Experimental Validation of Dynamic Nonlinear FE Model of Full-Scale Five-Story Reinforced Concrete Building

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ABSTRACT: A full-scale reinforced concrete building specimen, furnished with a variety of nonstructural components and systems (NCSs), was built and tested on the NEES (Network for Earthquake Engineering Simulation)-UCSD (University of California, San Diego) outdoor shake table in 2011-2012. The building specimen was subjected to a sequence of dynamic tests including scaled and unscaled historical earthquake ground motions. In order to simulate and predict the nonlinear dynamic response of the building specimen, a detailed three-dimensional nonlinear finite element (FE) model of the structure was developed utilizing the FE analysis software DIANA and used for pre-test response simulation as well as for guidance regarding the input ground motions used for the shake table tests. The pre-test FE model was compared with the experimental results and important sources of discrepancies were identified. Some important sources of discrepancies are the kinematic interaction existing between the structure and some NCSs and the pinching behavior exhibited by RC structural components.

KEY WORDS: Nonlinear finite element response simulation, reinforced concrete structure, pre-test simulation, full-scale shake table test.

1 INTRODUCTION

In 2011-2012, a landmark research project was completed at the University of California, San Diego (UCSD), which consisted of testing a full-scale five-story reinforced concrete (RC) building specimen (Figure 1) on the NEES (Network for Earthquake Engineering Simulation)-UCSD large high performance outdoor shake table (LHPOST) [1-2]. The building was outfitted with a variety of nonstructural components and systems (NCSs), including a fully functional elevator, metal stair subsystem, complete exterior facade, interior partition walls, ceiling sub-system, piping, and various roof mounted equipment, to name a few. Different architectural occupancies including home office, laboratory environment, computer server room, Intensive Care Unit (ICU), and surgery unit were designated at each level of the building. The main objectives of the project were to study the dynamic performance of the full-scale RC building including the NCSs under seismic excitations, to investigate the interaction between the structure and NCSs, and to contribute to the development of performance-based design methodologies for NCSs. The building was first tested in a base-isolated configuration with the foundation resting on four elastomeric bearings. After completion of the base-isolated tests, the base isolators were removed, the building specimen was fixed to the shake table platen and the building with its nonstructural components and systems (BNCS) was tested under fixed-base condition.

Pre-test simulations in the form of advanced nonlinear finite element (FE) analyses were conducted, using the as-built specifications of the test specimen. The main goals of the pre-test FE simulations were to: predict the response of the test specimen, estimate the seismic demands on the structural and NCSs, support the design of the seismic test protocol, and guide the selection of the instrumentation layout for both the structure and NCSs. Both the computational cost and accuracy were important aspects of the FE simulation framework. This paper describes the pre-test simulation efforts for the fixed-base building specimen. First, the building specimen and testing protocol are presented briefly. Then, the FE model developed is described in some details. Finally, the prediction capability of the pre-test FE model is evaluated by comparing FE predicted building response quantities with their experimental counterparts.

2 DESCRIPTION OF TEST SPECIMEN AND OVERVIEW OF SHAKE TABLE TESTS

2.1 Structural and non-structural systems

The building structural skeleton consisted of a two-bay by one-bay cast-in-place RC frame. It was 11.0 m by 6.6 m in plan and had a floor to floor height of about 4.3 m resulting in a total height of 21.3 m from the top of the foundation to the top of the roof slab. Structural details of the test specimen are shown in Figure 2. The test specimen had a 1.5 m thick posttensioned RC foundation designed to remain linear elastic during all seismic tests. During the first phase of the seismic testing, the building was base isolated, resting on four high damping rubber isolators inserted between the shake table platen and the foundation at the four corners. Subsequently, the building was anchored to the shake table through posttensioned rods installed along the foundation perimeter, providing the fixed-base test configuration during the second phase of the seismic testing.

The test specimen had six identical 0.66 m by 0.46 m RC columns reinforced with 4 #9 (A = 645 mm² per rebar) and 6...
The moment resisting frame beams had different details at different floors. The beams on the first two floors (levels 2 and 3) were reinforced with high strength steel with nominal yield strength of 827 MPa. Level 4 had a special moment frame (SMF) system with hybrid upturned beams post-tensioned after concrete pouring through a set of four unbonded tendons at the center of the beam cross-section. The upturned beams were connected to the columns at both ends via Dywidag ductile rod connectors (DDCs). DDCs were also used at level 5 in combination with conventional frame beams. At the roof level, the moment frame system was conventional.

