



# Assessing Different Modeling Approaches for a Four-Story Reinforced Concrete Building

Hamid Khodadadi Koodiani<sup>1</sup>(✉), Adolfo Matamoros<sup>1</sup>, Insung Kim<sup>2</sup>, Ariel Creagh<sup>2</sup>, Tarbin Basnet<sup>1</sup>, and Wassim Ghannoum<sup>1</sup>

<sup>1</sup> The University of Texas at San Antonio, San Antonio, USA  
{hamid.khodadadikoodiani, adolfo.matamoros, wassim.ghannoum}@utsa.edu

<sup>2</sup> Degenkolb Engineers, San Francisco, USA  
{ikim, acreagh}@degenkolb.com

**Abstract.** OpenSees and Perform3D models of a four-story reinforced concrete building tested on the E-Defense earthquake simulator were created in accordance with ASCE 41–17 provisions. The computational models differed in the techniques used to simulate the hysteretic response of beam-column joint, column, and wall elements. OpenSees building models followed lumped plasticity and distributed plasticity (fiber section) approaches. In OpenSees fiber section models, columns and walls were discretized using force-based fiber section elements with 5 integration points and Gauss Lobatto integration, while beams were modeled with zero-length hinge elements at member ends. The stress-strain behaviour of steel elements in the fiber section model was adjusted so the minimum and maximum strain limits would cause loss of lateral load capacity at a rotation similar to the capping point in the backbone curve specified in ASCE 41. Calculated performance metrics were compared with measured responses for strong motion records used in the tests. The output of OpenSees and Perform3D models were compared to show the difference in calculated deformation obtained with different analysis engines and modeling techniques. Analyses showed that approaches that capture bi-directional moment-axial interaction provided significantly different estimates of response than lumped-plasticity. Models that accounted for bi-axial interaction of column moments resulted in larger estimates of column rotation and damage.

**Keywords:** Fiber model · Global response · Lumped plasticity · Modeling parameters · Nonlinear modeling

## 1 Introduction

This study evaluates the effect of different modeling approaches on calculated column and building deformations of a 4-story reinforced cast-in place reinforced concrete structure. The building, shown in Fig. 1, was instrumented and tested on the E-Defense earthquake simulator in Japan in 2010.

The comparisons presented in this paper are important to validate the Tier 3 performance assessment methodology in ASCE 41. Performance-based assessment methods in ASCE 41 require that building performance level be established on the basis of

calculated element deformations. This study presents a comparison of measured and computed deformations in the building and its columns for different analytical models permitted in ASCE 41. Emphasis is placed on columns because they are critical to performance assessments due to their fundamental role in maintaining the stability of buildings and transmitting gravity load to the foundation. The fundamental question being addressed is whether evaluators using different modeling approaches to evaluate building performance would arrive at similar outcomes.

The case study structure was assessed following ASCE 41–17 [1] nonlinear dynamic procedure. The OpenSees [2] and Perform3D [3] software platforms were used to create nonlinear models with nonlinear and linear elastic joint elements.

OpenSees building models were created using lumped plasticity and distributed plasticity component models. One of the fundamental differences between the OpenSees models was that the lumped-plasticity model did not account for biaxial moment nor axial load-moment interactions in columns, while the distributed plasticity model (fiber section) did. Columns in the Perform3D model had PMM hinges, a lumped plasticity approach that accounts for both axial load-moment and biaxial bending interactions through an interaction surface based on plasticity theory.



**Fig. 1.** Four-story RC structure tested at E-Defense shake table 2010. [4]

## 2 Case-Study Building and Analytical Models

The study structure consists of two moment frames in the longitudinal direction and two wall frames and a moment frame in the transverse direction. Frames in both directions were 7.2 m apart, and the building was structurally regular. The building was designed in accordance with the seismic design guidelines of the Architectural Institute of Japan (1999) [5] and complied with most of the requirements of ACI 318–11 [6] for special moment frame and wall systems. The floors were 3 m high, and a pan joist system with a 130 mm slab thickness was used at all levels. The two walls were 2500 mm in length and

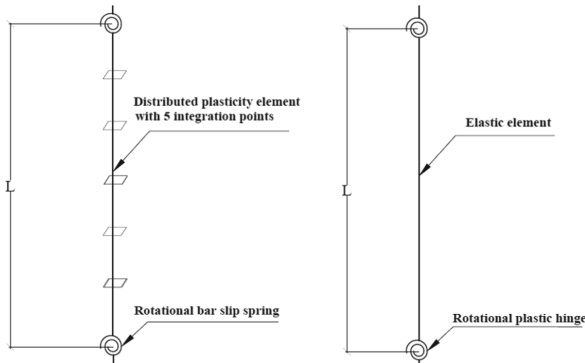
250 mm thick. All columns had a 500 mm square cross section. Beams were 300 mm wide by 600 mm deep in the frame direction, and in the transverse direction the beams connecting the walls to the corner columns were 300 mm wide and deep.

