USING FEMA P695 TO INTERPRET ASCE 41 SEISMIC PERFORMANCE OF SPECIAL MOMENT FRAMES

M.S. Speicher¹, J. Dukes², and K.K.F. Wong³

ABSTRACT

ASCE 41 contains methodologies used by practicing engineers for the assessment of existing buildings and the design of new buildings. The National Institute of Standards and Technology recently completed a study investigating the relationship between ASCE 41 and traditional design standards (e.g., ASCE 7 and AISC 341). The results indicated there are many inconsistencies between the two approaches, with some unwarranted conservatism in ASCE 41. To further investigate this relationship, this paper presents the results of a collapse assessment of a set of four steel special moment frames. The goal is to verify that the design intent is being met (i.e., no greater than 10% probability of collapse given a risk-targeted maximum considered earthquake). The effects of modeling assumptions, such as using default ASCE 41 backbone curves versus experimentally-derived backbone curves, is discussed. The results are used to scrutinize the performance indicated by the ASCE 41 assessment. In general, the performance indicated by an ASCE 41 assessment is shown to be conservative relative to the collapse performance indicated by a FEMA P695 assessment.

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ASCE 41 contains methodologies used by practicing engineers for the assessment of existing buildings and the design of new buildings. The National Institute of Standards and Technology recently completed a study investigating the relationship between ASCE 41 and traditional design standards (e.g., ASCE 7 and AISC 341). The results indicated there are many inconsistencies between the two approaches, with some unwarranted conservatism in ASCE 41. To further investigate this relationship, this paper presents the results of a collapse assessment of a set of four steel special moment frames. The goal is to verify that the design intent is being met (i.e., no greater than 10% probability of collapse given a risk-targeted maximum considered earthquake). The effects of modeling assumptions, such as using default ASCE 41 backbone curves versus experimentally-derived backbone curves, is discussed. The results are used to scrutinize the performance indicated by the ASCE 41 assessment. In general, the performance indicated by an ASCE 41 assessment is shown to be conservative relative to the collapse performance indicated by a FEMA P695 assessment.

**Introduction**

Performance-based seismic design is intended to let the structural engineer develop earthquake-resistant building design solutions that are more efficient and cost-effective than those obtained using the prescriptive building code requirements found in ASCE 7 [1]. ASCE 41 [2] contains performance-based methodologies used by some practicing engineers for both the assessment of existing buildings and the design of new buildings. The National Institute of Standards and Technology (NIST) recently completed a study [3-5] investigating the relationship between ASCE 41 and traditional design standards (e.g., ASCE 7 and AISC 341 [6]) for a variety of framing systems, including steel special moment frames. In general, the study showed inconsistencies between the two approaches, with some potentially unwarranted conservatism in ASCE 41. It was recognized that to more fully contextualize the results and to see if the intended performance

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objectives of ASCE 7 is being met, the collapse likelihood of each building subjected to strong earthquake ground motions needs to be determined.

This paper presents the results of the collapse assessment of four special moment frames originally investigated in the NIST study. First the methodology is discussed in terms of setting up the models and determination of collapse fragilities. The buildings are modeled in OpenSees [7] using commonly accepted approaches. FEMA P695 [8] is used as a guide in conducting analysis to determine the probability of collapse in a “standardized” manner. The results indicate that these ASCE 7-designed frames meet the intent of the code. This helps contextualize the observations made in Harris and Speicher [3], which showed that, in many cases, ASCE 41 is overly conservative.

Methodology

Four steel special moment framed buildings are assessed in this paper. The modeling details are given in Fig. 1 and the member sizes are given in Table 1. Harris and Speicher [3] conducted an ASCE 41 performance assessment using 3D models in Perform-3D. Given the cost of running incremental dynamic analysis, 2D models were created in OpenSees. It was found that the 2D and 3D models matched well, which is not surprising given the buildings are symmetric and the lateral systems in each direction are non-intersecting.

