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COLLAPSE ESTIMATES OF U.S. CODE-COMPLIANT STEEL FRAMES AND IMPLICATIONS FOR AN ASCE 41 ASSESSMENT

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Abstract

ASCE 41 is a standard that contains performance-based engineering procedures sometimes used to assess and retrofit existing structures. In 2015, the U.S. National Institute of Standards and Technology completed a study investigating the relationship between ASCE 41 and traditional new building design standards. A key observation from this study was that there are inconsistencies between the two approaches, some of which may be caused by suspected conservatism in the ASCE 41. To further investigate this relationship, this study will present the results of a FEMA P695 assessment of a set of six steel special moment frames. The goal is to verify that the ASCE 7 design intent of no greater than 10 % probability of collapse given a risk-targeted maximum considered earthquake is being met. The effects of various modeling assumptions, such as backbone curves, damping, and p-delta columns, are discussed. The sensitivity of these modeling approaches is summarized, and it is found that consideration of the beam-slab composite action is most significant. Furthermore, the tendency for conservative assumptions for one component resulting in non-conservative results for another component is highlighted. It is expected this study will provide useful and timely information to engineers and standards committee members charged with improving performance-based seismic design approaches.

Keywords: Performance-based design, ASCE 41, collapse, FEMA P695, steel moment frames

1. Introduction

One general goal of performance-based seismic design is to allow an engineer to make design decisions outside the typically prescriptive requirements of traditional building codes and standards. In theory, this gives the engineer freedom to make creative and/or efficient choices that produce a well-performing and cost-effective structure. Currently in the U.S., the American Society of Civil Engineers (ASCE) standard 41 titled *Seismic Evaluation and Retrofit of Existing Buildings* is the consensus document used for both assessment of existing buildings and alternative, performance-based design of new buildings [1]. This standard is often simply referred to as ASCE 41. In contrast, most new buildings are constructed using design procedures from ASCE 7, *Minimum Design Loads and Associated Criteria for Buildings and other Structures* [2] and associated materials specific specifications such as the American Institute of Steel Construction (AISC) publication 360 titled *Specification for Structural Steel Buildings* [3] and publication 341 titled *Seismic Provisions for Structural Steel Buildings* [4].

Given these different available approaches for the design of new buildings, researchers at the National Institute of Standards and Technology (NIST) conducted a study investigating the relationship between ASCE 41 and the corresponding new building standards/specifications. The findings of the NIST investigation were published in a comprehensive series of reports [5-7] and journal papers [8-11] over the last several years. An over-arching conclusion from the NIST investigation is that ASCE 41 tends to be overly-conservative and new provisions should be considered to better align ASCE 41 with new building standards/specification. However, the authors noted even though their study showed, in many cases, a building designed with the current conventional standards and specifications would fail an ASCE 41 assessment, there is reason to question whether this is, in fact, a misleading conclusion. In other words, though the buildings were properly designed to meet or exceed the requirements of ASCE 7, AISC 360, and AISC 341, there is no guarantee the building would actually meet the performance intent of the building code, which is a no greater than 10 % probability of collapse given a risk-targeted “maximum considered earthquake.”

Therefore, this paper investigates the NIST-designed special moment frames to more fully-contextualize the conclusions made in previous studies. This is done using a rigorous methodology established by the Federal Emergency Management Agency (FEMA) publication P695 titled *Quantification of Building Seismic Performance Factors* [12]. To quantify the performance, FEMA P695 provides an approach that incrementally increases the earthquake intensity to determine a structure’s collapse behavior. This process is repeated for a set of 44 earthquake records (22 pairs), and then the distribution of collapses is plotted relative to intensity measure to create a lognormal cumulative distribution function know as a fragility curve. The basis of this methodology is commonly referred to as incremental dynamic analysis (IDA). To facilitate efficient deployment of IDA, new nonlinear building models are created in OpenSees [13]. The OpenSees models also allow the use of current state-of-the-art nonlinear degradation models, which are important when considering collapse-level shaking.