Dynamic analyses and results presented in this paper were performed with the 100% JMA-Kobe record [7].

## 2.1 OpenSees Models

OpenSees models were created using the provisions in ASCE 41–17 with two different beam-column joint models, one that took into account nonlinear joint behavior (NJ) and another with linear elastic joint elements (EJ). Columns and walls were modeled using lumped-plasticity (LP) or fiber section (Fiber) approaches. A total of four models, OpenSees LP NJ, OpenSees LP EJ, OpenSees Fiber NJ, and OpenSees Fiber EJ were created. The fiber section models used fiber section elements for columns and walls and lumped-plasticity elements for beams, while the lumped-plasticity models used lumped-plasticity elements for all frame and wall members.

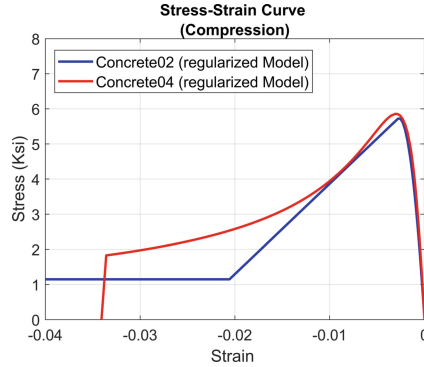
In lumped-plasticity models beams, columns, and walls consisted of elastic beam-column elements with zero-length rotational hinge elements at member ends, where inelastic deformations were expected (Fig. 2, right). Moment-rotation behavior of rotational springs was defined using the envelope curve and hysteresis rules of the peak-oriented Ibarra-Medina-Krawinkler (IMK) material model [8]. Hinge properties for all frame and wall members were defined using the ASCE 41–17 modeling parameters and backbone shape. Khodadadi et al. [9] proposed different set of formulas to predict the nonlinear deformation modeling parameters of concrete columns. Force-based fiber section elements with five integration points and Gauss Lobatto integration were used to discretize columns and walls (Fig. 2, left) in fiber section models. Fiber section elements included zero-length rotational springs at either end to account for bar slip. Beams in fiber section models were modeled using elastic beam elements with zero-length rotational hinges at member ends.



**Fig. 2.** Element topology for fiber section (left) and lumped plasticity (right) models.

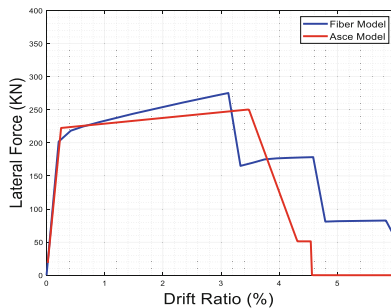
The fiber section elements were defined using the following material models: concrete02 for unconfined concrete in the column cover and wall web, concrete04 for confined concrete in the column core and wall boundary, and a steel 02 for reinforcing bars.

The concrete model developed by Mander et al. was used to calculate the fundamental parameters of the stress-strain curve for concrete in compression [10]. In order to avoid localized deformations after concrete entered the descending branch of the stress-strain curve, the stress-strain relationship (Fig. 3) was regularized.



**Fig. 3.** Regularized stress-strain relationships for concrete in compression.

The Steel02 material model with a MinMax material object were used to simulate the stress-strain relationship for steel in OpenSees. The MinMax material object used the stress-strain behavior provided by the Steel02 material model while strains remain within the minimum and maximum limiting values, and assumes that the material has failed if the limiting values are reached, at which point both stiffness and stress are set to zero. The minimum and maximum strain limits were calibrated to cause loss of lateral load capacity in the moment-rotation relationship at the corresponding modeling parameter in ASCE 41. Steel had calibrated strain limits of 0.11 for walls and 0.13 for all columns. Figure 4, which displays the calculated backbone curves for the fiber section column elements and the corresponding backbone curve derived in accordance with ASCE 41–17, illustrates the calibration approach. A limitation of the study is that although the method was successful in reproducing the plastic rotation at loss of lateral load capacity, it failed to produce the residual capacity specified in ASCE 41 because the MinMax object reduced strength to zero after exceeding the limiting strain.



