the prototype structure with all its dimensions scaled by 2/3, while the infill walls had two wythes of brick units. The scaling factors were determined according to the similitude requirement and some limitations of the test setup [8]. Time and acceleration scale factors of 0.54 and 2.26, respectively, were applied to the ground motions to satisfy the similitude requirements of the experiments [9]. The ground motion levels mentioned hereafter are with respect to the full-scale prototype structure. As shown in Figure 1, two steel towers were secured on the shake table on the north and south side of the structure to protect the shake table from a potential collapse of the structure during severe shaking. However, they did not interact with the structure during the tests as they were placed with a 0.75 inch gap from the specimen. More details about the test structure can be found in [8] and in a companion paper on system identification of the test specimen [6].

Fig. 1 Front view (left) and side view of the specimen (right)

Instrumentation Layout

The specimen and the steel towers were instrumented with an extensive array of sensors. They included 135 strain gages, 71 string pots and LVDTs, and 59 uniaxial accelerometers. The accelerometers were used to measure the accelerations along directions X, Y and Z, with X being the direction of base excitation (longitudinal), Y the transverse (out-of-plane) direction, and Z the vertical direction. The measured response data from 9 longitudinal (3 per floor), 9 vertical (3 per floor) and 6 transversal (2 per floor) acceleration channels were used to identify the modal parameters of the test structure [6]. The locations of the accelerometers are shown in Figure 2.

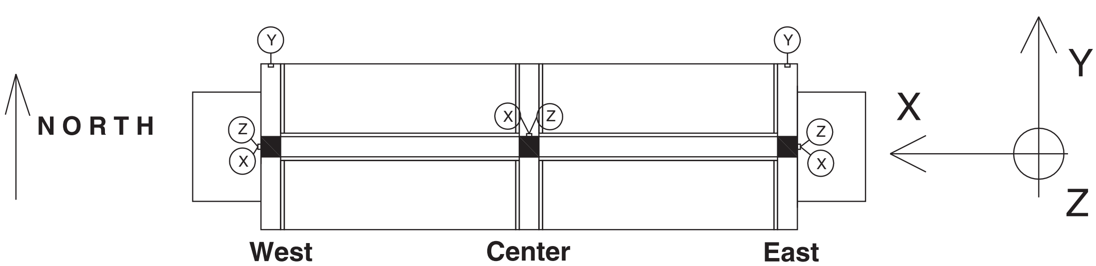


Fig. 2 Locations and directions of accelerometers used in this study on each floor level

Dynamic Tests Performed

A sequence of 44 dynamic tests was applied to the test structure including ambient vibration tests, free vibration tests, and forced vibration tests (white noise and seismic base excitations) using the UCSD-NEES shake table. The testing sequence incorporated earthquake ground motions of increasing amplitude to gradually damage the structure. Between the earthquake records, low amplitude white noise base excitation tests were performed and the modal parameters of the structure were identified at various damage states [6]. The ground motion accelerograms were obtained by compressing the time and scaling in amplitude the acceleration time history recorded at the Gilroy 3 station during the 1989 Loma Prieta earthquake. For structures with a fundamental frequency close to that of the infilled frame studied here, the Gilroy 3 motion scaled at 67% corresponds to a Design Basis Earthquake (DBE) for Seismic Design Category (SDC) D. Moreover, the unscaled Gilroy 3 motion corresponds to a Maximum Considered Earthquake (MCE). From the recorded data, damage was identified at seven damage states of the structure (S0 and S2-S7). Damage state S0 is defined as the undamaged (baseline) state of the structure before its exposure to the first seismic excitation (EQ1), while damage states S1 to S7 correspond to the state of the structure after exposure to the earthquakes of increasing amplitude. Table 1 summarizes the dynamic tests performed and the corresponding damage states of the test structure.

