PERFORMANCE OF A NONDUCTILE RC BUILDING FOR THE FEMA P695 FAR-FAULT GROUND MOTION DATA SET

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ABSTRACT

One of the limitations of the ASCE 41-13 Standard is that modeling parameters and acceptance criteria were proposed or calibrated to have different probabilities of exceedance for different types of elements, which causes inconsistencies in the seismic evaluation process. While modeling parameters were originally proposed to have a low probability of exceedance, recent updates of modeling parameters for columns were calibrated to provide the best estimate of the structural response. Although modeling parameters have changed, acceptance criteria remain similar to those proposed in FEMA 356.

This paper examines the seismic performance of an RC frame structure using a mathematical model assembled with the modeling parameters for nonlinear dynamic analysis in ASCE 41-13 and the recently balloted ACI 369 Standard. The mathematical model includes nonlinearities associated with flexural and shear failure. The seismic hazard consisted of the set of far-fault ground motions in FEMA P695, scaled to the intensity of the MCE ground motion at the building site according to the provisions in FEMA P695. Lognormal distributions for the probability of collapse due to lateral instability are presented using mathematical models with modeling parameters in ASCE 41-13 and the new ACI 369 Standard. Lognormal distributions are presented for the probabilities of achieving performance objectives of Immediate Occupancy, Life Safety, and Collapse Prevention on the basis of the acceptance criteria in ASCE 41-13 Standard.

Introduction

Older non-ductile reinforced concrete buildings built prior to 1970 have significantly higher probability of collapse than modern buildings because significant changes in building codes were

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implemented in response to the 1971 San Fernando Earthquake. Due to the potential loss of human life and damage to the economy, these buildings must be evaluated and retrofitted if necessary. Although building evaluation is inherently a creative endeavor, a set of common guidelines are important so that evaluations have a minimum level of consistency. Standards such as ASCE-41-13[1] provide guidelines and general procedures so that political authorities tasked with creating building ordinances can rely on a consistent methodology that prevents disparity between building owners.

The ASCE 41 [1] Standard has its origins in FEMA 356 [2], which in great part relied on engineering experience to formulate modeling parameters and acceptance criteria for RC elements. One of the biggest challenges in formulating those parameters in FEMA 356 was the limited experimental data for heavily damaged components. As more experimental data became available, new modeling parameters and acceptance criteria were proposed or calibrated, introducing disparities in the level of conservatism for different types of elements, which causes inconsistencies in the evaluation process. For example, while all modeling parameters were originally proposed to be conservative estimates of critical points in the response envelope, recently updated modeling parameters for columns were calibrated to provide the best estimate of the structural response. While modeling parameters have changed, acceptance criteria remain similar to those proposed in FEMA 356.

The objective of this paper is to develop a numerical model of an existing building using modeling parameters in the ASCE 41 and ACI 369 Standards, and to use these models to derive seismic fragility curves for the modeling parameters and acceptance criteria of structural components. This evaluation is helpful to understand the effect of changes in the ASCE 41-13 Standard on the outcome of building evaluations.

Building Description

The building studied for this paper is a seven-story reinforced concrete moment frame structure located in Van Nuys, California. The building was designed in 1965 and constructed in 1966 according to the prevailing codes at the time, so it lacks most of the details required by modern seismic codes. The structure consists of exterior moment-resisting perimeter frames intended to act as lateral force resisting system, and interior slab-column frames as gravity-load resisting system. Typical floor dimensions are 160 ft. in the longitudinal direction (East-West) and 64 ft. in the transverse direction (North-South). The building has eight bays in the longitudinal direction and three bays in the transverse direction. The frames have center-to-center column spacing of approximately 19.5 and 20 ft. in the longitudinal and transverse directions, respectively. The longitudinal elevation of the building is shown in Figure 1. A more detailed description of the building and the structural details is presented elsewhere [3].

This building was chosen because it was heavily instrumented and recorded several strong earthquakes, including the 1971 San Fernando and 1994 Northridge earthquakes.