2. Methodology

The suite of archetype steel buildings come from Harris and Speicher [5]. These buildings have a special moment frame (SMF) in the east-west direction and a special concentrically braced frame in the north-south direction. As mentioned previously, only the special moment frames are investigated in this study. For each building height, the frames were designed using the equivalent lateral force (ELF) procedure and the modal response spectrum analysis (RSA) procedure. Fig. 1 gives the general floor framing layout. Fig. 2 gives the member sizes for the moment frames, including the reduced beam section (RBS) dimensions. The floors are assumed to be composite slabs with 5 in (12.7 cm.) concrete slab. The stiffness of the slab is considered for both the design and assessment of the building, but composite-slab interaction is not considered when modeling the beam hinges for the baseline analysis. The effects of composite slab interaction are discussed in the follow-on NIST report related to this work [14]. The wide-flange sections are assumed to be A992 Grade 50 steel. Further design details can be found in Harris and Speicher [5].

Details of the nonlinear models used are shown in Fig. 3. The models used in Harris and Speicher [5] were created in Perform-3D [15]. Given the computational expense of running IDA, the three-dimensional Perform-3D models were converted into two-dimensional OpenSees models. This was advantageous both in terms of efficiency and enabling the use of nonlinear deterioration models that reflect state-of-the-art research. A rigorous comparison between the three-dimensional Perform-3D model and the two-dimensional OpenSees model and the results show reasonable agreement (see [14] for full details).

The moment frames have reduced beam section (RBS) connections with stiffness defined using a prismatic cross-section over the length of the RBS (length *b* in Fig. 3). The width of the RBS was approximated as the actual RBS width at *b*/3 away from the center of the RBS. A nonlinear rotational spring was placed at center of the RBS and 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 [14] Appendix D for further discussion). The nonlinear spring was assigned the modified Ibarra Medina Krawinkler (IMK) model which uses the OpenSees Bilin material model. The force-deformation parameters followed recommendations from Lignos and Krawinkler [16].

For the columns, nonlinear springs were placed half the depth of the column (d / 2) away from the face of the beam. These nonlinear springs followed the same approach as the beams. The force-deformation parameters followed the recommendations from NIST [17] using a monotonic backbone plus accompanying degradation. At the intersection of each beam and column, the panel zones were modeled explicitly using an approach outlined by Krawinkler [18] – which includes a set of “rigid” elements with pinned connections tied together with nonlinear rotational spring in one corner. The spring parameters were based on fundamental mechanics. Though the column splice was designed at 4 ft (1.2 m) above the beam-to-column joint, this was ignored in the model.

To account for the gravity frame in the model, a P-delta column is added and connected to the moment frame with rigid links at each story. The moment of inertia of the P-delta column was taken as tributary sum of the moment of inertias of the gravity and SCBF columns to account for the stiffening effect of the frame outside of the SMF. For simplicity, the P-delta column was assumed elastic along the height of the building. Additionally, the nonlinear models are given 3 % modal damping for all modes and 0.3 % stiffness proportional damping to damp out spurious higher modes. This damping selection aligns the OpenSees models with the models used in Perform-3D by Harris and Speicher [5].

IDA was performed to determine each frame’s collapse fragility. The IDA followed guidelines set forth in FEMA P695. A suite of 44 ground motion records (22 pairs) were used in the IDA, which are referred to as the “far-field” set in FEMA P695. The intensity measure used for the IDA is the median spectral acceleration of the earthquake suite at the fundamental period of the building, C_uT_a. Collapse is defined as any story drift exceeding 7.5 % or when the analysis failed to converge.

To determine the fragility curve from the IDA results, the probability of collapse at intensity measure x, $P(C \mid IM = x)$, can be calculated in Eq. (1) as follows:

$$P(C \mid IM = x) = \Phi\left(\frac{\ln(x/\theta)}{\beta}\right) \tag{1}$$

where the normal cumulative distribution function (CDF) is denoted by Φ, the median of the fragility function is denoted by θ, and the standard deviation of ln(*IM*) is denoted by β. Eqs. (2) and (3) show the formulation of the fragility function estimators (designated with the “hat” marking) over *n* number of earthquakes, which are method of moments estimators of a normal distribution [19].

$$\ln \hat{\theta} = \frac{1}{n} \sum_{i=1}^{n} \ln \left(IM_i \right) \tag{2}$$

$$\hat{\beta} = \sqrt{\frac{1}{n-1} \sum_{i=1}^{n} \left(\ln \left(IM_i / \hat{\theta} \right) \right)^2} \tag{3}$$

The analyses were performed in part using the parallel version of OpenSees, OpenSeesMP, on the Extreme Science and Engineering Discovery Environment (XSEDE) platform [20] and on local NIST computers.