Table 1. Dynamic tests used in this study (WN: white noise base excitation; EQ: earthquake base excitation)

Test No.	Test Date	Test Description	Damage State
5	11/3/2008	0.03g RMS WN, 5 min	S0
8	"	20% Gilroy EQ	
9	"	0.03g RMS WN, 5 min	S1
12	11/6/2008	40% Gilroy EQ	
13	"	0.03g RMS WN, 5 min	S2
21	11/10/2008	67% Gilroy EQ (DBE)	
25	11/12/2008	0.04g RMS WN, 5 min	S3
26	"	67% Gilroy EQ (DBE)	
27	"	0.04g RMS WN, 5 min	S4
28	"	83% Gilroy EQ	
29	"	0.04g RMS WN, 5 min	S5
33	11/13/2008	91% Gilroy EQ	
35	"	100% Gilroy EQ (MCE)	
36	"	0.04g RMS WN, 5 min	S6
40	11/18/2008	120% Gilroy EQ	
41	"	0.04g RMS WN, 5 min	S7

System Identification Results

The modal parameters of the specimen at all the damage states considered in this study were identified using the deterministic-stochastic subspace identification (DSI) method, based on the input-output data collected from low amplitude (0.03g and 0.04g RMS) white noise base excitation tests performed at various damage states (S0, S1, S2, S3, S4, S5, S6, and S7) [6]. The natural frequencies and mode shapes of the first two longitudinal vibration modes (1-L, 2-L) were used in the FE model updating procedure. Figure 3 shows the real part of the longitudinal mode shapes of the test structure identified based on data from Test 5 (damage state S0). The natural frequencies and damping ratios of these modes are reported in Table 2 for the eight damage states considered. Modal Assurance Criterion (MAC) values [10] were also computed to compare the mode shapes identified at each damage state with the corresponding mode shapes identified for the undamaged structure at state S0, which serves as a baseline condition of the structure. From Table 2, it can be observed that the identified natural frequencies and the MAC values between the identified mode shapes and their counterpart identified at S0 decrease as the level of damage increases. The high MAC values (close to one) in damage states S1 to S4 indicate that there is little change in the identified mode shapes between these damage states.

Table 2. Modal parameters of the infilled frame identified at different damage states

Damage State:		S0	S1	S2	S3	S4	S5	S6	S7
Mode 1-L	Frequency [Hz]	18.18	18.11	17.99	16.74	15.93	14.78	8.47	5.34
	Damping [%]	2.0	2.4	1.9	3.3	3.8	6.1	15.7	15.6
	MAC	1.00	1.00	1.00	1.00	1.00	0.98	0.80	0.71
Mode 2-L	Frequency [Hz]	41.22	41.09	41.56	40.21	38.56	35.50	27.34	22.57
	Damping [%]	1.1	1.0	1.0	1.4	3.0	4.4	4.8	4.2
	MAC	1.00	1.00	1.00	0.99	0.96	0.92	0.74	0.67

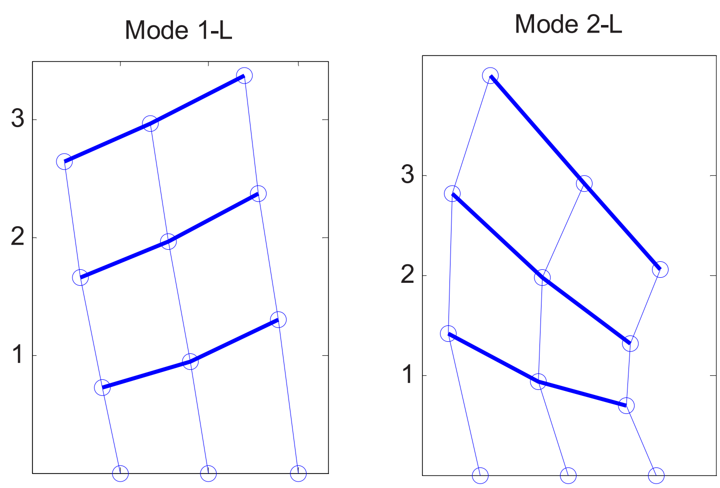


Fig. 3 First two longitudinal mode shapes of the infilled frame at damage state S0

Finite Element Model Updating for Damage Identification

In this study, a sensitivity-based FE model updating strategy is used to identify the damage in the structure after different levels of earthquake base excitation. The residuals used in the updating procedure are based on the identified natural frequencies and mode shapes for the first two longitudinal modes of the specimen. Damage is defined as a relative change in material stiffness (effective modulus of elasticity) of the finite elements in the different substructures of the FE model used for damage identification. For the purpose of damage identification, the effective moduli of elasticity of elements in the various substructures (each assumed to have a uniform value of the effective modulus of elasticity) are updated at each considered damage state of the structure through constrained minimization of an objective function.

