SEISMIC RESPONSE HISTORY ANALYSIS FOR THE NEXT GENERATION OF BUILDINGS

Rafael A. SALGADO¹, Serhan GUNER²

ABSTRACT

The performance-based earthquake engineering (PBEE) has been increasingly employed as a comprehensive dynamic analysis methodology for the next generation of buildings. To assess the performance of a given structure subjected to earthquake loading, the nonlinear dynamic analysis (NLDA) is considered one of the most accurate methods to analyze a numerical model up to the failure conditions. The complexity of a nonlinear analysis and its computationally-demanding nature, however, hinder the suitability of this analysis for the PBEE. In this study, the applicability of three numerical modeling approaches for reinforced concrete frame buildings is evaluated in a performance-based NLDA process. The simulation accuracy of each model is assessed by comparing the numerical predictions with experimentally-observed shake-table test results. The PBEE is conducted through a set of ground-motion vs. time histories to develop fragility curves and evaluate the probabilities of exceedance for each model. The computational effort required, numerical stability characteristics, prediction accuracy, and the calculated probabilities are discussed to assess each numerical model. Structural modeling guidelines are proposed to help practitioners to efficiently create numerical model for the performance-based design. Research results indicate that the nonlinear models provide significantly more accurate response predictions than the linear-elastic model with concentrated hinges, and that the total computational time demand required by a numerical model is composed of three phases that far exceeds the required analysis time. It was also found that the nonlinear models with different levels of material model comprehensiveness calculated similar structural performance probabilities but required significantly different total computational time and had different levels of general applicability.

Keywords: nonlinear analysis; performance-based earthquake engineering; dynamic analysis; shear-critical structures; computational demand

1. INTRODUCTION

Performance-based earthquake engineering (PBEE) uses nonlinear structural analysis methods to accurately predict the inelastic response of buildings during seismic excitations. The nonlinear dynamic analysis (NDA) method is known to provide the most realistic simulation of structural behavior (Chen et al. 2014, Kalkan and Kunnath 2007) and has been widely studied for the analysis of the next generation of buildings. This analysis method, however, requires multiple NLDA to assess (or design) a structure using PBEE and employs methods that are computationally-intensive, which significantly limits its applicability in practical situations.

Previous studies have either focused on proposing simplified analysis procedures (Ghaﬀarzadeh et al. 2013, Goel et al. 2010, Liao and Goel 2014) to substitute the need for the NLDA method, or on evaluating the influence of local element assumptions and modeling approaches on the overall structural response (Celik and Ellingwood 2009, Elwood and Moehle 2008). There is still a lack of studies that investigate the numerical response accuracy and reliability when a structural system is analyzed with different modeling techniques. In this study, various numerical modeling techniques with different complexity levels are created and their simulation accuracy and computational demand characteristics are evaluated. For this purpose, a PBEE structural assessment of a previously-tested RC frame is conducted using three modeling approaches, and the calculated structural risk to a set of performance limits is evaluated by means of fragility curves.

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2. PERFORMANCE-BASED EARTHQUAKE ENGINEERING

PBEE requires performing a set of analysis stages to obtain a comprehensive assessment of the structural behavior (Zareian and Krawinkler 2006, FEMA 2012). First, the building location, importance, and soil conditions are used to determine the earthquake hazard level and the response spectrum, as per the applicable building codes. Structural analysis is then conducted using a numerical model subjected to a series of ground motions (GM) acceleration histories that match the response spectrum. The performance is evaluated based on the calculated responses and the structural risk is expressed by means of fragility curves. These curves indicate the probability of a structure to exceed a certain damage state (i.e., damage measures or performance levels) based on the engineering demand parameters (EDP) (e.g., story drift, floor accelerations, or velocities) calculated by the structural analysis. A loss analysis is finally conducted, based on the previously calculated probability of exceedance, to quantify the financial, downtime, casualty, or other types of losses.