The floor system of the building specimen consisted of a 0.2 m thick concrete flat slab doubly reinforced at top and bottom. The perimeter of the flat-slabs strengthen with band beams (integral beams), which framed into the intermediate and west columns (see Figure 2). A pair of 0.15 m thick concrete walls, reinforced with a single grid of reinforcement in their middle plane, was placed in the north-south direction on either side of the elevator shaft to provide gravity support for the elevator system and additional torsional stiffness for the building.

More than 600 sensors including accelerometers, linear and string potentiometers, strain gauges, load cells, GPS receivers, and digital cameras were deployed on the test specimen to record the response of various structural and nonstructural components. The main accelerometer array, measuring the response of the structure, consisted of four tri-axial EpiSensor accelerometers installed at the four corners of each floor slab and of the foundation surface.

The fixed-base building specimen was subjected to a sequence of dynamic tests including six earthquake base motions of increasing intensity. Table 1 shows the sequence of seismic tests performed on the building specimen. The FE model developed prior to the start of the shake table tests was rerun using the base input motions achieved by the shake table, which differ from the target earthquake motions due to the imperfect nature of the shake table controller. However, the FE modeling assumptions and parameters, as described in the following sections, were not revised to preserve a true comparison between the pre-test simulation and measurement results. The acceleration time histories of the east-west horizontal component of the achieved motions measured on top of the foundation for FB-1: CNP100 to FB-5: DEN67 are shown in Figure 3.

Table 1: Seismic tests performed in the fixed-base configuration of the building.

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Seed Motion</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>FB-1: CNP100</td>
<td>Canoga Park - 1994</td>
<td>Spectrally matched serviceability level</td>
</tr>
<tr>
<td>FB-2: LAC100</td>
<td>Northridge earthquake</td>
<td>Spectrally matched serviceability level</td>
</tr>
<tr>
<td>FB-3: ICA50</td>
<td>ICA - 2007 Pisco (Peru) earthquake</td>
<td>Original earthquake record, 50% scale</td>
</tr>
<tr>
<td>FB-4: ICA100</td>
<td>ICA - 2007 Pisco (Peru) earthquake</td>
<td>Original earthquake record, 100% scale</td>
</tr>
<tr>
<td>FB-5: DEN67</td>
<td>TAPS Pump Station 9 - 2002 Denali earthquake</td>
<td>Spectrally matched earthquake, 67% scale (targeted design level)</td>
</tr>
<tr>
<td>FB-6: DEN100</td>
<td>TAPS Pump Station 9 - 2002 Denali earthquake</td>
<td>Spectrally matched earthquake, 100% scale</td>
</tr>
</tbody>
</table>

NOTE: With the exception of FB-3 and FB-4, the earthquake ground motions listed are seed motions that were spectrally matched and amplitude scaled as noted in the third column.

3 NONLINEAR FE MODELING AND ANALYSIS

Considering the specific configuration of the test specimen and its plan dimensions, the slabs were expected to have a dominant influence on the earthquake resistance of the structure. The large stairwell opening in the south-east portion of the slabs close to the frame beams (see Figure 2) was expected to complicate the interaction of the floor slab with the south frame of the structure. The elevator opening in the north-west portion of the slabs of the building was expected to perturb the load transfer mechanism in the diaphragm. Furthermore, the flat slab-column connections were expected to influence the nonlinear response behavior of the building.
Therefore, proper modeling of the RC slabs was an important aspect that needed special consideration in the FE modeling of the test specimen. To satisfy these modeling requirements, the DIANA finite element analysis software [3] was selected as the FE simulation platform. This software supports state-of-the-art nonlinear material constitutive models dedicated to the modeling of RC structures. Moreover, these material models can be used in conjunction with the structural type finite elements needed such as 3D beam-column elements and plate/shell elements with embedded steel reinforcement.