Finite Element Model of Test Structure in FEDEASLab

A linear elastic FE model of the structure is developed using a general-purpose structural analysis program, FEDEASLab [11]. This FE model is defined by 35 nodes and 54 linear elastic shell and frame elements, as shown in Figure 4. A four-node linear elastic flat shell element (with four Gauss integration points) available in the literature [12, 13] and implemented in FEDEASLab by He [14] is used to model the infill walls. The members (beams and columns) of the RC frame are modeled with Bernoulli-Euler frame elements. The distributed mass properties of the structure are discretized into lumped translational masses applied at each node of the FE model. The initial FE model of the structure is based on the geometry of the physical specimen and the measured material properties. Figure 4 shows the FE model of the test structure in FEDEASLab with the six infill substructures used in model updating.

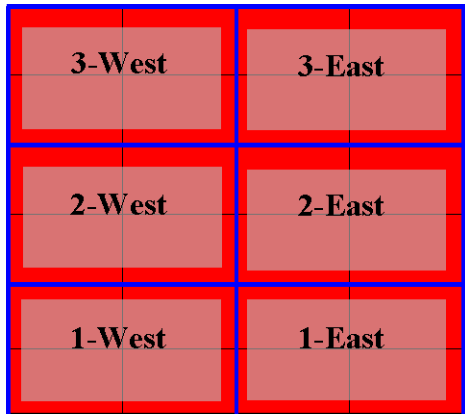


Fig. 4 FE model of the RC frame structure with the six infill substructures used in model updating

Objective Function

The objective function used for damage identification is defined as

$$f(\boldsymbol{\theta}) = \mathbf{r}(\boldsymbol{\theta})^T \mathbf{W}\, \mathbf{r}(\boldsymbol{\theta}) + (\mathbf{a}(\boldsymbol{\theta}) - \mathbf{a}^0)^T \mathbf{W}^a\, (\mathbf{a}(\boldsymbol{\theta}) - \mathbf{a}^0) = \sum_j [w_j r_j(\boldsymbol{\theta})^2] + \sum_k [w_k^a (a_k(\boldsymbol{\theta}) - a_k^0)^2] \quad (1)$$

where $\boldsymbol{\theta}$ = set of physical parameters (i.e., effective moduli of elasticity in the finite elements of the various substructures) which must be adjusted in order to minimize the objective function; $\mathbf{r}(\boldsymbol{\theta})$ = residual vector containing the differences between the FE computed and experimentally identified modal parameters; $\mathbf{a}(\boldsymbol{\theta})$ = vector of dimensionless damage factors representing the level of damage in each of the substructures of the FE model used for damage identification; $\mathbf{a}^0$ = vector of initial damage factors used as starting point in the optimization process. At each considered state of the building, $\mathbf{a}^0$ is selected as the vector of the identified damage factors at the previous state and $\mathbf{a}^0 = \mathbf{0}$ for the first damage state considered in the model updating procedure (which is S2). In Equation (1), $\mathbf{W}$ is a diagonal weighting matrix for modal residuals. Weight factors assigned to mode shape residuals are equal to the weight factor for that mode's natural frequency normalized/divided by the number of mode shape residuals for each mode. Weights of 1 and 0.5 are assigned to the modal residuals corresponding to the natural frequencies of the first and second longitudinal modes, respectively (i.e., $w_1 = 1, \quad w_2 = 0.5$). $\mathbf{W}^a$ is a weighting matrix for damage factors, a diagonal matrix with each diagonal coefficient determining the relative cost (or penalty) associated with the change of the corresponding damage factor. The weights for damage factors are used for regularization and can reduce the estimation error of the damage factors in the presence of estimation uncertainty in the modal parameters, especially for the substructures with updating parameters to which the employed residuals are less sensitive. In this study, these weights are set to $w_k^a = 0.02 \times w_1 = 0.02$, with $k = 1, \ldots, n_{sub}$ where n_{sub} denotes the number of substructures used in the FE model updating process. A combination of residuals in natural frequencies and mode shape components is used in the objective function, i.e.,