After the IDA is completed, collapse margin ratios (CMRs) can be determined directly from the fragility curves. The intensity measure of interest, S_T, is defined as the median spectral acceleration of the record set at period T. The CMR is calculated as follows:

$$CMR = \frac{\hat{S}_{CT}}{S_{MT}} \tag{4}$$

where $\hat{S}_{CT}$ is the median collapse intensity (i.e., the value of S_T that results in 50 % of the ground motions causing collapse) and S_{MT} is the value of the MCE_R spectrum at period T.

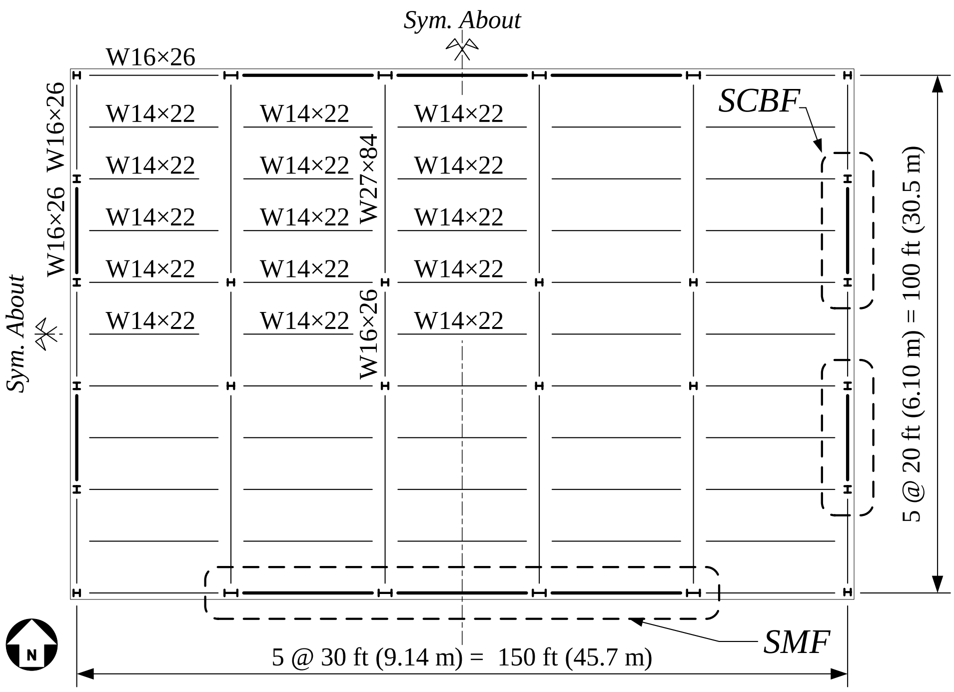

Fig. 1 – Typical building floor plan showing the structural framing layout

Fig. 2 – The six SMF designs investigated in this study [14]