Figure 1. OpenSees modeling details for the 4- and 8-story special moment frames.
The moment frames have reduced beam section (RBS) connections. The RBS stiffness was modeled using a prismatic cross-section over the length of the RBS (denoted as \(b\)). The width of the RBS was assumed to be equivalent to the actual RBS width at \(b/3\) away from the center of the RBS. The nonlinear behavior was captured by a nonlinear rotational spring placed at the RBS center. The nonlinear spring was assigned a stiffness of 10 times that of the unreduced beam. Since this spring is in series with the elastic beam elements, the elastic beam stiffness was increased to give an overall beam (including the RBS and nonlinear spring) stiffness equal to that without the nonlinear spring (see [9] for more discussion on this approach).

**Table 1.** Member sizes for 4-story and 8-story frames.

<table>
<thead>
<tr>
<th>Beam</th>
<th>ELF</th>
<th>RSA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Size</td>
<td>Size</td>
</tr>
<tr>
<td></td>
<td>(a) (mm)</td>
<td>(b) (mm)</td>
</tr>
<tr>
<td>4-story</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td>W24×84</td>
<td>W24×55</td>
</tr>
<tr>
<td>B2</td>
<td>W24×84</td>
<td>W24×55</td>
</tr>
<tr>
<td>B3</td>
<td>W24×76</td>
<td>W24×55</td>
</tr>
<tr>
<td>B4</td>
<td>W24×55</td>
<td>W24×55</td>
</tr>
<tr>
<td>8-story</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td>W27×114</td>
<td>W24×55</td>
</tr>
<tr>
<td>B2</td>
<td>W27×114</td>
<td>W24×55</td>
</tr>
<tr>
<td>B3</td>
<td>W27×94</td>
<td>W24×55</td>
</tr>
<tr>
<td>B4</td>
<td>W24×55</td>
<td>W21×44</td>
</tr>
<tr>
<td>Column</td>
<td>ELF</td>
<td>RSA</td>
</tr>
<tr>
<td></td>
<td>Column</td>
<td>ELF</td>
</tr>
<tr>
<td>4-story</td>
<td></td>
<td></td>
</tr>
<tr>
<td>E1</td>
<td>W14×159</td>
<td>W14×257</td>
</tr>
<tr>
<td>E2</td>
<td>W14×145</td>
<td>W14×233</td>
</tr>
<tr>
<td>8-story</td>
<td></td>
<td></td>
</tr>
<tr>
<td>E1</td>
<td>W18×175</td>
<td>W18×258</td>
</tr>
<tr>
<td>E2</td>
<td>W18×158</td>
<td>W18×258</td>
</tr>
<tr>
<td>E3</td>
<td>W18×130</td>
<td>W18×234</td>
</tr>
<tr>
<td>E4</td>
<td>W18×71</td>
<td>W18×143</td>
</tr>
</tbody>
</table>

Note: ELF = equivalent lateral force, RSA = response spectrum analysis. ELF and RSA were created for each building height.

OpenSees can capture cyclic and in-cycle degradation using the Ibarra Modified Krawinkler (IMK) model with the Bilin material. The force-deformation parameters for the RBS followed the recommendations made by Lignos [10], which were derived using multivariate regression analysis of a suite of experimental results. These parameters include the plastic rotation capacity, \(\theta_p\), the post-capping rotation capacity, \(\theta_{pc}\), the yield strength, \(M_y\), the capping strength, \(M_c\), the ultimate rotation, \(\theta_u\), the residual strength ratio, \(\kappa\), and the reference cumulative plastic rotation parameter, \(\Lambda\).

Nonlinear springs were added to the columns at \(d/2\) away from the face of the column (see Fig. 1), where \(d\) is the depth of the column. These nonlinear springs followed the same approach (10 times stiffness) as that discussed for the beams. The force-deformation parameters for the column hinges follow the recommendations produced by ATC project 114 [11] using the monotonic backbone (since our model captures degradation). The panel zones were modeled explicitly using the approach outlined by Krawinkler [12]. A series of “rigid” elements with pinned connections was placed at the panel zone region with one corner being tied together with nonlinear rotational spring.
The spring parameters were based on fundamental mechanics. Additionally, though the column splice was designed at 1.2 m above the beam-to-column joint, this was ignored in the model.