$$\mathbf{r}(\boldsymbol{\theta}) = \left[\mathbf{r}_f^T(\boldsymbol{\theta}) \quad \mathbf{r}_s^T(\boldsymbol{\theta}) \right]^T \quad (2)$$

in which $\mathbf{r}_f(\boldsymbol{\theta})$ and $\mathbf{r}_s(\boldsymbol{\theta})$ represent the eigen-frequency and mode shape residuals, respectively, defined as

$$\mathbf{r}_f(\boldsymbol{\theta}) = \left[\frac{\lambda_j(\boldsymbol{\theta}) - \tilde{\lambda}_j}{\tilde{\lambda}_j} \right], \quad \mathbf{r}_s(\boldsymbol{\theta}) = \left[\frac{\phi_j^l(\boldsymbol{\theta})}{\phi_j^r(\boldsymbol{\theta})} - \frac{\tilde{\phi}_j^l}{\tilde{\phi}_j^r} \right] \qquad (l \neq r),\ j \in \{1 \quad 2 \quad \cdots \quad N_m\} \quad (3)$$

where $\lambda_j(\boldsymbol{\theta})$ and $\tilde{\lambda}_j$ denote the FE predicted and experimentally identified eigenvalues for the jth vibration mode, i.e., $\lambda_j = \left(2\pi \cdot f_j\right)^2$, in which f_j is the natural frequency; $\boldsymbol{\phi}_j(\boldsymbol{\theta})$ and $\tilde{\boldsymbol{\phi}}_j$ denote the FE predicted and experimentally identified mode shape vectors, respectively. It should be noted that for each vibration mode, the mode shapes $\boldsymbol{\phi}_j(\boldsymbol{\theta})$ and $\tilde{\boldsymbol{\phi}}_j$ are normalized in a consistent way, i.e., scaled with respect to the same reference component. In

Eq. (3), the superscript r indicates the reference component of a mode shape vector (with respect to which the other components of the mode shape are normalized), the superscript l refers to the mode shape components that are used in the FE model updating process (i.e., at the locations and in the directions of the sensors), and N_m denotes the number of vibration modes considered in the damage identification process. In this study, the natural frequencies and mode shapes of the first two longitudinal vibration modes of the structure are used to form the residual vector $\mathbf{r}(\boldsymbol{\theta})$, which has a total of 34 residual components (based on 17 channels of acceleration response measurements) consisting of 2 natural frequencies and $2\times(17-1)=32$ mode shape component residuals, respectively. One mode shape component cannot be used since it is normalized to one.

In the process of FE model updating, the material stiffness (i.e., effective modulus of elasticity) of each of the damaged substructures is used as an updating parameter in the FE model of the building. Instead of using directly the absolute value of each updating parameter, a dimensionless damage factor is defined as

$$a_k^{Si} = \frac{E_k^{S0} - E_k^{Si}}{E_k^{S0}} \tag{4}$$

where E_k^{Si} is the effective modulus of elasticity of all finite elements in substructure k at damage state Si. Thus, the damage factor a_k^{Si} indicates directly the level of damage (i.e., relative change in effective modulus of elasticity) in substructure k when FE model updating is used for structural damage identification.

Optimization Algorithm

The optimization algorithm used to minimize the objective function defined in Eq. (1) is a standard Trust Region Newton method [15], which is a sensitivity-based iterative method available in the MATLAB optimization Toolbox. In this study, the damage factors were constrained to remain in the selected range [0% - 99%] at all states considered. The optimization process was performed using the “fmincon” MATLAB function, with the Jacobian matrix and a first-order estimate of the Hessian matrix calculated based on the analytical sensitivities of the modal residuals to the updating variables. The use of the analytical Jacobian, rather than the Jacobian estimated through finite difference calculations, increases significantly the efficiency of the computational minimization of the objective function.