**Fig. 4.** Lateral force-drift ratio relationships for fiber section model and ASCE 41 envelope of first story column.

Modeling parameters were calculated in accordance to the ASCE 41 provisions considering the maximum axial load calculated with a pushover analysis of the building. Effective flange widths were assigned to the beams to account for slab out-of-plane bending explicitly. The model did not take into account the joists that supported the slab.

The maximum compressive axial load ratio was less than 10% of the gross sectional capacity of the column over the entire height of the building. All columns were expected to be controlled by flexure due to the high levels of confinement and the fact that shear strengths were significantly greater than shear demands associated with flexural hinging. For these reasons all frame members were modeled without any reduction to the elastic shear modulus.

For similar reasons all beams were expected to be controlled by flexure. An effective flange width was determined and included in the estimation of beam moment strength according to the provisions in ASCE 41–17.

Beam-column joints were classified as conforming according to the provisions in ASCE 41–17, despite the relatively wide hoop spacing (140 mm). Using ACI 318–19 methodology and a static mechanism analysis, it was found that beam-column joints exceeded their shear strength at the lower stories of the building in the frame direction but not in the wall direction. Beam-column joints were modeled in the OpenSees EJ (Elastic Joint) model using elastic components that extended from the beams and columns and connected directly at the center of the joint. Joint elastic stiffness was modeled implicitly according to ASCE 41–17 Sect. 10.4.2.2.1. The lumped plasticity and fiber section OpenSees NJ (Nonlinear Joint) models all modeled joints in the same manner. In both models, joint elastic elements extending from beams and columns remained but rotational springs were introduced in the frame direction, to generate a scissor motion in the joint. Nonlinear rotational spring behavior was calibrated to match the modeling parameters in ASCE 41–17 Sect. 10.4.2 [1].

Given the high shear strength-to-shear demand and height-to-length ratios, modeling parameters for hinges at the wall foundation interface were defined according to provisions for walls controlled by flexure in ASCE 41–17.

The loading sequence applied to the building in the earthquake simulator consisted of multiple runs with the JMA-Kobe motion with increasing scale factor. The preceding motions applied to the building in the experimental program were not applied to nonlinear models; instead, the models were only subjected to the three components of the 100% JMA-Kobe motion.

## 2.2 Perform3D Model

Perform3D models employed lumped-plasticity hinges with P-M-M interaction for columns and fiber elements for the walls. The shear wall element in Perform3D was used to model the walls at all stories. Each shear wall element mesh contained both a nonlinear shear material and distributed nonlinear vertical fibers. The fibers and nonlinear stress-strain relationships of the shear material were calibrated to replicate the rotation or drift responses presented by the nonlinear modeling parameters in ASCE 41–17.

Frame beams were modeled in Perform3D using “complex frame components” that included an elastic frame segment along the clear span of the beam, a moment rotation hinge made of inelastic material, and a rigid end-zone at either end.

Compound frame elements were used to model columns. These components included an elastic frame segment along the column clear span, an inelastic P-M-M hinge at each end, and rigid end zones when necessary at the beam joint.

Beam-column joints were modeled with elastic linear frame elements (EJ) or a nonlinear rotational spring at the panel zone node when joint nonlinearity was considered (NJ).

### 3 Performance of Different Analytical Models

#### 3.1 Natural Periods

Table 1 shows a comparison of measured building fundamental periods at the start of the 100% JMA-Kobe motion with calculated periods from the lumped-plasticity and fiber section models. Because effective stiffness factors specified in ASCE 41 were used to reduce gross section properties in the lumped-plasticity model, it was expected that periods calculated with the lumped-plasticity model would be greater than periods calculated with the fiber model, where the modulus of elasticity was not reduced by any factor. Periods computed using fiber section models were generally greater than those recorded experimentally and slightly lower than those calculated using the lumped-plasticity model. The Perform3D model produced the largest period in the frame direction in comparison to OpenSees models. In the wall direction, the period calculated with the Perform3D model was slightly larger than the OpenSees fiber model and smaller than the OpenSees lumped plasticity model. This comparison shows an inconsistency between modeling approaches that should be addressed in future editions of ASCE 41.