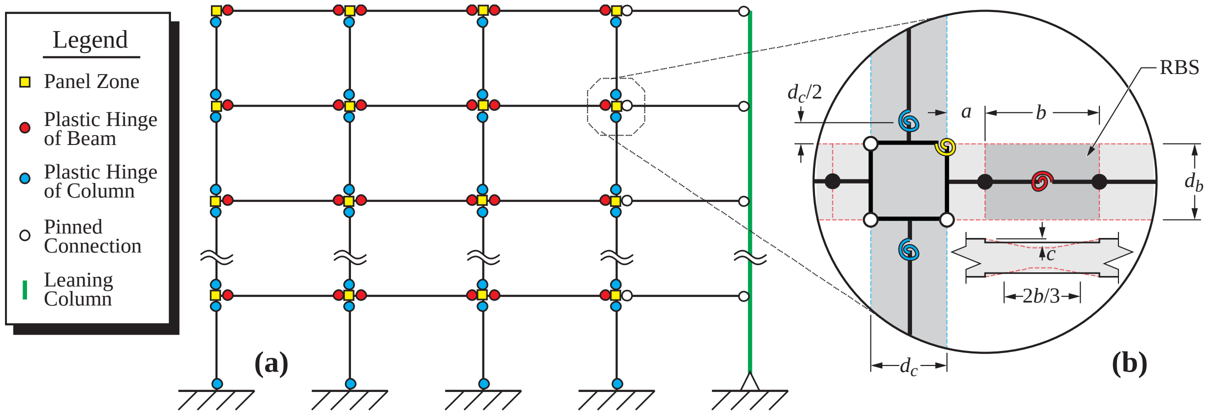

Fig. 3 – OpenSees modeling details for the special moment frames

3. Results

The results of the IDA are shown in terms of maximum story drift ratio versus normalized intensity measure S_T / S_{MT} and in terms of a fitted cumulative distribution function. Fig. 4 shows the IDA curves for the 4, 8, and 16-story ELF and RSA-designed frames. The IDA curves illustrate the progression of nonlinear response as the intensity of each earthquake is increased. Recall that the intensity measure, S_T, is the median spectral acceleration of the suite of earthquakes, not the spectral acceleration for any individual earthquake. Fig. 5 shows the fragility curves for the 4, 8, and 16-story ELF and RSA-designed frames. Since S_{MT} is the spectral acceleration at the MCE_R level, it can be quickly deduced what the probability of collapse is given an MCE_R. However, spectral shape and other variability has not yet been accounted for, which is necessary to complete the FEMA P695 process. Therefore, the median spectral acceleration (P(Collapse) = 0.5) is the value determined from the fragility curve; this value is referred to as the collapse margin ratio, *CMR*.

4. Discussion

To complete the FEMA P695 assessment, the *CMR* from Fig. 5 needs to be adjusted to account for the frequency content of the earthquake. The period-based ductility factor, μ_T, considers the period elongation caused by yielding in the structure. The μ_T, is obtained from pushover plots presented in [14]. The spectral shape factor, *SSF*, then uses the μ_T and accounts for the difference between the frequency content of rare ground motions versus less rare ground motions. Finally, the adjusted collapse margin ratio, *ACMR*, can be calculated as follows in Eq. (5):

$$ACMR = SSF \times CMR \tag{5}$$

The *ACMR* is then compared to an acceptable collapse margin ratio, given the desired collapse probability target and the total system uncertainty. The total system uncertainty values are largely based on judgement and explained in FEMA P695. The total collapse uncertainty, β_{TOT}, is calculated using the square-root-of-the-sum-of-the-squares (SRSS) of the (1) the record-to-record uncertainty, β_{RTR}, (2) the design requirements-related collapse uncertainty, β_{DR}, (3) the test data-related uncertainty, β_{TD}, and (4) the modeling-related uncertainty, β_{MDL}. For β_{RTR}, a fixed value of 0.40 is assumed because FEMA P695 states that this is appropriate for the performance evaluation of systems with significant period elongation, which is expected to be true for special steel moment frames designed with a response modification coefficient, *R*, of 8. For β_{DR}, it is assumed there is high confidence in the basis of the design requirements and medium confidence in terms in completeness and robustness. Therefore, the result is a "good" rating, and a β_{DR} is set to 0.2. For