Damage Identification Results

In this study, the FE model updating algorithm outlined above was implemented to identify the damage of the test structure. The residuals, $\mathbf{r}(\boldsymbol{\theta})$, used in the objective function were formed using the natural frequencies and mode shapes of the first two longitudinal vibration modes. The first step in the damage identification process consists of deriving a reference/baseline FE model of the undamaged structure (at state S0). In this step, the initial FE model is updated to match as closely as possible the identified modal parameters at the undamaged state of the structure by updating the stiffness (effective moduli of elasticity) of twelve substructures. The twelve substructures used in the calibration of the initial FE model to the reference FE model include six substructures for the infill walls (two per story as shown in Figure 4), and six substructures for the columns (two per story, one for the two outside columns and one for the inside column).The effective modulus of elasticity is assumed to be constant over each substructure.

The effective moduli of elasticity of the different substructures of the initial and the baseline model are reported in Table 3. The values for the initial model correspond to the measured values of the moduli of elasticity of concrete and masonry infill. These were obtained from uniaxial compression tests of concrete cylinders and masonry prisms, respectively, performed on November 10th (See Table 1). The masonry prism test was selected, since this is the closest representation of the masonry assembly. Moreover, a simplifying assumption is made in that the masonry infill is modeled as a homogeneous isotropic material. During the calibration of the initial FE model to the reference FE model, the “damage factors” were constrained in the range [-2,   0.9]. Once the reference model is obtained, the moduli of elasticity of the six infill walls shown in Figure 4 are updated from the reference FE model (at the undamaged/baseline state S0) to states S2, S3, S4, S5, S6 and S7. The stiffness parameters of the elements representing the RC frame members are kept fixed at their values in the reference FE model. This assumes that the concrete frame remains elastic, which is a good approximation for the early damage states based on the inspection of the physical specimen. However, at damage states S5 to S7, this assumption is not accurate. Due to the very small change in the modal parameters from S0 to S1, FE model updating was not performed at state S1.

Table 3. Effective moduli of elasticity of structural components at different substructures for initial and reference FE models

Substructure	Effective Moduli of Elasticity [ksi]	
	Initial FE model	*Reference FE model (S0)*
Infill 1-West	785	687
Infill 1-East	785	761
Infill 2-West	777	936
Infill 2-East	777	816
Infill 3-West	946	963
Infill 3-East	946	908
Column 1-Outside	2380	2424
Column 1-Intside	2380	2389
Column 2-Outside	2530	1987
Column 2-Intside	2530	2539
Column 3-Outside	2463	2251
Column 3-Intside	2463	2465

In updating the FE model at each considered damage state of the infilled frame, the k^{th} dimensionless damage factor was constrained to be in the range $[\mathbf{a}_{Si}^{0} - 0.1, \quad 0.95]$. At each damage state considered, the vector of initial damage factors $\mathbf{a}_{Si}^{0}$, used as starting point in the optimization process, was selected as the damage factors identified at the previous damage state or zero for state S2. The damage factors (relative to the reference FE model or reference state) obtained at different damage states are presented in a bar plot in Figure 5. These results indicate that the severity of structural damage increases as the structure is exposed to stronger earthquake excitations; the extent of damage decreases rapidly along the height of the structure (damage is concentrated in the bottom story) and it is uniform at each story (East and West side infills have similar damage factors). Table 4 presents the natural frequencies computed from the updated FE model at each state considered together with their counterparts identified from white noise base excitation test data as well as the MAC values between FE predicted and experimentally identified mode shapes. To compare the FE predicted and experimental mode shapes, the former were truncated to include only the degrees of freedom corresponding to the locations and directions of the accelerometers. From Table 4, it is observed that the FE predicted natural frequencies and mode shapes for the first two longitudinal modes match well their experimentally identified counterparts. Moreover, the MAC values between FE predicted and experimentally identified mode shapes are very close to unity at all damage states. MAC values are in general higher for the first mode than for the second mode. This can be expected since smaller weight factors are assigned to the residuals corresponding to the second mode shape.