3.1 Material constitutive models

The employed concrete constitutive model is a total strain rotating smeared crack model with a pseudo-elastic concrete material law. The constitutive model for the uniaxial tensile behavior of concrete consists of an initial linear elastic region (denoted 1 in Figure 4) followed by a nonlinear softening region (denoted 2 in Figure 4) according to the Reinhardt exponential softening model [4] as shown in Figure 4. In this figure, \( f'_{cc} \) denotes the concrete linear elastic modulus defined as \( E_c = \frac{5 f'_{cc}}{3} \), where \( f'_{cc} \) = confined compressive strength and \( \varepsilon_{cc} \) = peak confined compressive strain. The concrete tensile strength is assumed as \( f'_t = 0.33 \sqrt{f'_{cc}} \) (MPa), in which \( f'_t \) = unconfined compressive strength of concrete. The peak tensile strain – or cracking strain – is denoted as \( \varepsilon_{tu} = \frac{f'_t}{E_c} \), and the ultimate tensile strain is \( \varepsilon_{tu} \). The concrete material fails in tension upon reaching \( \varepsilon_{tu} \), i.e., when this strain is reached, both the concrete tensile stress and stiffness vanish permanently. A fracture energy concept is implemented to handle the spurious mesh sensitivity caused by tensile softening [5].

The constitutive model for the uniaxial compressive behavior of concrete consists of an initial linear elastic branch followed by two parabolic branches as shown in Figure 4. The values of \( f'_{cc} \), \( \varepsilon_{cc} \), and \( \varepsilon_{tu} \), the ultimate confined compressive strain, are computed based on the initial – undamaged – properties of the concrete material and the confinement effect of the transverse reinforcements [6]. To incorporate the confinement effect, \( f'_{cc} \) and \( \varepsilon_{cc} \) are determined as the average prediction from different confined concrete models such as the modified Kent and Park [7] and Mander model [6]. The parameter \( \varepsilon_{cu} \) is determined following suggestion of Scott et al. [8]. To regularize the localization in compressive failure of concrete, the softening branch of the compressive stress-strain curve for each finite element is defined as a function of the fracture energy in compression [9]. \( G_c \), and the element size, \( h \), such that the total post peak dissipated energy (per unit of cross-section area) is independent of the element size.

The Modified Compression Field Theory (MCFT) [10] is employed as a built-in subroutine in DIANA to continually update the stress-strain relation for the concrete in compression based on the maximum transverse tensile strain [11]. “Model B” as proposed by Vecchio and Collins [12] guides the reduction in the compressive strength of concrete in one principal direction due to the presence of tensile cracking in the other principal directions.

As illustrated in Figure 4, the unloading and reloading in tension and compression are modeled via a linear secant path passing through the origin. This is clearly a simplifying assumption since, in reality, residual strains appear upon stress removal (unloading) both from a compressive or tensile state. Nevertheless, this assumption assists with providing numerical stability to FE numerical algorithms built around this material model [13].
hysteretic stress-strain behavior of the reinforcing steel. The parameters of the constitutive model (e.g., modulus of elasticity, yield stress/strain, and strain hardening ratio) are obtained from tensile tests performed on representative rebar samples.

### 3.2 Finite elements

The beam and column members are modeled using three-node Mindlin-Reissner displacement-based 3D beam-column elements with 6 DOFs per node (i.e., 18 DOFs per element). This element formulation is based on (i) quadratic shape functions that imply a linear variation of the axial strain and the curvature along the element, and (ii) constant transverse shear strain over the cross-section [15]. For the sake of computational efficiency, different integration schemes for the beam-column elements – along their length and over their cross-section – are defined based on their expected level of nonlinearity (See Table 2 and Figure 5). The columns are restrained assuming full fixity at their base because of the large stiffness of the foundation, which was anchored to the shake table during the fixed-base tests. The beam and column longitudinal steel reinforcements are modeled as fully bonded embedded steel bars.

<table>
<thead>
<tr>
<th>Component</th>
<th>Location</th>
<th>Integration scheme</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams</td>
<td>PHs at levels 1, 2, 3, and 4</td>
<td>ISB1: 3 (length), 3 (width), 7 (depth)</td>
</tr>
<tr>
<td></td>
<td>PHs at level 5 and roof</td>
<td>ISB2: 3 (length), 3 (width), 5 (depth)</td>
</tr>
<tr>
<td></td>
<td>Others</td>
<td>ISB3: 3 (length), 3 (width), 3 (depth)</td>
</tr>
<tr>
<td>Columns</td>
<td>PHs at level 1-2</td>
<td>ISC1: 3 (length), 3 (width), 7 (depth)</td>
</tr>
<tr>
<td></td>
<td>PHs at levels 2-4</td>
<td>ISC2: 3 (length), 3 (width), 5 (depth)</td>
</tr>
<tr>
<td></td>
<td>Others</td>
<td>ISC3: 3 (length), 3 (width), 3 (depth)</td>
</tr>
</tbody>
</table>

In this table, PHs: plastic hinges, ISB: integration scheme for beam, ISC: integration scheme for column.

