EFFECT OF MODELING PARAMETERS ON COLLAPSE SIMULATION OF RC BUILDING

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Abstract

The ability to simulate the response of buildings up to significant levels of component damage is of fundamental importance to evaluate resilience under different retrofit scenarios. In the United States there is an increasing reliance on the ASCE-41 Standard for developing computer models to simulate the response of existing structures during strong earthquakes. Modeling parameters for reinforced concrete columns in the ASCE-41 Standard have undergone significant changes since their inception in the early 2000s. The original set of modeling parameters, adopted from FEMA 356, was updated in 2007 to incorporate findings from component tests investigating the drift ratio at axial failure of reinforced concrete columns. Consistent with the philosophy of FEMA 356, the set of modeling parameters introduced in 2007 was calibrated to have a low probability of exceedance. A new set of modeling parameters was recently proposed based on mean values to represent expected behavior.

While these are positive improvements that are likely to provide more accurate models, the calibration of the modeling parameters in ASCE-41 was based on data sets from component tests, and the effect of changing the modeling parameters on the response of building systems is unknown. Furthermore, while modeling parameters for columns have undergone two updates, modeling parameters for beams remain unchanged. This paper presents the results from incremental dynamic analyses performed with different sets of modeling parameters on a non-ductile reinforced concrete building that was instrumented during the Northridge earthquake. The effect of the change in modeling parameters on the calculated intensity measure at collapse and the corresponding collapse mechanism are described.

Keywords: modeling parameters, dynamic analysis, collapse, non-ductile, reinforced concrete

Introduction

Reinforced concrete buildings built prior to 1970s are susceptible to significant damage and collapse during strong earthquakes. One of the greatest concerns about the detailing of older buildings is that columns with inadequate transverse reinforcement are prone to shear failure, which has been shown to trigger the loss of lateral load carrying capacity, precipitating axial failure and collapse. To improve the resilience of vulnerable reinforced concrete structures, it is of fundamental importance to further develop the accuracy of existing evaluation methods. The ASCE 41 (2014) standard is widely used in the United States as a guide to develop computational models to assess vulnerability to earthquakes and to gage the effectiveness of strengthening schemes in reducing the expected cost of repair. Since its inception, the modeling parameters for building components in the ASCE 41 Standard (2014) have undergone revisions with the goal of improving the accuracy of nonlinear dynamic analysis models. This paper investigates the effect of changes in modeling parameters on the calculated response of non-ductile reinforced concrete structures through a case study of a building that was instrumented during several strong earthquakes.

Building Description

The building analyzed in this paper is a seven-story reinforced concrete frame structure located in Van Nuys, California, operating as a Holiday Inn hotel. The building was designed in 1965 and constructed in 1966. It is rectangular in plan with eight bays in the east-west direction (total dimension of 150’-0”) and three bays in the north –south direction (total dimension of 62’-0”). The structural system of the building consists of exterior beam-column frames and interior slab-column frames. The interior flat slabs are 10 in. thick on the 2nd floor, 8.5 in. thick on the 3rd through 7th floors and 8 in. thick at the roof. Typical exterior columns dimensions are 14 by 20 in. and interior square columns have dimensions of 20 by 20 in the first story and 18 by 18 for the remaining stories. The strong axis of the columns is oriented in the north-south direction. Normal weight concrete was used throughout the building, with design strengths ranging from 3000 to 5000 psi. Grade 40 reinforcing steel was used in the beams and slabs and grade 60 steel was used in the columns. The foundation system of the building consists of pile caps supported by groups of cast-in-place concrete friction piles. A detailed description of the dimensions and reinforcement configuration of the elements used in the building models is presented by Suwal (2015).

The case-study structure, located at 8244 Orion Avenue, Van Nuys, has been exposed to three major earthquakes: the 1971 San Fernando, the 1987 Whittier Narrows, and the 1994 Northridge earthquake. The 1994 Northridge earthquake caused severe structural damage concentrated in the 4th and 5th levels of the east-west perimeter frames, where several columns suffered shear failure. The discussion in this paper is limited to the response in the east-west direction because it had the most severe damage.