3. HAZARD DETERMINATION

The structure examined in this study was considered to be located in Portland, Oregon, USA, and constructed over ‘type D’ soil, which is the standard soil type in ASCE 7 (2013) when no sufficient detail is available. The design response spectrum was calculated, and seven acceleration histories were considered to meet the requirements of the NEHRP (2015) provisions. The ground motion characteristics included: ‘strike-slip’ fault type, less than 50 km to the epicenter, and Richter magnitude between 6 and 8 (see Table 1). Time histories were obtained from the Pacific Earthquake Engineering Research (PEER) online NGA-West2 database (PEER 2017).

Table 1. Selected ground motion characteristics.

<table>
<thead>
<tr>
<th>ID</th>
<th>Earthquake Name</th>
<th>Year</th>
<th>Station Name</th>
<th>Mag.</th>
<th>Epicenter Distance, km</th>
<th>Scale Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Imperial Valley-02</td>
<td>1940</td>
<td>El Centro Array #9</td>
<td>6.95</td>
<td>12.98</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>Imperial Valley-06</td>
<td>1979</td>
<td>Agrarias</td>
<td>6.53</td>
<td>2.62</td>
<td>2.6</td>
</tr>
<tr>
<td>3</td>
<td>Victoria, Mexico</td>
<td>1980</td>
<td>Cerro Prieto</td>
<td>6.33</td>
<td>33.73</td>
<td>1.2</td>
</tr>
<tr>
<td>4</td>
<td>Superstition Hills-02</td>
<td>1987</td>
<td>El Centro Imp. Co. Cent</td>
<td>6.54</td>
<td>35.83</td>
<td>1.6</td>
</tr>
<tr>
<td>5</td>
<td>Landers</td>
<td>1992</td>
<td>Desert Hot Springs</td>
<td>7.28</td>
<td>27.32</td>
<td>2.7</td>
</tr>
<tr>
<td>6</td>
<td>Erzincan, Turkey</td>
<td>1992</td>
<td>Erzincan</td>
<td>6.69</td>
<td>8.97</td>
<td>1.5</td>
</tr>
<tr>
<td>7</td>
<td>Parkfield-02, CA</td>
<td>2004</td>
<td>Parkfield - UPSAR 13</td>
<td>6.00</td>
<td>12.59</td>
<td>2.6</td>
</tr>
</tbody>
</table>

The one-third-scale frame examined had a natural period of 0.303 s, which corresponds to a full-scale period of 0.525 s. The selected ground motions were scaled such that the average spectral acceleration follows the requirements of NEHRP (2015), with the result shown in Figure 1.

![Figure 1. Spectral response](image-url)
4. STRUCTURAL ANALYSIS

The frame examined was a one-third scale, three-story, three-bay planar structure designed by Ghannoum and Moehle (2012) to develop a flexure-shear-critical failure mechanism (i.e., the columns yield in flexure prior to a shear failure) typical of pre-1970s construction. Two columns had widely-spaced shear reinforcement (referred as ‘non-ductile columns’), and the other two were designed according to ACI 318-08 (referred as ‘ductile columns’). The beam-column joints were designed to avoid joint failure prior to a column failure. 26.68-kN lead weight packets were distributed on the beams over two points located at approx. 0.4 m from the face of each column. A sketch of the frame is shown in Figure 2.

![Figure 2. Frame and section design details](image)

Four shake-table tests using the March 3, 1985, Chile Earthquake (Llolleo Station, Component 100) were performed: half-yield (HY), and dynamic tests 1, 2, and 3 (DT1, DT2, and DT3). The response of the frame to each test and the ground motion scale factors are presented in Table 2 (Ghannoum and Moehle 2012).