The damping characteristics of the building specimen are modeled using the Rayleigh proportional damping model [16] by defining 2% damping at the first mode and at 20 Hz. The Rayleigh damping parameters are derived based on the initial/uncracked linear elastic stiffness matrix of the FE building model and are kept constant during the time history analyses, but with the stiffness coefficient applied to the tangent stiffness matrix.

The self-mass of the beams, columns, and shear walls is modeled by assigning the proper material mass density to the corresponding elements. Since the individual mass contributions of the interior partition walls, ceilings, balloon frame facade at levels 2 to 4, and the installed contents on each floor are not significant, their inertial effects are modeled as a uniform distributed mass added over each floor slab. On the other hand, the masses of the precast concrete claddings, stairs, elevator, roof penthouse, and cooling tower (at the roof level) are significant; therefore, they were each modeled with a set of lumped masses (See Figure 5).

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4 NONLINEAR DYNAMIC TIME HISTORY ANALYSIS

Each nonlinear analysis starts by applying the gravity loads quasi-statically and incrementally. The regular incremental-iterative Newton method is used to solve the nonlinear static equilibrium equations for gravity loads. The nonlinear time history analysis for earthquake base excitation is then performed from the state of the structure after application of the gravity loads. Newmark average acceleration method [16], with a constant time step of 0.025 sec, is used to integrate the equations of motion in time. The constant time step of 0.025 sec is selected based on a preliminary convergence study with respect to the integration time step size to ensure the accuracy of the results. The quasi-Newton (secant) method based on the Broyden–Fletcher–Goldfarb–Shanno (BFGS) stiffness update method [17] is employed as the iterative method to solve the nonlinear dynamic equilibrium equations. At the end of each time step of analysis, the last obtained secant stiffness matrix is stored and used as the initial stiffness matrix at the first iteration of the next time step. The convergence criterion is based on the relative norm of the last displacement increment vector or the relative norm of the last unbalanced force vector with a convergence tolerance of $10^{-4}$, whichever occurs first, while the number of iterations per time step is limited to 30. If none of the two convergence criteria are satisfied within 30 iterations, the iterative procedure at that time step is terminated, the current unbalanced force vector is transferred to the next time step, and the analysis goes on.

4.1 Structural response at global level

Figure 6 compares the FE predictions and experimental measurements from the shake table tests for the peak floor absolute accelerations (PFAAs) and the peak interstory drift ratios (PIDRs) for each of the five floors or stories. In this figure, the coefficient of determination ($R^2$) measures the level of agreement between the FE predictions and measurements for the peak values of the considered response parameters at each floor level or story for the five different seismic tests.

In Figure 7(a) and (b), the envelope plots compare the FE predicted and measured PFAA and PIDR, respectively, along the height of the building – positive and negative peaks are treated separately. To compare the relative difference between the FE predicted and measured peak response parameters of the building in a more comprehensive and concise way, a relative error measure ($E_i$) is defined, which expresses the relative closeness of the FE predicted and measured peak response value at the $i^{th}$ level – or story – of the building. Denoting a peak response quantity obtained from FE prediction and experimental measurements at the $i^{th}$ level – or story – by $R_{i, FE}^R$ and $R_{i, Exp}^R$, in the positive direction, and by $R_{i, FE}^-R_{i, Exp}^-$ in the negative direction, respectively, the relative error measure, $E_i$, is defined as

$$E_i = \max \left\{ \frac{R_{i, FE}^R - R_{i, Exp}^R}{\max\left\{R_{i, Exp}^R\right\}}, \frac{R_{i, Exp}^- - R_{i, FE}^-}{\min\left\{R_{i, Exp}^-\right\}} \right\}$$