Building model

Analyses discussed in this paper stem from three different models of the building, all of which had the same configuration but different sets of modeling parameters. The two-dimensional computer models comprised one half of the building and included one exterior moment-resisting frame and one interior slab-column frame. Models were built using the Open System for Earthquake Engineering Simulation (OpenSees) platform (Mazzoni et al., 2006). Figure 1 shows a schematic diagram of a portion of the models. The lumped plasticity approach was used to model beam and column elements, with zero-length rotational springs simulating nonlinear deformations at the plastic hinges. The hysteretic response of the zero-length elements was simulated using the one-dimensional material model developed by Ibarra et al. (2005). The backbone curve of the material model is shown in Fig. 3, which has the following key parameters: elastic stiffness (K_e), yield moment (M_y), ratio of capping-to-yield moment (M_c/M_y), inelastic rotation at capping (θ_p), inelastic rotation at loss of lateral load capacity (θ_{pc}), residual strength (c), and the inelastic rotation at total loss of capacity (θ_u). The model is capable of capturing reductions in strength and stiffness due to cyclic deterioration. Each deterioration component is defined by two parameters, the normalized energy dissipation capacity and an exponent term to describe the rate at which cyclic deterioration changes with accumulation of damage.

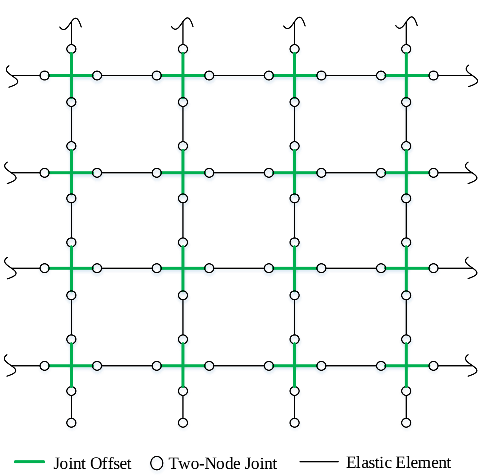


Figure 1. Schematic of OpenSees Model

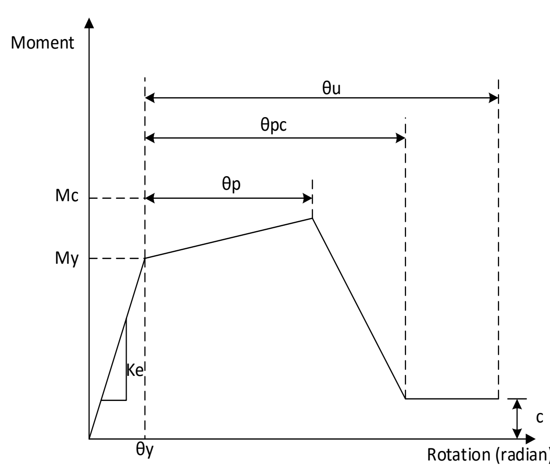


Figure 2. Nonlinear material model by Ibarra et al. (2005)

Analytical Approach

Approximately 300 nonlinear dynamic analyses were performed using OpenSees. The dynamic excitation for all analyses consisted of the 60 sec ground motion record from the 1994 Northridge earthquake recorded at the base of the building scaled to different intensity measures. All models had 2% Rayleigh damping. Initial stiffness for exterior beams and columns was equal to the effective stiffness specified in the ASCE 41-13 Standard, including the effects of the slab. Yield moment for the rotational springs was calculated using a longitudinal reinforcement yield strength equal to 1.25 times the nominal strength, and concrete strength was assumed equal to 1.5 times the specified compressive strength. The shear capacity of beam and column elements was determined according to the provisions of ACI 318-14 and ASCE 41-13.

The interior slab-column frame was modeled using beam-column elements, with modeling parameters corresponding to slab-column connections. The effective beam width model was used with an effective beam width factor of 0.48 and an effective stiffness factor of 0.33. Following the provisions in ASCE 41-13, these values were calculated based on recommendations by Hwang and Moehle (2000).