Modeling Approach

The mathematical model of the building was developed using the finite element software framework, OpenSees (Open System for Earthquake Engineering Simulation) [4]. Two different models were created using the parameters in the ASCE 41-13 [3] and ACI 369 Standards. Tables with modeling parameters for different reinforced concrete elements are specified in Chapter 10 of the ASCE 41-13 Standard for nonlinear dynamic analysis. The ACI 369 Standard is intended to
be an update to Chapter 10 of the ASCE 41 Standard, and is currently under review for adoption by the ASCE 41 committee. The ACI 369 Standard includes a new set of modeling parameters for interior and exterior columns balloted by ACI Committee 369.

Effective stiffness of structural components was calculated following the provisions in the ASCE 41-13 Standard. For the interior slab-column moment frame, the effective beam model proposed by Hwang and Moehle [5] was adopted.

**Figure 1. Longitudinal elevation of case study building.**

**Building Model**

Taking advantage of the symmetrical configuration and lack of irregularities, two-dimensional frame models included only one half of the building in both the East-West and North-South directions. Each model consisted of two frames, exterior and interior, linked together with rigid link elements that allowed displacement compatibility between frames, as shown in Figure 2. Although analyses were performed for the two directions, only results from the longitudinal (East-West) direction are presented.

**Hysteretic Model**

The nonlinear behavior of beam and column elements was modeled using a lumped plasticity approach. The model consists of elastic beam-column elements with zero-length nonlinear rotational springs at each end. All inelastic deformations were assumed to be localized at the zero-length rotational springs. The moment-rotation relationship of the nonlinear rotational springs was modeled using the uniaxial material model originally developed by Ibarra et al. [6], and implemented in OpenSees by Altoontash [7]. This is a peak-oriented model which captures monotonic and cyclic deterioration of strength and stiffness. A modified version of the Ibarra-Medina-Krawinkler model, implemented in OpenSees by Lignos [8], was used in the computer simulations. Seven parameters were defined following the provisions in the ASCE 41 and ACI 369 Standards: initial stiffness ($K_e$), yield moment ($M_y$), ratio of capping to yield moment ($M_c/M_y$), plastic rotation corresponding to the capping point ($\theta_p$), plastic rotation corresponding to the point at which the post-capping slope intersects the moment axis ($\theta_{pc}$), residual moment ($M_r$) and ultimate rotation ($\theta_u$). The model simulates four modes of cyclic deterioration: basic strength...
deterioration, post-capping deterioration, accelerated reloading stiffness deterioration and unloading stiffness deterioration. Cyclic deterioration is based on an energy index that has two parameters: normalized energy dissipation capacity, $\lambda$, and an exponent term to describe the rate of cyclic deterioration with accumulation of damage, $c$. The values of these parameters were adopted from calibrations of RC columns by Haselton [9]. For the case study building, the values of $\lambda$ and $c$ were set to 135 and 1.0, respectively.

![OpenSees model of the case study building](image)

**Figure 2.** OpenSees model of the case study building

**Calibration of Numerical Models**

Different numerical models were developed by varying joint flexibility, damping ratio, and effective amount of reinforcement in the slabs. The accuracy of the numerical models was evaluated by quantifying the goodness-of-fit between measured and calculated roof displacements for records from the 1994 Northridge Earthquake using the FDE index [10]. Results in this paper correspond to building models with the best correlation between measured and calculated roof drift as indicated by the lowest FDE Index.

**Nonlinear Dynamic Analysis Procedure**

Building models with modeling parameters from the ASCE 41-13 and ACI 369 Standards were evaluated using the nonlinear dynamic procedure. An incremental dynamic analysis (IDA) [11] was performed for a representative set of earthquake records to develop fragility relationships for the elements in the moment frames. For each earthquake record, an IDA was conducted by increasing the intensity measure (IM) in increments of 0.05 to obtain the relationship between IM and inter-story drift ratio. Earthquake records used for the analyses were those specified in FEMA P695 [12], which provides far-fault and near-fault ground motion subsets. Records from these two sets were obtained from the PEER-NGA database, which contains two horizontal and one vertical component for each record in the sets. For this study, only the horizontal components of the records were used. Both far-fault and near-fault earthquake record sets were evaluated, although in this paper only the results from far-fault ground motion results are discussed. Records were scaled using the procedure described in FEMA P-695, with MCE parameters for the building site.
specified by USGS. An intensity measure of 1.0 corresponds to the intensity at the site for the MCE, at the fundamental effective period of the structure.