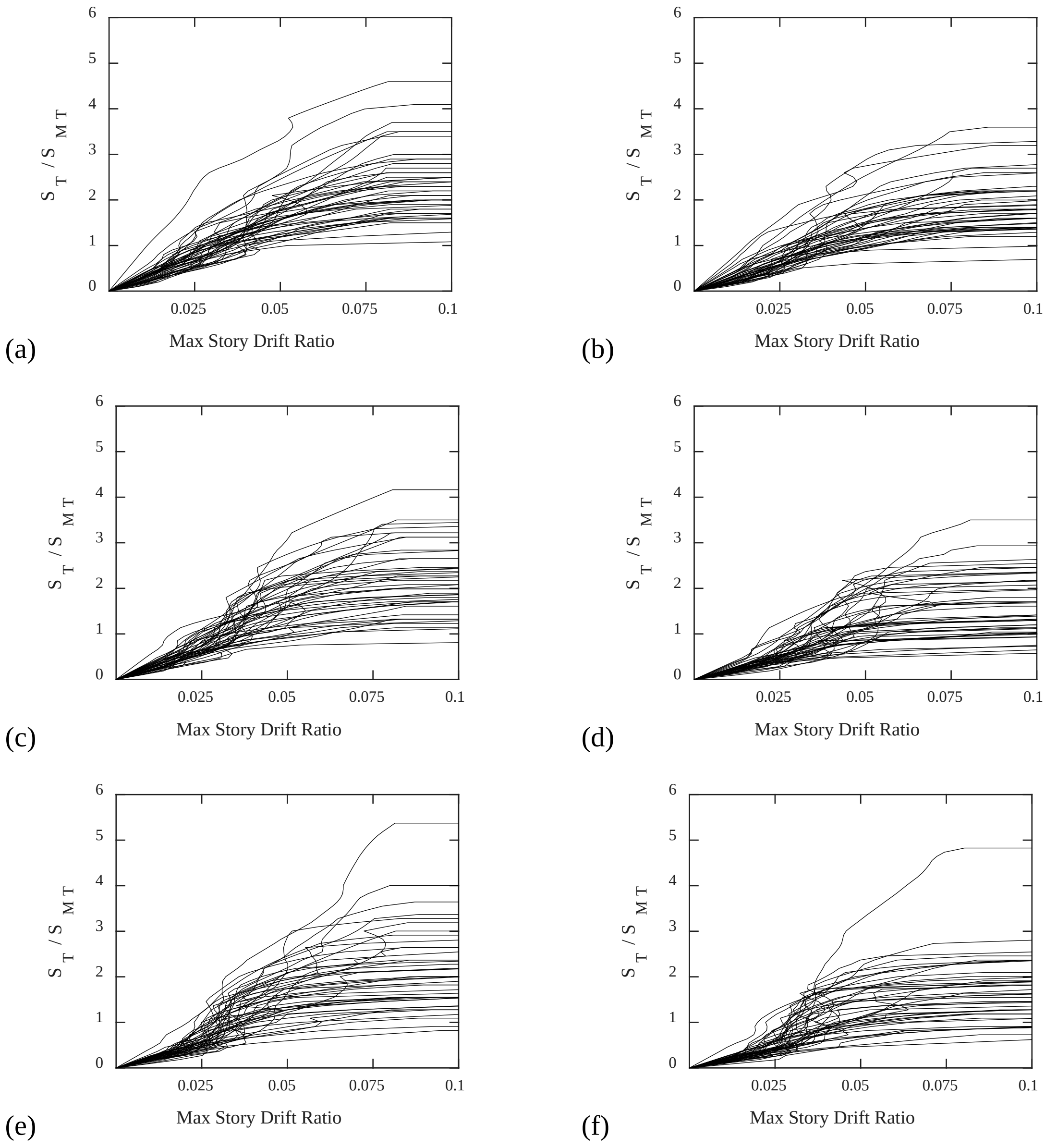

Fig. 4 – Incremental dynamic analysis curves for the (a) 4-story ELF, (b) 4-story RSA, (c) 8-story ELF, (d) 8-story RSA, (e) 16-story ELF, and (f) 16-story RSA-designed frames

β_{TD}, it is assumed there is high confidence the test results but a medium level of completeness and robustness of test results. This also gives a "good" rating, and a β_{TD} is set to 0.2. Lastly, for β_{MDL}, it is assumed that the accuracy and robustness of the models is high, but the representation of collapse characteristics is medium. This again gives a "good" rating, and a value of 0.2 is assigned. Now combining all the uncertainties using the SRSS, the total system collapse uncertainty is found to be 0.53. Using this total uncertainty number enables the determination of an acceptable value of *ACMR* from FEMA P695 Table 7-3, which is 1.96 for 10 % probability of collapse and 1.56 for 20 % probability of collapse. For a suite of new archetype building designs, the generally accepted collapse probability target is 10 % or less, given a MCE_R. It is recognized that for an individual building design, probability of collapse, given a MCE_R, may reach as high as 20 %, which is still considered acceptable. Therefore, since a limited suite of archetypes are investigated in this