Modeling parameters

In all cases, modeling parameters for slab-column connections of the interior frame were adopted from the ASCE 41-13 Standard. The main difference between the three building models considered was that different sets of modeling parameters were used for the columns of both frames and for the beams of the perimeter moment-resisting frame. Modeling parameters for each of the models are described in the following.

ASCE 41-13 Model (Model 1). Modeling parameters and acceptance criteria for nonlinear analyses are provided in Chapter 10 of the ASCE 41-13 Standard. The shape of the backbone curve (Fig. 3) is defined using three modeling parameters: *a*, *b* and *c*. Parameter *a* represents the inelastic rotation at loss of lateral load capacity, parameter *b* the inelastic rotation at axial failure (or failure in the case of beams), and parameter *c* the residual strength ratio. Modeling parameters *a*, *b* and *c* are defined on the basis of detailing and element demands. For example, in the case of columns parameters *a*, *b* and *c* are defined based on shear strength, plastic shear demand, and detailing of transverse reinforcement. The modeling parameters in ASCE 41-13 were calibrated to have low probabilities of exceedance (Elwood et al., 2007), i.e., 35% for columns controlled by flexure and 15% for columns controlled by shear. Modeling parameters for beams and columns were adopted from the ASCE 41 Standard.

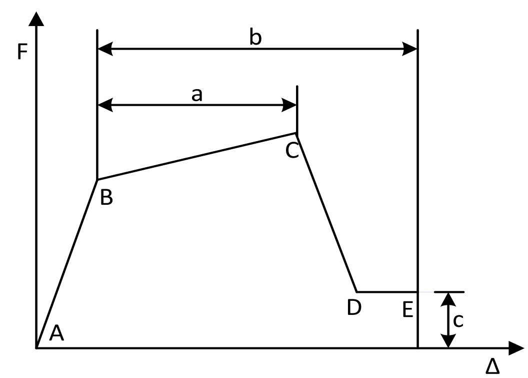


Figure 3: Generalized force-deformation relation

ACI-369 Model (Model 2). A new set of modeling parameters for columns is currently being balloted by ACI Committee 369 based on a proposal by Ghannoum and Matamoros (2014). This new set of modeling parameters was calibrated to have 50% probability of exceedance to avoid bias in modeling. Modeling parameters for the interior and exterior columns were calculated using the new provisions proposed by ACI committee 369 (Eq. 1, 2, and 3):

$$a_{nl} = 0.042 - 0.043\frac{N_{UD}}{A_g f'_{cE}} + 0.63\rho_t - 0.023\frac{V_{yE}}{V_{OE}} \geq 0.0 \qquad (1)$$

$$b_{nl} = \frac{0.5}{5+\frac{N_{UD}}{0.8A_g f'_{cE}} \frac{1}{\rho_t} \frac{f'_{cE}}{f_{ytE}}} - 0.01 \geq a_{nl} \tag{2}$$

$$c_{nl} = 0.24 - 0.4\frac{N_{UD}}{A_g f'_{cE}} \geq 0.0 \tag{3}$$

where a_{nl}, b_{nl}, and c_{nl} are the inelastic rotation corresponding to capping in units of radians, the inelastic rotation corresponding to total loss of lateral load capacity in radians, and the residual strength ratio of the column element, respectively. The term f'_{cE} is the expected concrete compressive strength, in units of psi, N_{UD} is the axial force in kips, A_g is the gross sectional area of the column in in^2, ρ_t is the transverse reinforcement ratio, and V_{yE} and V_{oE} are the expected values of plastic shear demand and the nominal shear demand in kips, respectively. Modeling parameters for beams were adopted from the ASCE 41 Standard.