Acceptance Criteria

The ASCE 41 and ACI 369 Standards originate from a performance-based engineering approach with acceptance criteria for different building performance levels. A building performance objective is defined as the ability of a structure to achieve a pre-defined performance level for a given seismic hazard. Acceptance criteria are defined in these standards for performance levels of immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The acceptance criteria provided in the ASCE 41 and ACI 369 Standards are presented as maximum acceptable inelastic rotation demand for each of the three performance levels (IO, LS, and CP). Acceptance criteria in the ACI 369 Standard are similar to those in ASCE 41 with the exception of columns.

Fragility Relationships

Fragility Relationships for Collapse due to Lateral Instability

In buildings where the elements have sufficient toughness, or can at least maintain the ability to carry gravity loads after heavy damage, fragility relationships can be developed through incremental dynamic analyses to quantify the probability of building collapse due to lateral story dynamic instabilities. Analyses performed in this study simulated this behavior by allowing column elements and slab-column connections to maintain the ability to carry axial loads after loss of lateral stiffness, at inelastic deformations greater than parameter $b$ (inelastic rotation at loss of gravity load capacity) in the ASCE 41 and ACI 369 Standards. The dynamic instability limit was attained when a small increment in the intensity measure of the ground motion caused a large increment in the lateral displacement of the structure, precipitating a single or multi-story collapse mechanism. Figure 3 shows the fragility relationships for lateral instability of the mathematical models with modeling parameters from the ASCE 41-13 and ACI 369 Standards. The main difference between the two standards is that the modeling parameters in ASCE 41 were calibrated to provide a low probability of exceedance (15% for shear-critical columns and 35% for flexure-shear critical columns), while modeling parameters in the ACI 369 Standard were calibrated to provide a probability of exceedance of 50%. Consistent with higher conservatism in the ASCE 41 modeling parameters, the probability of collapse was higher compared with that of ACI 369 modeling parameters.

Fragility Relationships for Limit States

In non-ductile buildings, localized collapse may occur at lower intensities of shaking precipitated by elements that sustain sufficient damage to lose the ability to carry gravity loads. In these buildings, the IM at collapse or any other limit state corresponds to spatial distributions of inelastic demand within the structure in which a maximum number of elements exceed the intended target limit state. Two types of fragility relationships were developed in this study, the first corresponds to modeling parameters and the second to acceptance criteria, both for limits specified in the ASCE 41-13 and ACI 369 Standards. Fragility relationships were calculated for limit states corresponding to acceptance criteria for three different performance levels (IO, LS, and CP). Component inelastic rotation was used as a limit state for each performance level.

The building in this case study has nonductile details, so probabilities of reaching all modeling parameters and performance limit states were much higher than the probability of collapse due to lateral dynamic instability. This indicates that the building is most likely to have collapse initiated
by localized collapse of columns or slab-column connections.

![Graph showing probability of collapse due to lateral instability for models ACI 369 and ASCE 41 modeling parameters.](image)

**Figure 3.** Probability of collapse due to lateral instability for models ACI 369 and ASCE 41 modeling parameters.

**Fragility Relationships for Modeling Parameters**

Fragility relationships corresponding to relevant modeling parameters for columns are shown in Figure 4. In all cases, the fragility relationships presented correspond to the largest intensity IM at which all of the building components met the corresponding limit state. The probability of exceeding the yield point for columns (green lines in Figure 4) was identical for the models with ACI 369 and ASCE 41 modeling parameters. This similarity was expected because the two standards have similar provisions for effective stiffness and differ in column modeling parameters $a$ (inelastic rotation at the capping point) and $b$ (inelastic rotation at loss of gravity load capacity), which define the post-yield region of the moment-rotation relationship. The fragility relationships for parameters $a$ (blue line) and $b$ (red line) were almost the same for each of the two models. This finding suggests that in the case of both models, after the first element reaches the capping point it takes a very small increment in IM to reach the limit corresponding to loss of gravity load carrying capacity.