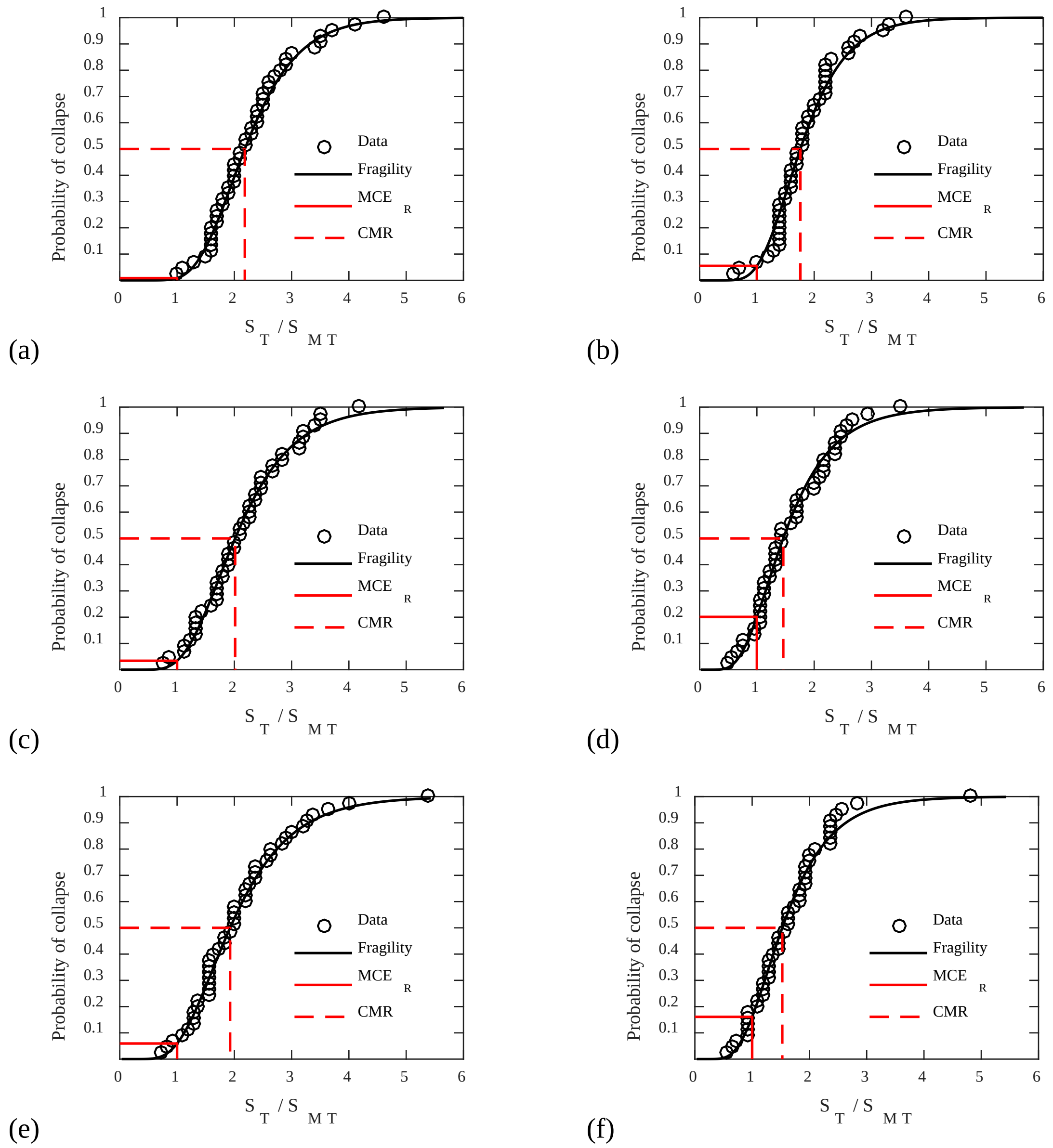

Fig. 5 – Fitted fragility curves for the (a) 4-story ELF, (b) 4-story RSA, (c) 8-story ELF, (d) 8-story RSA, (e) 16-story ELF, and (f) 16-story RSA-designed frames

study, the targets of $ACMR_{10\%}$ and $ACMR_{20\%}$ are both presented.