Model with New Modeling Parameters for Beams (Model 3). Modeling parameters for beams in ASCE 41 remain the same listed in FEMA 356. For this reason, a new set of modeling parameters for beams was developed for the analyses presented in this paper based on experimental results. An expression for the total rotation corresponding to capping was developed based on a linear regression analysis from 56 tests in the PEER database with axial load ratios ranging between 0 and 0.12. Because there are very few tests that track the response of beam specimens to the point of total loss of lateral load capacity, a recommendation was developed by evaluating an axial load ratio of 10% in the equation proposed by ACI Committee 369 for columns (Eq. 3).

$$a_{nl} = 0.12\sqrt{\rho_t \frac{f_{ytE}}{f'_{cE}}} + \frac{L}{600\ h} - \frac{s}{1400\ d_b} + \frac{V}{300\ b\ d\sqrt{f'_{cE}}} - \theta_y \tag{4}$$

$$b_{nl} = \frac{0.5}{5+\frac{0.1\ f'_{cE}}{f_{ytE}} \frac{1}{\rho_t} \frac{h}{d_c}} - \theta_y \tag{5}$$

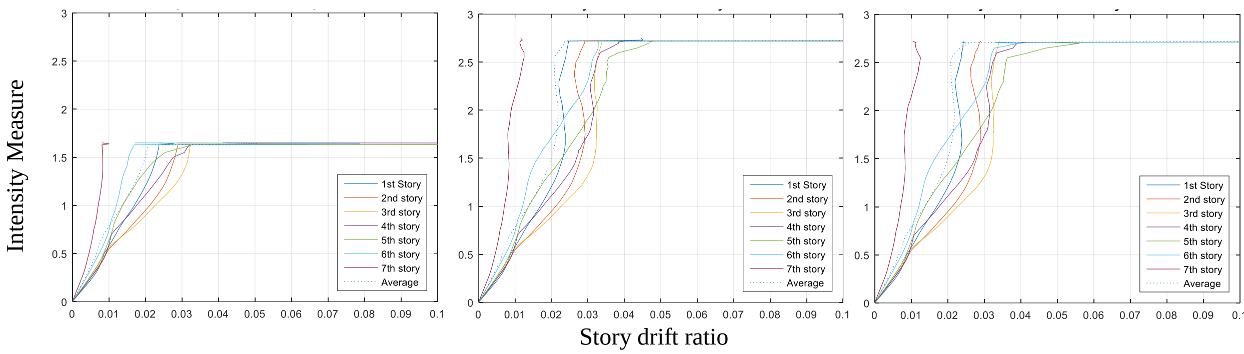
where a_{nl} and b_{nl} are the inelastic rotations corresponding to the capping point and total loss of load capacity, in radians, f_{ytE} is the expected yield strength of the transverse reinforcement in psi, f'_{cE} is the expected concrete compressive strength in psi, L is the clear span of the beam, h is the beam depth, s is the transverse reinforcement spacing, d_b is the longitudinal bar diameter, d_c is the depth of the beam core, V is the shear strength of the beam in lbs, ρ_t is the transverse reinforcement ratio, and b is the width of the beam in in. Modeling parameters for columns in Model 3 were those proposed by ACI Committee 369.

Results and Discussion

Incremental Dynamic Analysis. An incremental dynamic analysis of each of the three building models was performed using the Northridge ground motion with scale factors starting at 0.05, in increments of 0.05, until the model became unstable due to excessive lateral deformations. At intensities near lateral instability, the scale factor increment was reduced to 0.01. Results from the incremental dynamic analyses are presented in Fig. 4, which shows the variation of story drift ratio with respect to intensity measure for each of the three models. In all cases, the lateral instability of the building was triggered by tow-story mechanisms developing between the 4th and 5th stories of the building, where severe column damage was observed after the Northridge earthquake.

The intensity measure at which lateral instability developed was 1.64 for Model 1, 2.73 for Model 2, and 2.72 for model 3. Story drift ratios at lateral instability were as large as 3% for model 1, 4.5% for model 2, and 5.5% for model 3. Simulation results clearly showed that the intensity measure corresponding to lateral instability was strongly affected by the choice of modeling parameters for the columns and not sensitive to the modeling parameters for beams.