The fragility relationships for parameters $a$ (blue line) and $b$ (red line) differed between the two models. For example, the IM corresponding to a 50% probability of exceedance of points $a$ and $b$ for the building model with ASCE 41 modeling parameters was approximately 0.55, while the IM for the same probability of exceedance for the model with ACI 369 modeling parameters was approximately 0.65. This is consistent with the change in the calibration of the modeling parameters introduced in the ACI 369 Standard.

Fragility relationships were also calculated for the beam element subset. The probability of exceeding the yield point for the beam element subset was significantly higher than for the column subset, which is attributed to gravity load effects. The probability of at least one beam exceeding yield was nearly 100% for an IM of 0.15 MCE, while columns approached 100% at an intensity
measure of approximately 60% of MCE (Figure 4). The gap between fragility relationships corresponding to parameters $a$ and $b$ (labeled capping and post-capping) was greater for beams than it was for columns.

The calculated fragility relationships show that probabilities of reaching relevant modeling parameters for beams were higher than they were for columns, which indicates higher levels of inelastic deformation in the beams than in the columns.

![Graph](image1)

![Graph](image2)

a) Model with ACI 369 MP  

b) Model with ASCE 41 MP  

Figure 4. Probability of exceedance of column modeling parameters (MP).

**Fragility Relationships for Acceptance Criteria**

Limit state fragility relationships were calculated for building models with modeling parameters from the ASCE 41 and ACI 369 Standards, for performance levels of IO, LS, and CP. In addition, fragility relationships were calculated for zero, one, two and three elements exceeding each performance limit. These results are presented to illustrate the effect of acceptance criteria predicated on a single element in the entire structure exceeding the prescribed limit versus two or more elements allowed to exceed the prescribed limit.

Results for the column subset are presented in Figure 5. In Figures 5(a) and 5(b) the fragility relationships correspond to the building model with ACI 369 modeling parameters and zero elements exceeding the performance limits in the ACI 369 and ASCE 41 Standards, respectively. It is noteworthy that the difference between the probability of exceedance of the three performance levels was greater using acceptance criteria in ACI 369 than acceptance criteria in ASCE 41. Also, the results in Figure 5 show that the gap between the LS and CP performance levels was significantly larger for ACI 369 acceptance criteria than it was for ASCE 41 acceptance criteria. The fragility relationships corresponding to IO indicate a similar probability of exceedance for both standards.
Figure 5: Probability of exceedance for model with ACI 369 modeling parameters (MP) and zero elements exceeding acceptance criteria from ACI 369 and ASCE 41.

Table 1 shows the IM corresponding to probabilities of exceedance of 50% and 75% for the column element subset, and acceptance criteria from ASCE 41 and ACI 369. IM values in Table 1 correspond to 1, 2 and 3 column elements exceeding the IO acceptance criteria for columns. Results were similar for both standards, with an IM of approximately 0.35 for 50% probability of exceedance and 0.45 for 75% probability of exceedance. The IMs calculated had a very small increment between zero and three elements exceeding the acceptance criteria, which indicates that in this particular building a significant number of elements exceeded the acceptance criteria at approximately the same IM.

Similarly, Table 2 and Table 3 show IM values corresponding to one, two and three column elements exceeding the LS and CP acceptance criteria for columns in ASCE 41 and ACI 369. Results in Table 2 show that IMs corresponding to ACI 369 LS acceptance criteria were slightly lower (had higher probabilities of exceedance) than LS acceptance criteria in ASCE 41. The trend was reversed for the CP performance level. In this case IMs corresponding to ACI 369 CP acceptance criteria were slightly higher (had lower probabilities of exceedance) than CP acceptance criteria in ASCE 41. The effect of setting system failure based on zero, one, two, or three elements exceeding the performance limit were negligible for both LS and CP limit states, so the trend was consistent for all three limit states.