To illustrate the difference in force-deformation behavior between the OpenSees model (used in this study) and the Perform-3D model (used in [3]), a subassembly model was created in both platforms as shown in Fig. 2. We assumed doubly-symmetric bending for both the beams and columns, thus zero moment at the mid-spans. As shown in Fig. 3, assuming the columns remain elastic, the (a) total drift can be broken down into the following actions: (b) elastic deformation of the beams and columns, (c) inelastic behavior of the beam hinges, and (b) joint panel zone deformation. The results of a displacement-controlled reversed cyclic simulation are shown in Fig. 4. The member properties and RBS hinge properties are those of the 4-story ELF design and are from the locations as indicated in Fig. 1.

The force-displacement behavior reasonably matches for drift levels up to approximately 5%. Beyond this level, the Perform-3D model loses all strength and the response travels along the zero-force level. The Perform-3D model’s force-deformation curve was created using the default \( a, b \) and \( c \) parameters given in ASCE 41-06 Table 5-6 [13]. The endzone stiffness parameter used in Perform-3D was set to 2.5, which accounts for a portion the stiffness differences between the two models. Perform-3D also includes elastic shear deformations and the OpenSees model does not, which contributes to the stiffness difference.

Note that the Perform-3D force-displacement response at the top of the subassembly has a sudden post-capping strength decrease due to the RBS hinge (even though the RBS inelastic spring has a more gradual decrease). This can be explained by examining the drift contributions as conceptually illustrated in Fig 5. At step k, the force is shown reaching the capping (peak) level. Once the strength starts to drop on the subassembly, say at step \( k+1 \), the remaining elastic elements have a reduced contribution to the drift thus the effective drop is sudden. Note, we assume displacement \( \Delta_{k+1} \) is only slightly larger than \( \Delta_k \), though Fig. 5 shows this exaggerated. Figure 4 shows the actual behavior for the Perform-3D model with the sudden drop.

In contrast, the OpenSees subassembly model has a gradual post-capping strength deterioration behavior. Lignos and Krawinkler [14] reported that the slope of this degradation is one of the most
important parameters that influences the nonlinear response. The slope is established by $\theta_{pc}$. This descending branch continues until the force-deformation reaches the residual strength or the ultimate rotation. For this paper, $\theta_u$ was taken as 0.20 radians for all hinges. Lignos and Krawinkler [10] reported $\theta_u$ values of approximately 0.07 radians for fully-reversed cyclic tests, but monotonic tests may be up to 3 times larger. The residual strength factor, $\kappa$, was taken as 0.4, which is double the value given in ASCE 41 and used in Harris and Speicher [3]. Note, the values of $\theta_u$ and $\kappa$ have been reported to be non-critical in studies where collapse capacities are being determined [10].

Figure 3. Drift contributions for a moment frame subassembly.

Figure 4. Comparison of the response of subassembly to cyclic loading for OpenSees and Perform-3D (model used in Harris and Speicher [3]).

Figure 5. Explanation of drift between (a) some point, $k$, at the capping moment and (b) some point, $k+1$, just beyond the capping moment.
Fragility Curve Development

To estimate each building’s collapse fragility, incremental dynamic analysis (IDA) was carried out. IDA involves analyzing a structural model to multiple ground motion records that are scaled over a range of levels to produce a response curve [15]. The suite of ground motions used for the IDA is referred to as the “far-field” set in FEMA P695. This set consists of 22 record pairs from sites that were more than 10 km from the fault rupture. Given our 2D model, we applied each pair independently, resulting in 44 earthquake runs at each intensity level. The ground motions were first normalized per FEMA P695 using the FEMA 695 Toolkit [16] and then scaled up to the risked-targeted maximum considered earthquake (MCER).

The analyses were performed using the parallel version of OpenSees, OpenSeesMP, on the Extreme Science and Engineering Discovery Environment (XSEDE) platform [17]. The initial number of dynamic analyses for each building type was 44 earthquakes times 20 scale factors (0.1 to 2.0). If the structure did not reach the collapse criteria for any earthquake by the last scale factor, additional analyses were performed at higher scale factors until collapse was achieved.