<table>
<thead>
<tr>
<th>Test</th>
<th>GM Scale Factor</th>
<th>Frame Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>HY</td>
<td>0.3625</td>
<td>Minor flexural cracks.</td>
</tr>
<tr>
<td>DT1</td>
<td>4.06</td>
<td>Column 3 shear and axial failure at 5.2% story drift.</td>
</tr>
<tr>
<td>DT2</td>
<td>4.06</td>
<td>Column 4 advanced shear damage. No failure.</td>
</tr>
<tr>
<td>DT3</td>
<td>5.80</td>
<td>Column 4 failure. Partial collapse of frame's east side.</td>
</tr>
</tbody>
</table>

5. NUMERICAL MODELING

Three numerical models were created for this study: a fully nonlinear model that employs distributed-plasticity fiber-based elements, called Nonlinear Fiber-Based (NLFB) model; a simplified nonlinear model with fewer and longer flexure-only elements with combined shear hinges, called Nonlinear Fiber-Based...
Shear Hinge (NLFBSH) model; and a fully-elastic model with concentrated flexure, axial, and shear hinges, called Elastic with Concentrated Plasticity Hinges (ECPH) model. Due to their computational efficiency and analytical accuracy, all models employed one-dimensional beam-column elements.

5.1 Nonlinear fiber-based (NLFB)

The NLFB model was developed using the analysis package VecTor5 (Guner and Vecchio 2008) which employs the frame element developed by Guner and Vecchio (2010), based on the Disturbed Stress Field Model (DSFM) (Vecchio 2000). The DSFM accounts for the coupled flexure, axial and shear effects while incorporates several second-order material behavior models that are specific to reinforced concrete (RC) structures (See Table 3). In addition, the DSFM accounts for the local stress and strain conditions at cracks and calculates the widths and orientations of cracks throughout the load-deformation response of the structure. The shear strains are calculated using a parabolic strain distribution (Guner and Vecchio 2010). The element employs a smeared, rotating-crack approach based on a total load, secant-stiffness formulation. Confinement effects on the concrete are inherently accounted for using in- and out-of-plane reinforcement components.

<table>
<thead>
<tr>
<th>Material behavior</th>
<th>Model</th>
<th>Material behavior</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression base curve</td>
<td>Popovics (NSC)</td>
<td>Cracking criterion</td>
<td>Mohr-Coulomb (Stress)</td>
</tr>
<tr>
<td>Compression post-peak</td>
<td>Modified Park-Kent</td>
<td>Crack width check</td>
<td>Max crack width of Agg/5</td>
</tr>
<tr>
<td>Compression softening</td>
<td>Vecchio 1992-A</td>
<td>Concrete hysteresis</td>
<td>Nonlinear w/plastic offsets</td>
</tr>
<tr>
<td>Tension stiffening</td>
<td>Modified Bentz 2003</td>
<td>Slip distortion</td>
<td>Walraven</td>
</tr>
<tr>
<td>Tension softening</td>
<td>Linear</td>
<td>Rebar hysteresis</td>
<td>Seckin w/Bauschinger</td>
</tr>
<tr>
<td>Confinement strength</td>
<td>Kupfer/Richart</td>
<td>Rebar dowel action</td>
<td>Tassios (Crack slip)</td>
</tr>
<tr>
<td>Concrete dilatation</td>
<td>Variable - Orthotropic</td>
<td>Rebar buckling</td>
<td>Refined Dhakal-Maekawa</td>
</tr>
</tbody>
</table>

In the NLFB model, the concrete uniaxial stress-strain response was modeled using the Popovics and Modified Park-Kent models for the pre- and post-peak responses (Wong et al. 2013). The longitudinal reinforcement was modeled using discrete bars while the shear reinforcement was smeared into the relevant concrete layers. As recommended by Guner and Vecchio (2010), each beam and column were divided into elements of about half of its cross-section height (see Figure 3), and the number of fibers used in all cross-sections was kept at about 30 fibers. The NLFB model incorporated a nonlinear concrete hysteretic response with plastic offsets proposed by Vecchio (1999). The reinforcing steel hysteretic response was based on the Seckin model with Bauschinger effect in tension, and the Refined Dhakal-Maekawa model for compression (Wong et al. 2013), as shown in Figure 4. The primary energy dissipation mechanism of the NLFB model occurs due to the nonlinear hysteretic material constitutive models. The average Newmark integration method, which typically requires a minimal amount of damping for numerical stability, was used in the dynamic analyses of all models. The NLFB model achieved numerical stability with a Rayleigh damping ratio of 0.5% for the first two modes.
5.2 Nonlinear fiber-based shear hinge (NLFBSH)