Table 1 gives a summary of the FEMA P695 assessment with all relevant values. Note that the FEMA P695 assessment considers the building performance as a whole, not as individual components as done in ASCE 41. Table 2 gives a summary of a nonlinear performance assessment using ASCE 41 reported in [8]. The nonlinear dynamic assessment procedure in ASCE 41 is the most comparable to a collapse assessment using the FEMA P695 methodology. Four out of the six buildings do not pass the ASCE 41 nonlinear dynamic assessment due to deficient beam-to-column connections. The columns and the panel zones also fail in some of the buildings. In contrast, all of the buildings pass the nonlinear FEMA P695 assessment at the more stringent 10 % probability of collapse threshold. This supports the conclusion made in Harris and Speicher [5], in which the ASCE 41 nonlinear assessment methodology is overly-conservative. As suggested in a follow-on set of journal papers by Speicher and Harris [9-11] and Maison and Speicher [21], ASCE 41 should investigate converting the acceptance criteria to a more robust measure that is not dependent on loading history. Acceptance criteria based on cumulative ductility demands (e.g., energy-based) could be especially sensible for well-performing systems that meet current building code provisions. Older, more archaic, systems may still need to be assessed with the current ASCE 41 approach, given that failure modes may not be as well-known or controlled. Regardless, further research should explore updating the acceptance criteria in performance-based design to ensure robust measurement science.

Table 1 – Summary of collapse performance evaluation for the suite special moment frames [14]

	Assessment parameters					Acceptance check (Pass/Fail = P/F)					
Building	Static Ω	CMR	μ_T	SSF	ACMR	$ACMR_{10\%}$	P/F ratio	P/F	$ACMR_{20\%}$	P/F ratio	P/F
04-ELF	2.78	2.18	5.4	1.48	3.22	1.96	1.64	**Pass**	1.56	2.06	**Pass**
04-RSA	2.22	1.76	4.5	1.43	2.51	1.96	1.28	**Pass**	1.56	1.61	**Pass**
08-ELF	2.61	2.01	4.1	1.40	2.83	1.96	1.44	**Pass**	1.56	1.81	**Pass**
08-RSA	1.85	1.46	3.1	1.35	1.97	1.96	1.01	**Pass**	1.56	1.27	**Pass**
16-ELF	1.92	1.93	3.6	1.38	2.65	1.96	1.35	**Pass**	1.56	1.70	**Pass**
16-RSA	1.68	1.53	3.7	1.38	2.11	1.96	1.08	**Pass**	1.56	1.35	**Pass**

Table 2 – Summary of predicted component performance by the nonlinear procedures for the collapse (CP) structural performance level (SPL) at the basic safety earthquake (BSE)-2 earthquake hazard level (EHL) for each archetype building (adapted from [8])

Building Height	Design Procedure	Nonlinear Static			Design Procedure	Nonlinear Dynamic (mean)		
		BC	CM	PZ		BC	CM	PZ
4-story	ELF	Pass	Pass	Pass	ELF	Pass	Pass	Pass
	RSA	**Fail**	Pass	Pass	RSA	**Fail**	Pass	Pass
8-story	ELF	Pass	**Fail**	Pass	ELF	**Fail**	**Fail**	Pass
	RSA	Pass	**Fail**	Pass	RSA	**Fail**	**Fail**	**Fail**
16-story	ELF	Pass	Pass	Pass	ELF	Pass	Pass	Pass
	RSA	Pass	Pass	Pass	RSA	**Fail**	**Fail**	Pass

Note: BC = beam-to-column connection, CM = column member, PZ = panel zone

5. Conclusions

A set of steel special moment frames designed with current building standards and then assessed using ASCE 41 were shown to be deficient in previous investigations. To determine whether the building designs are indeed deficient, this paper presents the results of a FEMA P695 investigation. The results demonstrate that the current code-complying designed frames have less than or equal to a 10 % probability of collapse, given a risk-targeted maximum considered earthquake, for all cases assessed. This suggests ASCE 41 is overly-conservative in its assessment criteria. It is recommended that further study be done to address the conservatism of ASCE 41 implied in these results. It is then argued that a potential remedy is to migrate ASCE 41 assessment criteria into a cumulative-based or energy-based measure. This may be especially relevant for current code-conforming buildings or when using ASCE 41 as a performance-based alternative for new building design.

6. 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. This work used the Extreme Science and Engineering Discovery Environment (XSEDE) through allocation MSS170023 supported by NSF grant number ACI-1548562.