The ground motion intensity measure (IM) used in this paper is the median spectral acceleration of the suite of ground motions at the fundamental period \( C_uT_a \) of the building, \( S_T \). Collapse was defined as reaching 12.5 % interstory drift in any story. The IM at which the structure first reaches 12.5 % interstory drift is used in the calculation of the fragility curve. To develop the fragility curve, the IMs at which the structure collapses for each ground motion is aggregated across the ground motion suite and then is used to determine the fragility curve. The fragility function was calculated using “Method A” as described by Porter et al. [18], where the formulation is based on scaling all the ground motions until all specimens have collapsed. Equation 1 shows the calculation of the fragility function, where \( P(C \mid IM = x) \) is the probability of collapse of the structure at IM \( x \).

\[
P(C \mid IM = x) = \Phi\left(\frac{\ln(x / \theta)}{\beta}\right)
\]  

(1)

The normal cumulative distribution function is denoted by \( \Phi \), \( \theta \) represents the median of the fragility function, and \( \beta \) is the standard deviation of \( \ln(IM) \). Equations 2 and 3 show the formulation of the fragility function estimators \( \hat{\theta}, \hat{\beta} \) over \( n \) number of earthquakes, which are method of moments estimators of a normal distribution (Baker, 2015).

\[
\ln \hat{\theta} = \frac{1}{n} \sum_{i=1}^{n} \ln IM_i
\]  

(2)

\[
\hat{\beta} = \sqrt{\frac{1}{n-1} \sum_{i=1}^{n} (\ln(IM_i / \hat{\theta}))^2}
\]  

(3)
Results

The results are presented in the form of IDA curves and associated fragility curves. Fig. 6 shows the IDA curves for the 4 and 8-story ELF and RSA models. The IDA curves illustrate the progression of damage as intensities are increased. After approximately 7.5% interstory drift, a small increase in intensity generally results in a large increase in interstory drift. Therefore, the choice of collapse being defined as 12.5% interstory drift is deemed sufficient. Figure 7 shows the fragility curves of the 4- and 8-story ELF and RSA designed buildings. From these curves, the probability of collapse at the MCE\_R can be estimated. The MCE\_R spectral accelerations at $C_u T_a$ for the 4-story and 8-story buildings are 0.87 g and 0.51 g, respectively.

![Diagram](image)

(a) (b) (c) (d)

Figure 4. Incremental dynamic analysis curves for the (a) 4-story ELF-designed, (b) 4-story RSA-designed, (c) 8-story ELF-designed, and (d) 8-story RSA-designed frames.

Discussion

To understand the implications of the fragility curves, Table 2 shows a summary of results reported by Harris and Speicher [3]. For both the nonlinear static and dynamic procedures, at least one RBS connection fails the assessment in one out of four frames. For the nonlinear dynamic procedure, the assessment indicates worse performance with at least one RBS connection failing the criteria in three out of four frames. The column and panel zones also show failures, but the discussion presented here is focused on the RBS connection because the newest version of ASCE 41 (i.e., ASCE 41-17) has updated modeling assessment criteria that may correct the problems seen in the columns. Recall that the design intent given in ASCE 7 is that a building will have a probability
of collapse given an MCE$_R$ of less than or equal to 10%. However, the FEMA P695 methodology is intended to be for a suite of archetype buildings. If each of these buildings are taken as individuals, FEMA P695 suggests that a code-complying building may be acceptable if the probability of collapse given an MCE$_R$ reaches as high as 20%. The IDA and the resulting fragility curves shown in Fig. 7 suggest that the 4-story buildings and the 8-story ELF-designed building are adequate in terms of collapse probability. In contrast, RSA-designed 8-story frames has a probability of collapse of approximately 19%. Though this frame does not meet the 10% goal of a code-designed building as determined by the methodology described herein, it does fall below the 20% limit mentioned above. Further investigation is being carried out to verify these results and determine any deficiencies that could be addressed to help reduce the RSA-design collapse probabilities.