The NLFBSH model, developed using OpenSees (Mazzoni et al. 2006), had fewer and longer elements with a simplified material model formulation. The material constitutive models incorporated flexure and axial effects only while second-order RC material behaviors were not included. Lumped shear hinges were incorporated to account for the shear effects (see Figure 3). The model simplifications were intended to reduce the computational effort while including the critical global structural response mechanisms.

The beams and columns were subdivided into closely-spaced elements in series with uncoupled shear-springs at the ends of the beams and columns. Longer elements were used in the center of the beams and columns due to the reduced inelastic response of these regions (see Figure 3). This mesh layout incorporates the findings of Leborgne and Ghannoum (2013), who developed the shear-hinge constitutive model used in this study, based on the rotation of the plastic hinge element (see Figure 3). The constitutive model was developed considering a rotation-based shear failure of an element that yields in flexure prior to a shear failure with nominal shear strength as per ASCE 41 (ASCE 2014) and 20% residual strength. The plastic hinge length was conservatively chosen to be one-and-a-half times the cross-section height to contain the plastic hinge region (Leborgne and Ghannoum 2013).

The concrete constitutive stress-strain distribution was modeled using the Hognestad parabola and linear models for the pre- and post-peak responses, respectively. A trilinear concrete hysteresis model with pinching effects developed by Filippou (2010) was employed. Confinement effects on the concrete were calculated using the model proposed by Mander et al. (1988) because OpenSees does not inherently account for this effect. The longitudinal steel reinforcement was discretely modeled with a three-linear stress-strain response (see Figure 5). The hysteretic model incorporated the Menegotto-Pinto model (see Figure 5). A
Rayleigh damping ratio of 3% for the first two modes was required for numerical stability due to the reduced number of elements and simpler constitutive models employed.

![Figure 5. (a) NLFBSH hysteretic concrete and steel reinforcing material models; (b) ECPH hysteretic model](image)

5.3 Elastic with concentrated plasticity hinges (ECPH)

In the ECPH model, each beam and column were modeled using linear-elastic elements with concentrated-plasticity hinges as the only mechanism that simulated the post-cracking and nonlinear behavior of the elements. The model was developed using the computer program SAP2000 (CSI 2015) with hinge constitutive models created as per ASCE 41 (2014) (see Figure 5b), where point B represents hinge yielding; point C represents the ultimate capacity of the hinge; and points D and E represent the residual strength and total failure conditions, respectively. It should be noted that the concentrated-plasticity hinges do not account for the nonlinear state of the element; rather, they limit the resistance of the elements (i.e., moment, shear, or axial) at the locations at which they are placed.

Coupled flexure-axial force and uncoupled shear hinges were used in the beam and column elements. The flexure-axial hinge response was calculated by SAP2000 as per ACI 318-02 (CSI 2015). The shear hinge response, on the other hand, was manually calculated to conform with the newer ACI 318-14 (2014). The flexure-axial force interaction hinges were incorporated at the faces of the beam-column and column-footing interfaces. Shear hinges were placed d away (i.e., the effective depth of the element) from the beam-column and column-footing interfaces as per ACI 318-14 (2014) (see Figure 3). The moment hinge length was taken as equal to the cross-sectional height as per CSI (2015). A shear hinge length of 1.5 times the cross-section height was adopted as for the NLFBSH model. The moment of inertia of the elements was reduced by factors of 0.35 and 0.7 for beams and columns, respectively, to account for the cracked conditions of the members (ACI 318-14 2014).