Limit state fragility relationships were calculated also for the exterior and interior (slab-column connections) beams. The probability of exceeding the IO performance limit state was significantly higher for exterior beams than it was for interior beams. A similar trend was observed for the fragility relationships corresponding to LS and CP performance limit states. In both of these cases, exterior beams had higher probabilities of exceeding the limit state. A comparison between fragility relationships for all elements show that the performance level of this building would be
governed by the performance of exterior beams and not slab-column connections or columns.

Table 1. Intensity measure for a prescribed number of column elements exceeding Immediate Occupancy column acceptance criteria in ASCE 41-13 and ACI 369.

<table>
<thead>
<tr>
<th>ASCE 41 Acceptance criteria for IO</th>
<th>ACI 369 Acceptance Criteria for IO</th>
</tr>
</thead>
<tbody>
<tr>
<td>IM for 1^2</td>
<td>IM for 2^2</td>
</tr>
<tr>
<td>50% P.O.E^1</td>
<td>0.35</td>
</tr>
<tr>
<td>75% P.O.E</td>
<td>0.45</td>
</tr>
</tbody>
</table>

^1 P.O.E refers to Probability of Exceedance
^2 1, 2 and 3 indicates 1, 2 and 3 elements exceeding the limit

Table 2. Intensity measure for a prescribed number of column elements exceeding Life Safety column acceptance criteria in ASCE 41-13 and ACI 369.

<table>
<thead>
<tr>
<th>ASCE 41 Acceptance criteria for LS</th>
<th>ACI 369 Acceptance Criteria for LS</th>
</tr>
</thead>
<tbody>
<tr>
<td>IM for 1</td>
<td>IM for 2</td>
</tr>
<tr>
<td>50% P.O.E</td>
<td>0.56</td>
</tr>
<tr>
<td>75% P.O.E</td>
<td>0.69</td>
</tr>
</tbody>
</table>

Table 3. Intensity Measure for a prescribed number of column elements exceeding Collapse Prevention column acceptance criteria in ASCE 41-13 and ACI 369.

<table>
<thead>
<tr>
<th>ASCE 41 Acceptance criteria for CP</th>
<th>ACI 369 Acceptance Criteria for CP</th>
</tr>
</thead>
<tbody>
<tr>
<td>IM for 1</td>
<td>IM for 2</td>
</tr>
<tr>
<td>50% P.O.E</td>
<td>0.58</td>
</tr>
<tr>
<td>75% P.O.E</td>
<td>0.72</td>
</tr>
</tbody>
</table>

Conclusions

This study shows the effects of changes in modeling parameters and acceptance criteria between the ASCE 41-13 and the ACI 369 standards. At the time this paper is written, the ACI 369 Standard has undergone public discussion and is in the process of being printed. Changes introduced in the ACI 369 Standard are being balloted for adoption by the ASCE 41 Committee.

The fragility relationships calculated in this paper show that the case study building, due to the lack of ductile detailing, will reach collapse at much lower earthquake intensities than would cause dynamic instabilities.

Changes in the ACI 369 Standard had the desired effect of creating a more realistic representation of building behavior without an abrupt change in acceptance criteria. Comparison of fragility relationships based on ASCE 41 and ACI 369 standards show that acceptance criteria for IO were very similar, although the changes in acceptance criteria were noticeable for the LS and CP limit.
states. In the opinion of the authors the greater gap between the curves corresponding to LS and CP observed for the ACI 369 acceptance criteria is a better representation of performance objectives defined in Chapter 2 of ASCE 41 where, for example, the Enhanced Objective corresponds both to seismic hazard level BSE-1 (10% probability of exceedance in 50 yrs) with performance level LS and seismic hazard level BSE-2 (2% probability of exceedance in 50 years) with performance objective CP.

One of the concerns that is apparent in this evaluation is that the performance level of the case study building was controlled by the exterior beams, which are the elements of least concern to the gravity load system. The findings from this paper suggest that a broader approach to acceptance criteria, in which the effect of element damage on the probability of collapse is considered, should be developed.

Acknowledgments

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