![Fitted fragility curves for the (a) 4-story ELF-designed, (b) 4-story RSA-designed, (c) 8-story ELF-designed, and (d) 8-story RSA-designed frames.](image)

Given these findings, it appears ASCE 41 is overly-conservative at identifying performance issues in the set of frames, though the trends between frames are similar. One issue that needs further investigation is the conservative nature of the Perform-3D model which used modeling parameters from ASCE 41. This model has a backbone curve with a relatively steep descending branch beyond the capping point, which results in an almost vertical drop in stiffness for a subassembly. As mentioned before, the nonlinear response has been shown to be sensitive to this slope. ASCE 41 does not give clear guidance on the post-capping slope, and in this case the approach used in [3] may be conservative and result in unfavorable performance, especially for the nonlinear dynamic procedures (as opposed to the nonlinear static procedures). Other reasons for conservatism are
explored in a companion study looking at other steel framing systems [19-21], which includes (1) the method used for selecting and scaling ground motions and (2) ASCE 41 modeling parameters, derived from fully-reversed cyclic loading protocols, which may not be a true representation of buildings at MCE\(_R\) or greater level earthquakes [22]. Regardless, the results presented in this paper provides further context to the results seen in Harris and Speicher [3] as summarized in Table 2.

Table 2. Summary of performance predicted using the nonlinear procedures in ASCE 41 for the 4- and 8-story buildings (the performance is for Collapse Prevention under Basic Safety Earthquake-2 hazard level) [3].

<table>
<thead>
<tr>
<th>Building</th>
<th>Design Approach</th>
<th>Nonlinear Static Procedure</th>
<th>Nonlinear Dynamic Procedure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RBS connection</td>
<td>Column Panel Zone</td>
<td>RBS connection Column Panel Zone</td>
</tr>
<tr>
<td>4-Story</td>
<td>ELF Pass Pass Pass</td>
<td></td>
<td>Pass Pass Pass</td>
</tr>
<tr>
<td></td>
<td>RSA × Pass Pass</td>
<td></td>
<td>× Pass Pass</td>
</tr>
<tr>
<td>8-Story</td>
<td>ELF Pass × Pass</td>
<td></td>
<td>× × Pass</td>
</tr>
<tr>
<td></td>
<td>RSA Pass × Pass</td>
<td></td>
<td>× × ×</td>
</tr>
</tbody>
</table>

In contrast to the Perform-3D model, the OpenSees model uses a more sophisticated and less conservative approach. The difference in backbone curves can be seen in the subassembly discussion presented in the Methodology section. Both in-cycle and cyclic degradation was accounted for therefore a monotonic backbone curve was used. This resulted in more energy being dissipated in the hysteresis and a more gradual strength loss beyond the capping point. Further investigation into the sensitivity of the modeling parameters and analysis approaches may be beneficial to fully vet these results.

Conclusion

A set of buildings designed using ASCE 7 and assessed using ASCE 41 were shown to be deficient (per ASCE 41) in a previous study by NIST. The results appeared to be conservative for the nonlinear dynamic procedure in ASCE 41. This paper presents a preliminary study of the collapse probability of four special moment frames using incremental dynamic analysis. The resulting fragility curves suggest three out of four frames has a less than or equal to 10 % probability of collapse given a risk-targeted maximum considered earthquake, therefore meeting the intent of ASCE 7. The remaining frame has a greater than 10 % probability of collapse, thus not satisfying the intent of the code (though the 8-story RSA-designed frame does not exceed 20 %, which can be considered the limit for an individual building). We note the high-level similarities and the differences in the results for the ASCE 41 assessment and the IDA, and suggest that implementing the ASCE 41 modeling parameters in a Perform-3D model may result in a conservative assessment outcome.

Disclaimer and Acknowledgements

Commercial software may have been used in the preparation of information contributing to this paper. Identification in this paper is not intended to imply recommendation or endorsement by NIST, nor is it intended to imply that such software is necessarily the best available for the purpose. No formal investigation of uncertainty or error is included in this study. This work used the Extreme Science and Engineering Discovery Environment (XSEDE) through allocation TG-
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