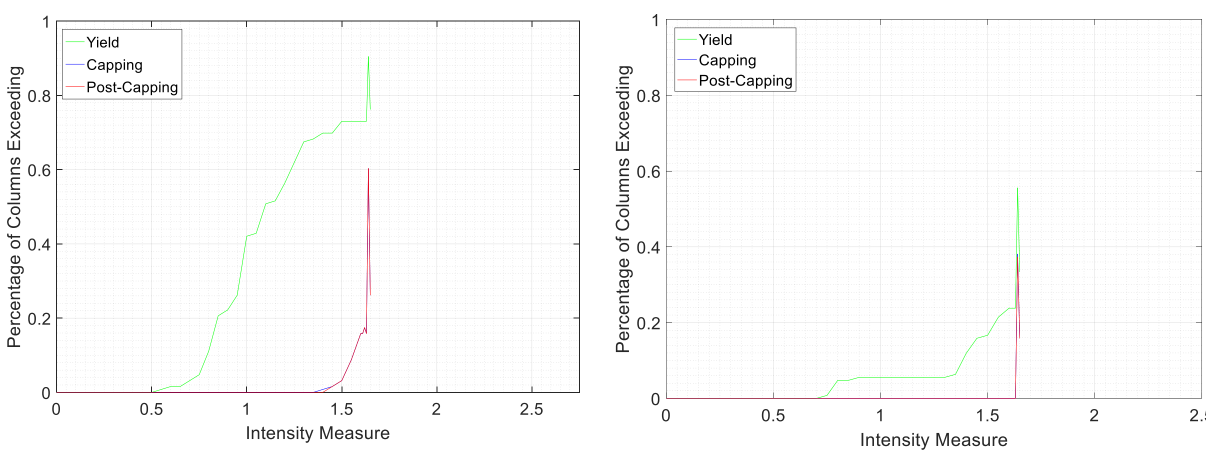
Figure 4 shows that the response of models 2 and 3 was very similar, with a small difference in the pattern of deformation at intensity measures near lateral instability. At this level of excitation, model 3, in which the beams of the perimeter frame had larger values of capping rotation (larger parameter *a*), had a maxim drift ratio in the fifth story approximately 1% higher than model 2. This indicates that the effect of changing the modeling parameters of the beams of the exterior frame allowed the frame to deform more, although the increase in drift ratio did not have a significant effect on the maximum intensity measure.



(a) Model 1 *(b) Model 2* *(c) Model 3*

Figure 4: Intensity measure vs. drift ratio for building models

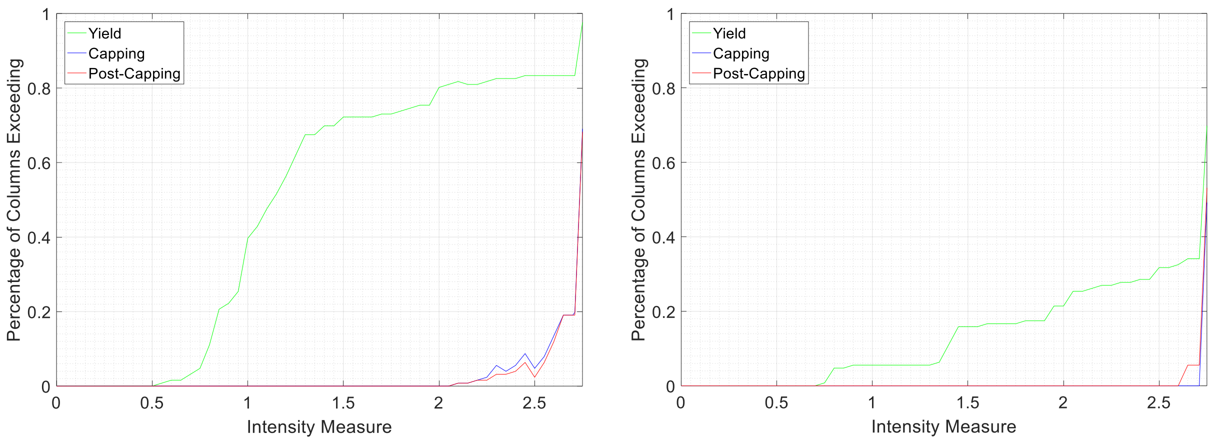
Component Damage. The effect of modeling parameters on the intensity measure at lateral instability is important to gage the vulnerability of the building and the potential for damage during strong earthquakes. In order to perform an accurate assessment of the expected level of damage and quantify resilience, it is also important to evaluate the damage expected in the building components. This is particularly important because building components can be subjected to large localized demands or there are components that are susceptible to damage at relatively low levels of deformation. Severe damage in building components can lead to irreparable damage and partial collapse of the structure, and these aspects of building response are not reflected in the curves shown in Fig. 4. For this purpose, the ASCE 41 standard has acceptance criteria for components that engineers can use to evaluate the expected level of damage at a higher level of granularity than the building response. For the purpose of this paper, a simple evaluation was performed by calculating the percentage of members that exceeded rotations at yield, capping and total loss of lateral load capacity (points B, C and E in Fig. 3).