In the ECPH model, only the nonlinear hinges exhibited a simple hysteretic response, which followed a linear path as shown in Figure 5b. A Rayleigh damping ratio of 5% was used for the first two modes as per ASCE 41 (2014) due to the fully-elastic elements employed.

5.4 Mechanisms not included

As characteristic of pre-1970s construction, the beam-column joint and bar-slip damage mechanisms should be included in the numerical models. However, because the joints of the frame examined in this study were designed as per modern seismic codes to prevent beam-column joint failures, semi-rigid end-offsets were incorporated in the beam-column and column-footings connections in the developed numerical models.

6 NUMERICAL MODELS CALCULATED RESPONSE

The simulation accuracy of the developed models was evaluated using a dynamic time-history analysis obtained from the experimental shake-table tests (see Table 2) performed by Ghannoum and Moehle (2012). The experimentally-recorded time step was linearly divided into 100 sub-steps to increase convergence and accuracy (Guner and Vecchio 2012). Same as the experimentally observed response, the structure failed at the shake-table dynamic test 1; thus, numerical analyses of the subsequent shake-table tests were not
performed in this study. To account for the cracked condition at the start of the dynamic analysis, the half-yield analysis was performed on the NLFB and the NLFBSH models only, because the ECPH model does not consider concrete cracking. The calculated results were compared to the experimental ones in terms of the base shear, first-story drift, damage progression, and failure conditions. The first-story drift and base shear responses of each model in the numerical analysis of the shake-table dynamic test 1 are shown in Figure 6.

Figure 6. Numerical models and experimental first-story drift and base shear responses for the numerical analysis of the shake-table dynamic test 1

The NLFB and the NLFBSH models predicted failure at the first story level of Column 3 at a time of approx. 22 seconds (see Figures 6 and 7), which correlated well with the 22.5 seconds at which the shear failure occurred experimentally. Furthermore, the calculated first-story drift and base shear values well simulated the experimental responses, with the calculated-to-experimental ratio discrepancies below 15%, as shown in Figure 8. The ECPH model calculated a moment hinge failure at the first story level of Column 3 (see Figure 7) at a time of 12.6 seconds (see Figure 6). The calculated frame response resulted in a significant underestimation of the structural capacity and presented the highest deviation from the experimental response (see Figure 8).

Figure 7. Failure load stage for a) NLFB, b) NLFBSH, and c) ECPH models

The hysteresis response of the models (see Figure 9) showed that the ECPH model calculated a slightly stiffer behavior while the NLFB and the NLFBSH models captured the experimental responses with reasonable accuracy. The discrepancy was caused by the lack of nonlinearity in the beam-column elements and the simplified linear hysteretic constitutive models employed by the plastic hinges. In addition, the cracked moments of inertia suggested by ACI 318 (2014) and the nonlinear contribution of the hinges were not sufficient to accurately calculate the experimental energy dissipation.
For assessing required computational time of each model, the total time demand was subdivided into three phases: the model development time, the analysis time, and the results acquisition time. It should be noted that the model development and result acquisition times will vary from analyst to analyst. In this study, these times were consistently obtained by a single analyst with similar levels of previous experience with each software program used. All analyses were performed on an Intel® Core™ i5-2500 quad-core 3.3GHz CPU with 8GB DDR3 1333MHz RAM.

The lack of any user interface, and an inconsistent users’ manual for elements and models used, exponentially increased the model development time of the NLFBSH model (i.e., approx. 80 hours). Both the NLFB and ECPH models, which possess graphical pre-processor interfaces, required a much lower model development times of approx. 8 and 4 hours, respectively. The analysis time, on the other hand, was found to be directly dependent on the comprehensiveness level of the numerical model. As such, the NLFB model, which considered the most comprehensive material modeling and employed the highest number of nodes and elements, required the highest analysis time of all models (i.e., approx. 22 hours). The NLFBSH and ECPH models required an analysis time of approx. 2.7 and 0.08 hours (5 minutes), respectively. The result acquisition time was found to be dependent on the availability of a graphical post-processing user interface. Consequently, the NLFBSH model, again, required the highest acquisition time (i.e., approx. 5 hours). The NLFB and ECPH models required approx. 0.5 and 0.3 hours for the result acquisition, respectively. As such, when considering the total time demand as the sum of each individual component, the model with the highest demand was the NLFBSH with 87.7 hours, followed by the NLFB with 30.3 hours.
and the ECPH with 4.4 hours.