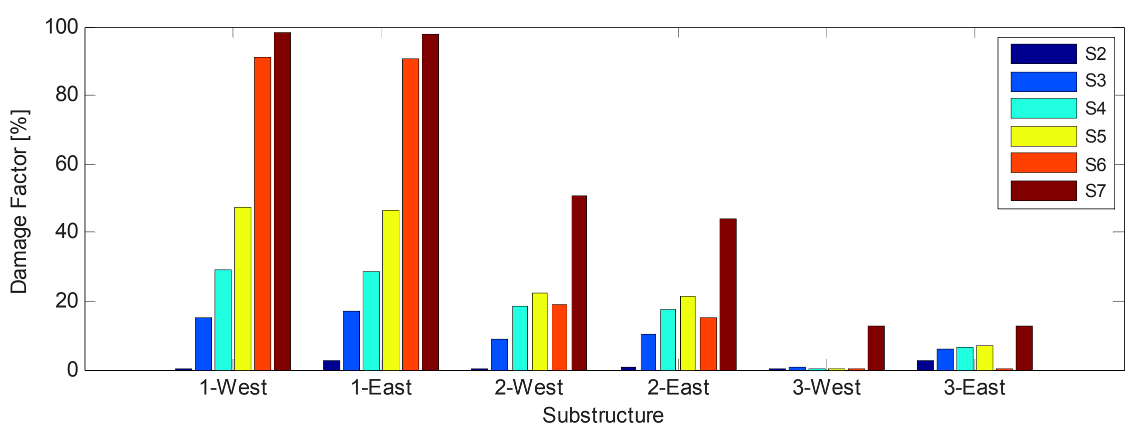


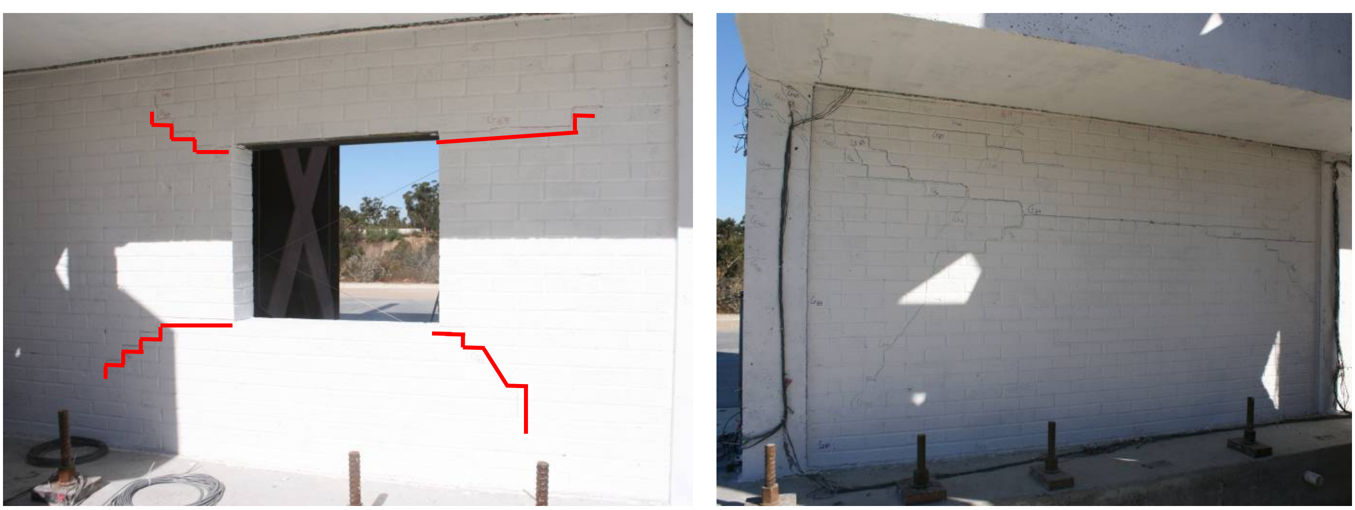
Fig. 5 Identified damage factors at various substructures