(a) Exterior Frame *(b) Interior Frame*

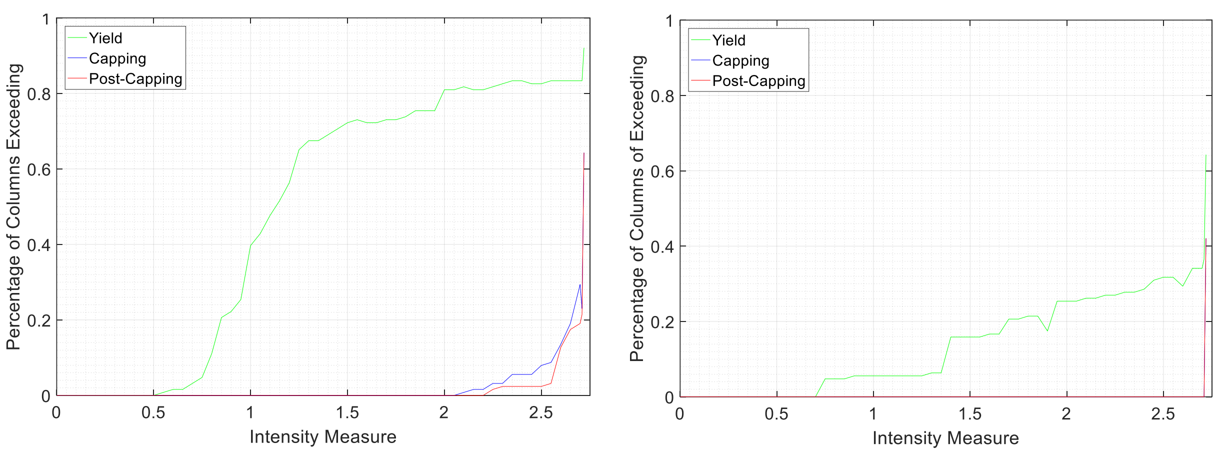
Figure 5: Percentage of column springs exceeding modeling parameters for model 1

Acceptance criteria in ASCE 41 are established based on these deformation thresholds, so they are used here to provide a coarse approximation of performance levels corresponding to immediate occupancy, life safety, and collapse prevention. Results are presented in the form of curves relating the percentage of beams and columns exceeding each deformation threshold as a function of the intensity measure (Figs. 5 through 10).