**Table 1.** Calculated and measured building periods.

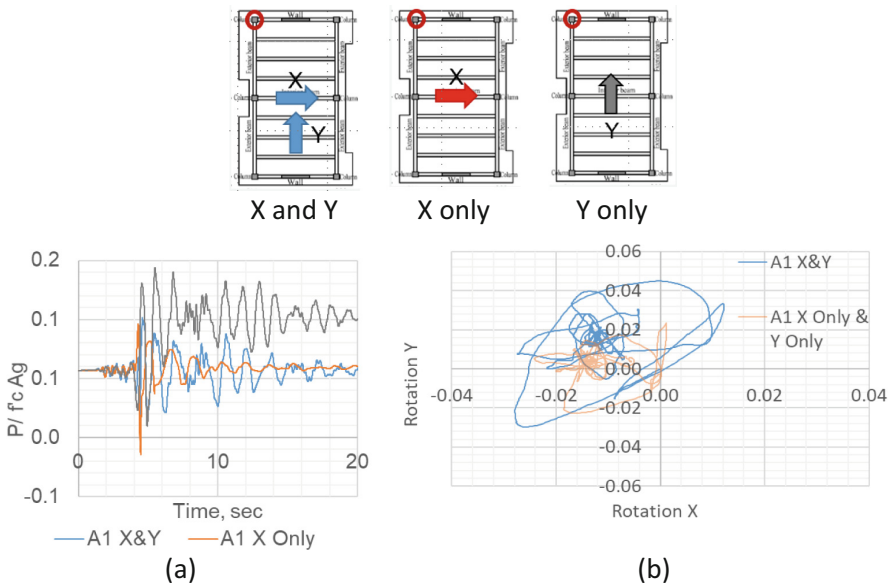
Period	Frame Direction	Wall Direction
Experiment - First mode period at the beginning of 100% JMA-Kobe	0.43 s	0.31 s
OpenSees LP NJ Model	0.64 s	0.40 s
OpenSees Fiber EJ and NJ Models	0.56 s	0.30 s
Perform3D Models	0.68 s	0.32 s

#### 3.2 Effect of Biaxial Interaction on Column Axial Load and Rotation

The effect of biaxial interaction on column response is illustrated in Fig. 5 (a) and (b). The Perform 3D PMM model was subjected to different components of the 100% JMA-Kobe

record to illustrate the effect of neglecting biaxial interaction on column response. In all cases the Perform 3D model did account for the effect of axial load-moment interaction. In Fig. 5(a) and 5(b) the X direction corresponds to the wall-frame direction, while the Y direction corresponds to the moment frame direction. All metrics shown are from the first story corner column shown with the red circle (designated A1 in the figures). Figure 5(a) shows the axial load history while Fig. 5(b) shows the calculated rotations in both directions. The red line in Fig. 5 (b) was obtained by combining the response to each of the two components carried out separate from each other.

The magnitude of the peak axial load in Fig. 5(a) was similar for the models with biaxial interaction (blue line) and earthquake excitation in the wall-frame direction only (red line), while the peak axial load was significantly higher for the model with earthquake excitation in the frame direction only (black line). The differences in axial load and the coupling effects also affected the hinge rotations. Figure 6 (b) shows that calculated column hinge rotations were higher when biaxial interaction was considered (blue line) than when biaxial interaction was neglected (red line). These differences are important because they show that engineers can arrive at significantly different performance outcomes depending on the modeling approach they implement, and that calculated rotations were significantly larger when biaxial moment interaction was considered.