Table 4. Comparison of FE computed and experimentally identified modal parameters

Damage State	Experimentally Identified Natural Frequencies [Hz]		FE Computed Natural Frequencies [Hz]		MAC	
	1-L mode	2-L mode	1-L mode	2-L mode	1-L mode	2-L mode
S0 (baseline)	18.18	41.22	17.86	42.59	0.99	0.96
S2	17.99	41.56	17.80	42.41	1.00	0.95
S3	16.74	40.21	16.92	40.25	0.99	0.97
S4	15.93	38.56	16.09	38.29	1.00	0.99
S5	14.78	35.5	14.86	35.37	1.00	0.99
S6	8.47	27.34	8.50	26.40	1.00	0.99
S7	5.34	22.57	5.34	23.48	0.99	0.99

Comparison of Damage Identification Results with Observed Damage

The structure practically remained elastic after the low level earthquakes (20% and 40% Gilroy) and the first noticeable deterioration of the structure occurred during the 67% of Gilroy, which corresponds to a Design Basis Earthquake (DBE) for this structure. At damage state S3, inspection of the physical specimen revealed the first cracks in the structure in the infill panel with the window at the first story, shown in Figure 6(a). However, the cracks were insignificant for the structural integrity of the specimen and did not alter the structural properties significantly. This observation is consistent with the damage identification results according to which no significant damage is identified at damage state S2, while 15-17% loss of stiffness is identified in the first story infill walls at damage state S3 (see Figure 5). Significant change in the structural properties is noted at damage state S6, which occurred after subjecting the structure to a Maximum Considered Earthquake (MCE). At this stage, significant cracks developed in both bays of the first floor and affected the columns which also cracked as illustrated in Figure 6(b). These cracks caused a natural frequency reduction of more than 50%, which translated to a 90% reduction of the stiffness at the first story infills (see Figure 5). The damage in the infill is exaggerated in the finite element model since it represents any damage occurring to non-updating components of RC frame such as the columns. Significant damage was induced in the specimen by the 120% of Gilroy, which included the first dominant shear cracks that developed in the middle column. These cracks further reduced the first mode frequency to less than one third of its initial value. Figure 7 shows the observed damage at this damage state. At this damage state, damage factors of 98% were identified for the two infills at the first story and 44-50% at the second story. These estimates of damage are very high and are not consistent with the experimental evidence. However, it should be noted that any damage to non-updating components of the structure (e.g., columns) will be reflected in the effective moduli of elasticity of infills.



(a) (b)

Fig. 6 Crack pattern at damage states S3 (a) and S6 (b)


Fig. 7 Crack pattern at damage states S6 in the solid infill and the middle column (left), and the infill with opening and the east column (right)

Conclusions

In this study, a FE model updating strategy is applied for vibration-based damage identification of a 2/3-scale, three-story, two-bay, infilled RC frame tested on the UCSD-NEES shake table. The objective function for damage identification is defined as a combination of natural frequency and mode shape residuals measuring the discrepancy between the numerically predicted (using a FE model) and experimentally identified modal parameters. The FE model of the structure is first calibrated through the model updating technique for the reference/baseline (undamaged) state, and then damage identification is performed at six damage states of the structure. These states correspond to the state of the physical specimen after being subjected to earthquake excitations of increasing intensity.

The obtained damage identification results are consistent with the actual damage observed in the structure, which showed a concentration of damage at the bottom story. The analytical modal parameters obtained from the updated FE models are in good agreement with their experimentally identified counterparts, which indicates the accuracy of the updated models. Finally, it should be noted that the success of vibration-based damage identification depends significantly on the accuracy and completeness of the identified modal parameters. Clearly, if estimation uncertainty of the modal parameters is larger than their changes due to damage, it is impossible to resolve/identify the actual damage in the structure.

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