(a) Exterior Frame *(b) Interior Frame*

Figure 6: Percentage of column springs exceeding modeling parameters for model 2



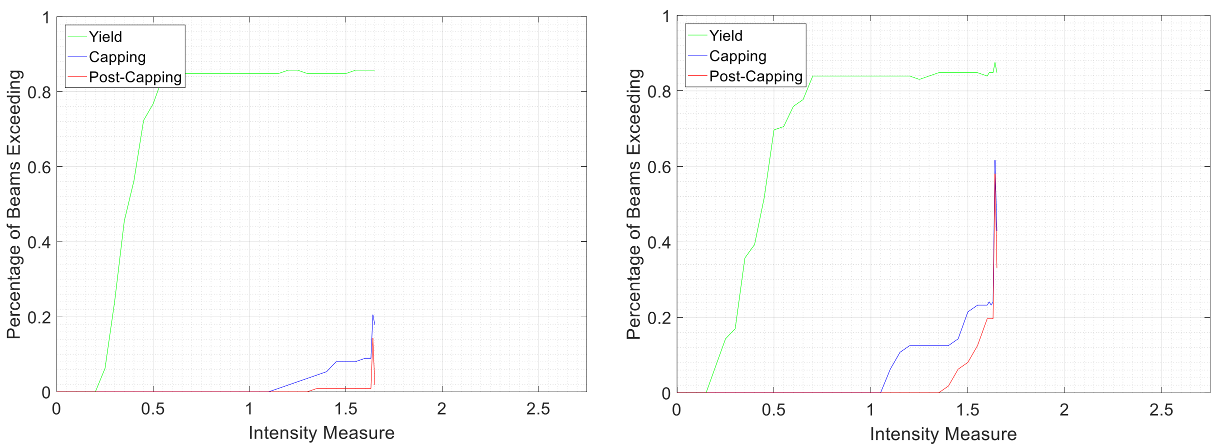
(a) Exterior Frame *(b) Interior Frame*

Figure 7: Percentage of column springs exceeding modeling parameters for model 3

Component response for building columns is summarized in Figs. 5 through 7. For model 1, with ASCE 41 column modeling parameters, a significant number of columns exceeded the capping rotation at an intensity measure of approximately 1.6, leading to a two-story mechanism involving the fourth and fifth story columns. For models 2 and 3, the larger capping rotation in the ACI 369 proposal caused shear failure and loss of lateral load capacity in the columns to begin at a significantly higher intensity measure of approximately 2.2, and progressively propagated until lateral instability occurred at an intensity measure of 2.75.

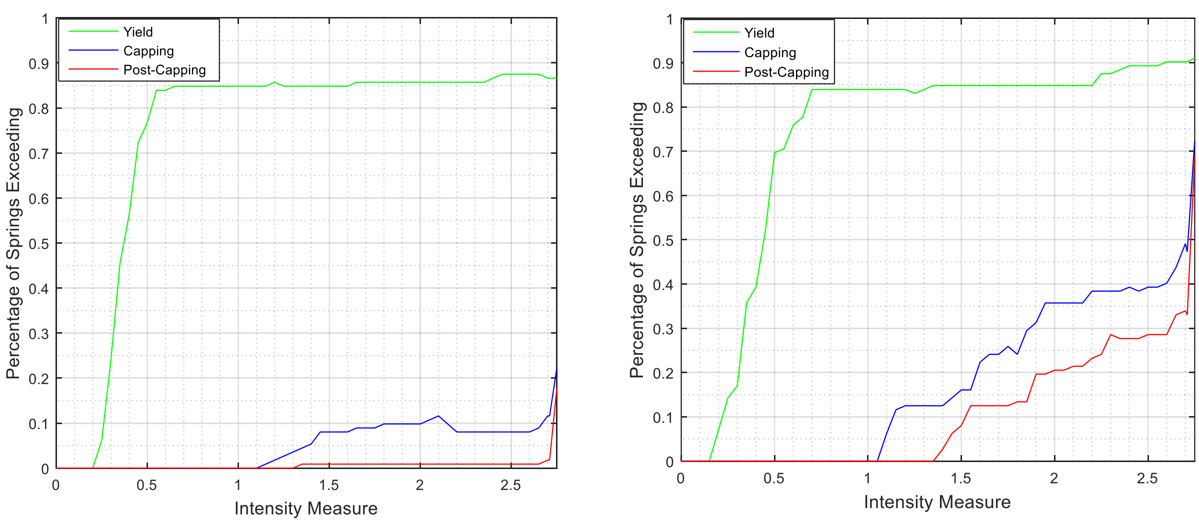
While column response is critical to the behavior of the exterior frame, beam response is critical to the behavior of the interior frames because the slab-column connections are susceptible to failure due to punching shear. Beam response is illustrated in Figs. 8 through 10. Figure 8b shows that for model 1, severe damage in the slab-column connections began at intensity measures of approximately 1.1, and that an intensity measure of 1.6 approximately 20% of the slab-column connections were under severe distress.