The value $E_i = 0$ indicates a perfect agreement at the $i^{th}$ level – or story – between FE prediction and experimental measurement for the peak response of interest. The more $E_i$ deviates from zero, the larger the error in the FE prediction. The distribution of this relative prediction error is shown by contour plots over the five seismic tests and the five floor levels (or stories) in Figure 8 for the PFAA and PIDR. By comparing parts (a) and (b) of Figure 6 to Figure 8, it is concluded that the PFAAs are generally better predicted than the PIDRs. The PFAAs are better predicted at upper floors for low intensity seismic tests and at lower floors for high intensity seismic tests, see Figure 8(a). Figure 7(b) clearly shows that the FE response simulation generally overpredicts the PIDRs in the low intensity seismic tests and underpredicts them in high intensity seismic tests. For low intensity tests, the PIDR is typically overestimated at all stories, although more so at the middle stories of the building, while for high intensity tests, the PIDR is underestimated at the lower stories and overestimated at the upper stories of the building. Figure 8(b) shows that the PIDR relative prediction error is more significant for the low intensity seismic tests (FB-1: CNP100 to FB-3: ICA50) and at the middle stories of the building (story 3).

$$P^2=94.6\%$$

$$R^2=68.1\%$$

Figure 6: Correlation of the FE predicted versus measured (a) peak floor absolute acceleration and (b) peak interstory drift ratio for each of the 5 floors or stories.

Figure 7: Comparison of the FE predicted and measured (a) peak floor absolute acceleration envelopes and (b) peak interstory drift ratio envelopes; see Fig. 6 for definition of symbols.
It is hypothesized that the nonstructural components, especially the partition walls, contribute significantly to the discrepancy between the FE predicted and measured PIDR results. The influence of partition walls on the earthquake response of buildings has been the focus of previous studies (e.g., [18]), in which it has been noted that the partition walls' effect is more significant at low IDR, since the walls are intact, well connected to the structure, and contribute their full original stiffness. As the base motion intensifies, the partition walls undergo damage and their connections to the building degrade at the lower stories of the building, where the IDR demand is usually higher, while at the upper stories, where the IDR demand is lower, they are still influential.

Figure 8(b) shows that the PIDR relative prediction error is the largest at story 3 for the low intensity seismic tests (FB-1: CNP100 to FB-3: ICA50). This can be explained by comparing the total length of the installed partition walls at each story of the building, as reported in Table 3. In this table, the total length of the exterior balloon framings are reported separately, since their connection details are different from those of the partition walls to reduce their interaction with the structural system. Table 3 indicates that stories 3 holds approximately 50% more partition walls than the other stories. This explains why the PIDR relative prediction error is the largest at story 3.

Table 3: Length of partition walls and balloon framing at different stories (units: meters).

<table>
<thead>
<tr>
<th>Story</th>
<th>Partition walls in E-W direction</th>
<th>Partition walls in N-S direction</th>
<th>Balloon framing in both directions</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>13.5</td>
<td>6.0</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>15.0</td>
<td>7.5</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>22.0</td>
<td>15.5</td>
<td>31.5</td>
</tr>
<tr>
<td>2</td>
<td>13.5</td>
<td>11.0</td>
<td>34.0</td>
</tr>
<tr>
<td>1</td>
<td>13.5</td>
<td>3.5</td>
<td>21.0</td>
</tr>
</tbody>
</table>

The reason for the underprediction of the PIDR at the lower stories of the building during large intensity tests (see Figure 7(b) for FB-4: ICA100 and FB-5: DEN67) can be explained by the difference between the actual and FE predicted hysteretic response of the structural components. The employed FE modeling technique is unable to capture the pinching hysteretic behavior in the beams and columns. As a result, the structural components have larger energy dissipation capacity under cyclic loading in the FE model than in the real structure at the same level of ductility in the response. Consequently, the FE predicted floor displacement responses are lower than the corresponding test results at the lower stories of the building, where the structural response has sever nonlinearity. Figure 7(b) shows that the PIDR is underpredicted at the first two stories for FB-4: ICA100 and FB-5: DEN67. This explanation is in agreement with the discussion on the response of the building at the story level in the next section.

To better investigate the effect of nonstructural components on the seismic response of the building, the time histories of selected response parameters are compared in Figure 9 for FB-1: CNP100 and FB-5: DEN67. This figure shows the time histories of (i) the roof drift ratio, which is the roof displacement relative to the base normalized by the roof height measured from the top of the foundation and (ii) the second story IDR. Figure 9(a) shows a clear difference in the predominant period of the two response time histories between FE prediction and experimental measurement, which is most likely due to the influence of the nonstructural components – mostly partition walls – on the building response.

This influence, which is not accounted for in the FE model of the building, results in a shortening of the natural periods of the building, and therefore of the predominant period of the
response. By comparing Figure 9(a) and 9(b), it is observed that the discrepancy in the predominant period of the response between FE prediction and experimental measurement vanishes for the large intensity seismic test FB-5: DEN67, since the partition walls suffered extensive physical damage in the seismic tests prior to FB-5. As a result, the nonstructural components’ contribution to the building response is significantly reduced for FB-5: DEN67.