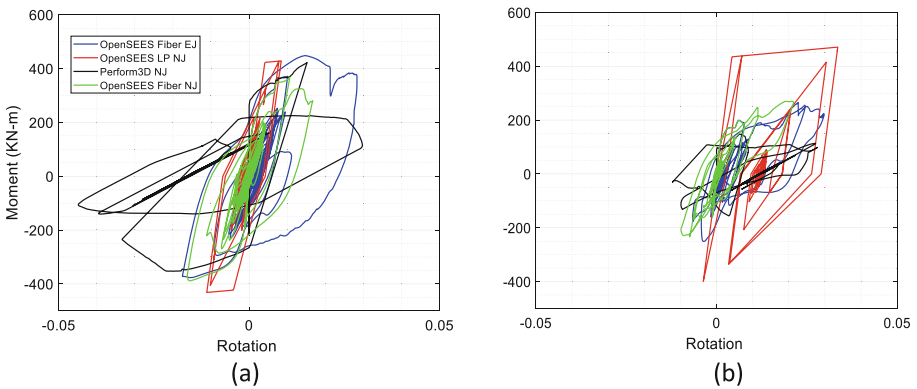


**Fig. 5.** (a) Axial load history and (b) hinge rotations calculated. All simulations correspond to Perform3D model in first story corner column A1 for: earthquake excitation in both directions, wall-frame direction only, and frame direction only.

Figure 6 shows a comparison of column rotations calculated with the different models evaluated in the study. OpenSees lumped plasticity models are the most limited of all models evaluated because they lack the capability to simulate axial load-moment

interaction and biaxial bending. In the frame direction moment demand was not significantly affected by axial load-moment interaction, so moment demands calculated with all models were similar. The lumped plasticity model produced the lowest rotation demand of all models, which indicates that biaxial interaction in this column was significant and that assessments based on lumped plasticity models are likely to result in more positive assessments of performance. A comparison between fiber section models show that joint modeling approach had a significant effect on calculated deformation demands on the columns. The model in which the joint was modeled to be linear elastic produced significantly higher deformation demands on the column elements than the model with nonlinear joints. In the latter, the larger deformation in the joint and the reduction in stiffness after shear damage had the effect of reducing deformation demands in the columns. Neither of the two fiber models appears to show loss of lateral load capacity on the column. The Perform 3D model produced the largest rotation demands on the columns, and appeared to show that the capping point of the column was exceeded.

In the wall direction, the OpenSees fiber models were in close agreement with the Perform3D model both in terms of calculated moment and rotation. There was a significant difference between moment demands calculated with the OpenSees lumped plasticity model and all models capable of simulating axial load-moment interaction, although peak deformation demands in the positive direction were similar in all models. In this direction it is likely that the axial load imposed on the columns by the motion of the wall was significant, but due to the large stiffness of the wall the effect of column flexibility on the response was not significant (Fig. 6).



**Fig. 6.** Moment-rotation relationships calculated with OpenSees LP, OpenSees Fiber and Perform3D models of first story corner column: (a) frame direction and (b) wall direction.

## 4 Conclusions

All building members were modeled using stiffness values that correspond to levels of cracking at yield in accordance to effective stiffness factors in ASCE 41-17. Therefore, computational models produced periods 22% to 46% larger than experimental measurements. It is anticipated in real buildings that not all members reach yield levels of



deformation, especially at higher floor levels, resulting in this discrepancy. The ASCE 41–17 approach to component stiffness may need to be updated to take into account the sensitivity of building response to varying stiffness across building height, and provisions to address effective stiffness in fiber section models should be considered.

The study shows that all models produced much more accurate estimates of global response metrics (drift ratio) than estimates of element response metrics (column rotation). Furthermore, calculated column rotations were shown to be sensitive to axial load-moment and biaxial moment interactions, not simulated in lumped plasticity models. The analyses performed shows that when variations in seismically induced axial load and biaxial interaction are significant, lumped plasticity models can produce significantly lower estimates of column rotation than fiber section and PMM hinge models.

The models also showed that joint modeling approach had a significant effect on calculated column deformation demand. Models with linear elastic joints produced higher estimates of column rotation demand than models where joints were modeled inelastically. This effect was particularly significant in this building because beams, columns and walls were detailed to achieve large deformations without loss of load capacity, while the joints were not. The distribution of damage in the joints estimated with the computer models was different than the pattern observed in the experiment, which showed that the outside joints only experienced mild damage while the internal joints sustained the most. Future research may justify a review, update, and justification of the ASCE 41–17 nonlinear modeling parameters and acceptance criteria for joint shear strength. Updates to the standard should include the base sliding mode of damage since it can significantly change the structure's overall mechanism and is not covered by ASCE 41–17.

It is worth mentioning that the interaction between moment and shear demands was not considered in the computational models used in this study. For this building the evaluations show that there is a critical need to enhance modeling capabilities. These include modeling strategies to account for bi-directional moment-axial coupling. ASCE 41–17 is not specific on when and how to treat bi-directional effects on member capacities. Such instructions would enhance the standard.

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