While the damage to the interior slab-column connections was slightly less severe for models 2 and 3, the level of distress in the slab-column connections was similar to that observed in model 1. While the change in column modeling parameters had the effect of allowing the exterior frame to maintain its load carrying capacity at much larger intensities, the larger deformation demands imposed on the interior frame would very likely lead to severe damage and loss of gravity load capacity at intensities lower than the intensity that causes lateral instability. A comparison between Fig. 9a and 10a show that implementing modeling parameters for beams based on the mean response of beam test data led to lower estimates of damage in the beams of the exterior frame.



(a) Exterior Frame *(b) Interior Frame*

Figure 8: Percentage of beam springs exceeding modeling parameters for model 1

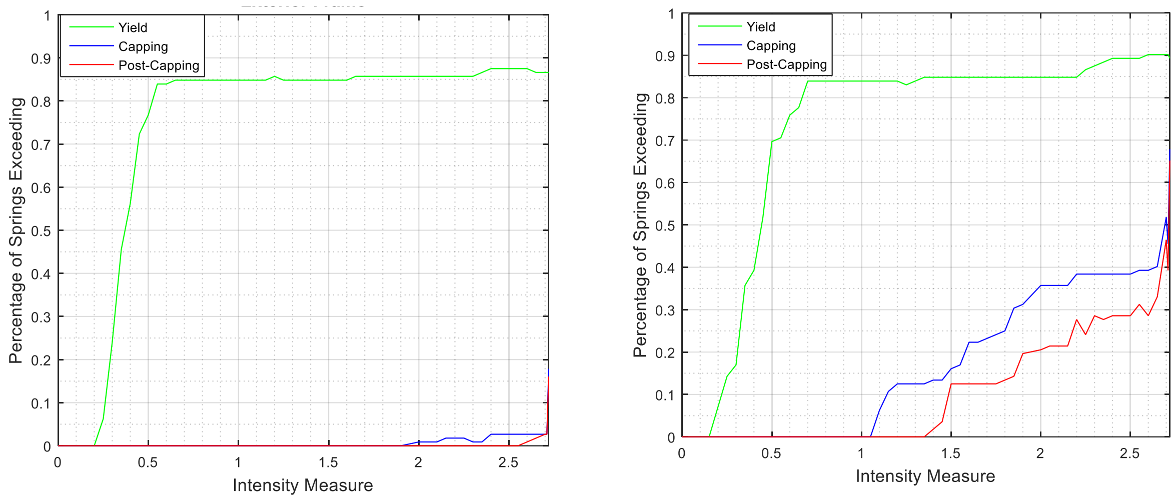


(a) Exterior Frame *(b) Interior Frame*

Figure 9: Percentage of beam springs exceeding modeling parameters for model 2

Conclusions

Simulation results showed that changes in modeling parameters for beams and columns had an important effect on the calculated nonlinear seismic response and distribution of damage of the case-study building. The intensity measure corresponding to lateral instability for the model with ASCE 41-13 modeling parameters was 1.63 (PGA = 0.77 g), whereas the maximum intensity measure for the model with ACI 369 modeling parameters was 2.71 (PGA = 1.27 g). The effect of using improved beam modeling parameters on the intensity measure corresponding to lateral instability was not significant for the case-study building, although the maximum story drift ratios before lateral instability did increase by approximately 1%. The intensity corresponding to lateral instability increased significantly with the adoption of modeling parameters representative of the mean response of component tests, which also led to a significant increase in the level of damage expected in gravity-load frames.



(a) Exterior Frame *(b) Interior Frame*

Figure 10: Percentage of beam springs exceeding modeling parameters for model 3

Acknowledgements

The authors would like to acknowledge the support of the National Science Foundation under award number 0618804 through the Pacific Earthquake Engineering Research Center (PEER).

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