4.2 Structural response at the story level

The total (inertial) story shear (TSS) is computed as

\[ V_i = \sum_{j=1}^{n} m_j a_{ij} \]

where \( V_i \) is the story shear evaluated at the building horizontal at mid-height between the \( j^\text{th} \) and \( (i+1)^\text{th} \) levels (i.e., total shear force within the \( j^\text{th} \) story), \( a_{ij} \) is the averaged (over the four floor corners) absolute floor acceleration at the \( j^\text{th} \) level in the east-west direction, and \( m_j \) denotes the estimated tributary mass of the \( j^\text{th} \) level. The TSSs are normalized by the estimated total building weight above the foundation, \( W \). The plots of the TSSs versus the corresponding IDRs characterize the hysteretic response of the building at the individual story levels. These plots are shown in Figure 10 for FB-1: CNP100, FB-4: ICA100, and FB-5: DEN67, respectively. Figure 10(a) shows that the FE prediction is in reasonable agreement with the measured response only at the first story for FB-1: CNP100, since less partition walls were installed in this story (see Table 3). For all other stories, the measured response is significantly stiffer than the predicted one. This can be explained by the contribution of the nonstructural components – mostly partition walls. As the base excitation intensifies from FB-1: CNP100 to FB-4: ICA100 and to FB-5: DEN67, the nonstructural components suffer increasing damage and their influence on the structural system response decreases, leading to better agreement in the stiffness between the FE predicted and measured responses at the upper stories. The FE prediction underestimates the IDR demands at the lower stories during FB-4: ICA100 and FB-5: DEN67 tests. The measured story responses show a higher level of nonlinearity than the corresponding FE predictions. The difference between the FE predicted and measured responses at the lower stories during the high intensity seismic tests can also be a consequence of improper modeling of the (linear viscous) damping properties (defined in addition to the hysteretic energy dissipation through inelastic action of the materials) in the FE model, which may result in high viscous damping energy dissipation and low hysteretic energy dissipation.

5 CONCLUSIONS

This paper presents the 3D nonlinear finite element (FE) model utilized for the pre-test numerical simulation of a full scale five-story reinforced concrete (RC) building specimen tested on the NEES-UCSD shake table. To investigate the likely shortcomings of the FE modeling technique used, selected FE predicted response parameters (at the global structure level and at the story level) are compared with the corresponding experimental measurements at different response levels.

In this model, only the mass (assumed as rigid) of the nonstructural components is accounted for in the FE model of the structure. The stiffness and strength properties of the nonstructural components are not modeled and therefore the full dynamic (especially kinematic) interaction between nonstructural components and the structural system is not modeled.

Comparison of the FE predicted and measured responses of the structure during the shake table tests shows that the nonstructural components have significant influence on the seismic response of the building. At low intensity earthquake excitations, when the nonstructural components are intact, the kinematic interaction between the structural and nonstructural systems increases significantly the overall stiffness of the building, which reduces the floor displacement and interstory drift response of the building (compared to the corresponding FE predictions) leading to some discrepancy between FE predicted and measured response. As the intensity of the earthquake increases, the nonstructural components get progressively damaged, i.e., undergo stiffness and strength degradation, and their influence on the stiffness and strength of the structural system diminishes. Then, the FE predictions become in better agreement with the experimental results.

In the FE model developed, some aspects of RC cyclic response behavior are not captured correctly, the most significant of which is the pinching hysteretic behavior of RC flexural members. Thus, in highly nonlinear regions of the structure, the FE model of the test structure has higher hysteretic energy dissipation capacity for flexural members.
than the actual specimen. Consequently, for the high intensity earthquakes, the FE model underestimates the floor displacement and therefore interstory drift demand of the building at the lower levels. Thus, improper modeling of the cyclic hysteretic response of the concrete and flexural structural members and the resulting hysteretic energy dissipation can be another source of discrepancy between the FE predictions and experimental results.

These observations and findings point to the direction of future research needs to improve the fidelity of nonlinear FE structural models for RC building structures. In the current research program, efforts are underway to update the pre-test FE model presented here by incorporating appropriate models for the nonstructural components. Furthermore, the material model for steel reinforcement is being modified to better capture some key features (e.g., pinching behavior) of RC components and structures.

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