7. PERFORMANCE ASSESSMENT AND FRAGILITY FUNCTIONS

Fragility functions, which defines the probability of incurring a performance limit as a function of ground motion intensity (FEMA 2012), were derived to study the probability of exceeding certain pre-defined performance levels. The performance of the structure was quantified by comparing the calculated maximum first-story drift (θ_{max}) – which was the chosen as the engineering demand parameter (EDP) – to three performance levels of immediate occupancy (IO), life safety (LS), and collapse prevention (CP) as per ASCE 41 (2014). The spectral acceleration, S_a, is the intensity measure parameter for the ground motions. The choice of which damage measure (i.e., the limits for the performance objective) a structure is going to be assessed against is a subject that depends on regulatory agencies, code specifications, and the building owner requirements. The maximum drift ratio for the IO is commonly considered to be the value at which the frame enters the inelastic range, which was determined to be 1.5% from the results of a pushover analysis, conducted using the NLFB model (see the initial pushover load in Figure 2). The LS drift ratio was taken as 2% as per FEMA 356 (2000) and ASCE 41 (2014). The CP drift ratio was taken as 3%, which represented 75% of the ultimate drift ratio (Erberik 2008). The fragility curves were derived using a cumulative probability distribution as per Equation 1.

\[ P[\theta \geq D] = 1 - \frac{1}{2} \left[ 1 + \text{erf} \left( \frac{\ln(D/\mu)}{\beta \sqrt{2}} \right) \right] \]

(1)

where \( P[\theta \geq D] \) indicates the probability of the defined engineering demand parameters (EDP) (i.e., drift ratio in this study) to exceed the allowable threshold \( D \) (i.e., IO, LS, or CP \( \theta_{max} \)); \text{erf} is the Gauss error function; \( \mu \) is the median value of the EDP at a given ground motion intensity; \( \beta \) is the standard deviation of the natural logarithm of the ground motion index of the damage state. The median value of the EDP is calculated by exponential regression of the \( \theta_{max} - S_a \) plot (see Figure 10).

Seven pre-selected ground motions were used for a parametric study, where each motion was scaled several times to produce a range of spectral accelerations at the first natural period of the structure. The imposed spectral accelerations were: 0.3g, 0.6g, 0.8g, 1g, 1.25g, and 1.5g. In total, 126 NLDAs were performed (i.e., 42 for each numerical model), requiring approx. 260 hours of analysis time. In Figures 10a, b, and c, the black diamond-shape points are the recorded maximum first-story drifts calculated by the numerical models on each nonlinear dynamic analysis. The red lines show the standard deviation from the mean, expressed by the red ‘x’ point. The dotted black lines show the exponentially-fitted curves for all the first story-drift points, which was used to obtain the median values in Equation 1.

When compared to the NLFB model, the structural response calculated by the NLFBSH model provided a slightly better statistical fit in this study. In Figures 10a, b, and c, the NLFBSH model had the lowest number of calculated drift points outside the plus or minus standard deviation range. The ECPH model provided the poorest data dispersion and curve fitting characteristics of all three models (see Figure 10c). Structural collapses were calculated even at low spectral ground motion acceleration levels (see Figure 10c) and for the ground motions with spectral acceleration above 1.0g, all calculated structural failures. The response obtained from the ECPH model resulted in considerably high drift values.

![Figure 10. Calculated structural response for models (a) NLFB, (b) NLFBSH, and (c) ECPH](image-url)
The developed fragility curves for the selected performance levels are shown in Figure 11. By comparing the NLFB and NLFBSH models (see Figure 11a), a similar calculated probability of exceedance was observed for all three performance levels, with the highest calculated difference between the two models occurring for the CP level. In general, the NLFB model calculated a higher and more conservative probability of exceedance of all performance levels apart from the high spectral acceleration region (i.e., greater than approx. 1g) of the IO and LS performance levels, where the NLFBSH model calculated higher probabilities. Figure 12a shows that the maximum difference in the probability calculated by the NLFB and NLFBSH models was approx. 20% in all the performance levels. When the ECPH model was considered, the calculated results presented a significant deviation from the other two models. The ECPH model calculated the most conservative results due to the higher overestimation of the drift response caused by the inability of the ECPH model to redistribute forces once the first hinge fails. The maximum difference in the calculated probabilities of the ECPH model was approx. 40% and 55% in all performance limits for the NLFB and NLFBSH models, respectively (see Fig. 12b and c).

![Figure 11. Fragility functions for the (a) immediate occupancy (IO), (b) life safety (LS), and (c) collapse prevention (CP) performance levels](image)

![Figure 12. Calculated probability difference between (a) NLFB and NLFBSH, (b) NLFB and ECPH, and (c) NLFBSH and ECPH numerical models](image)

From the results observed in Figures 11 and 12, the accuracy of the response calculated by the NLFBSH and the NLFB models were similar, despite the simplified material model formulation (i.e., no coupled shear effects, no second-order behaviors) and coarser mesh of the former compared to the latter. It should be noted, however, that the frame examined in this study exhibited a known primary failure mode (i.e., the analyst knew, from the experimental observation, that column shear failure was dominant) which, despite the model simplifications, was still accounted for by the NLFBSH model. In a different failure scenario, where other failure mechanisms that are not accounted for (i.e., due to the simplifications of the model) could have caused the NLFBSH model to calculate erroneous responses. As such, the NLFB model was found to be more reliable in accounting for possible failure modes due to its more comprehensible modeling approach and material modeling formulations.
8. CONCLUSION

The findings of this study support the following conclusions:

1) The nonlinear models examined in this study provided significantly more accurate response predictions, i.e., within 15% of the experimentally observed, than the linear elastic model with concentrated plasticity hinges, which calculated discrepancies of over 40% and a different failure mode from the experimentally observed.

2) The total time demanded was found to be mainly comprised of three phases: modeling, analysis, and result acquisition times. While the analysis time of each model was directly proportional to the level of comprehensiveness of the material models adopted in each model, the modeling and result acquisition times were proportional to the available user-friendly features of each modeling package such as pre- and post-processing tools, and clear documentation.

3) The maximum probability difference calculated by the NLFB and NLFBSH models were 20% for all the studied performance levels. The NLFBSH model, despite its relatively similar calculated probability to the NLFB model, should be used with caution due to its simplified modeling approach, material modeling formulation, and high total computational time demand. The NLFB model employs a more comprehensive modeling approach, material modeling formulation, and requires a lower total computational time demand, which makes it more reliable for practical applications for the next generation of buildings.

4) The derived fragility curves showed that the linear elastic model with concentrated plasticity hinges calculated a considerably higher probability of exceedance for the three performance limits (i.e., immediate occupancy, life safety, and collapse prevention) when compared to the other two models. Based on the results of this study, this model was found not well suited to a performance-based earthquake engineering assessment.

5) Numerical models that make use of simplified material model formulations and do not account for the majority of concrete and reinforcement material models may lead to inaccurate results in performance-based earthquake engineering analyses due to their reliance on the prior knowledge of the governing material behaviors and failure modes, which are not typically known for real structures.

9. ACKNOWLEDGEMENTS

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10. REFERENCES

ACI Committee 318. (2014). Building code requirements for structural concrete (ACI 318-14) and commentary. American Concrete Institute, Farmington Hills, MI. 519 pp.