The policy of the National Institute of Standards and Technology is to use the International System of Units in all its published materials. However, in this paper, some information is presented in U.S. customary units as this is the preferred system of units in the U.S. earthquake engineering industry. This paper is an official contribution of the U.S. National Institute of Standards and Technology; not subject to copyright in the United States.

7. References

[1] ASCE (2017): *Seismic Evaluation and Retrofit of Existing Buildings, ASCE/SEI 41-17*, American Society of Civil Engineers: Reston, VA.

[2] ASCE (2017): *Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-16*, American Society of Civil Engineers: Reston, VA.

[3] AISC (2016): *Specification for Structural Steel Buildings, ANSI/AISC 360-16*, American Institute of Steel Construction: Chicago, IL.

[4] AISC (2016): *Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-16*, American Institute of Steel Construction: Chicago, IL.

[5] Harris JL, Speicher MS (2015): *Assessment of First Generation Performance-Based Seismic Design Methods for New Steel Buildings Volume 1: Special Moment Frames*, NIST Technical Note 1863-1, National Institute of Standards and Technology: Gaithersburg, MD. https://doi.org/10.6028/NIST.TN.1863-1

[6] Harris JL, Speicher MS (2015): *Assessment of First Generation Performance-Based Seismic Design Methods for New Steel Buildings Volume 2: Special Concentrically Braced Frames*, NIST Technical Note 1863-2, National Institute of Standards and Technology: Gaithersburg, MD. https://doi.org/10.6028/NIST.TN.1863-2

[7] Harris JL, Speicher MS (2015): *Assessment of First Generation Performance-Based Seismic Design Methods for New Steel Buildings Volume 3: Eccentrically Braced Frames*, NIST Technical Note 1868-3, National Institute of Standards and Technology: Gaithersburg, MD. https://doi.org/10.6028/NIST.TN.1863-3

[8] Harris J, Speicher MS (2018): Assessment of Performance-Based Seismic Design Methods in ASCE 41 for New Steel Buildings: Special Moment Frames. *Earthquake Spectra*. 34(3): p. 977-999. https://doi.org/10.1193/050117EQS079EP.

[9] Speicher MS, Harris JL (2016): Collapse Prevention Seismic Performance Assessment of New Eccentrically Braced Frames using ASCE 41. *Engineering Structures*. 117: p. 344-357. https://doi.org/10.1016/j.engstruct.2016.02.018.

[10] Speicher MS, Harris JL (2016): Collapse prevention seismic performance assessment of new special concentrically braced frames using ASCE 41. *Engineering Structures*. 126: p. 652-666. https://doi.org/10.1016/j.engstruct.2016.07.064.

[11] Speicher MS, Harris JL (2018): Collapse Prevention seismic performance assessment of new buckling-restrained braced frames using ASCE 41. *Engineering Structures*. 164: p. 274-289. https://doi.org/10.1016/j.engstruct.2018.01.067.

[12] FEMA (2009): *Quantification of Building Seismic Performance Factors, FEMA P695*, Federal Emergency Managament Agency: Washington, D.C.

[13] McKenna F, Fenves GL (2016): *OpenSees command language manual. Version 2.5.0*, Pacific Earthquake Engineering Research Center: Berkeley, CA.

[14] Speicher MS, Dukes JD, Wong KKF (2020): *Collapse Risk of Steel Special Moment Frames per FEMA P695*, 2084, National Institute of Standards and Technology: Gaithersburg, MD. https://doi.org/10.6028/NIST.TN.2084

[15] CSI (2013): *Nonlinear Analysis and Performance Assessment for 3D Structures, PERFORM 3D*, Computers and Structures, Inc.: Berkeley, CA.

[16] Lignos DG, Krawinkler H (2011): Deterioration modeling of steel components in support of collapse prediction of steel moment frames under earthquake loading. *Journal of Structural Engineering*. 137(11): p. 1291-1302. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000376.

[17] NIST (2017): *Guidelines for Nonlinear Structural Analysis for Design of Buildings, Part IIa – Steel Moment Frames, prepared by the Applied Technology Council for the National Institute of Technology and Standards*, NIST GCR 17-917-46v2: Gaithersburg, MD. https://doi.org/10.6028/NIST.GCR.17-917-46v2

[18] Krawinkler H (1978): Shear in Beam-Column Joints in Seismic Design of Steel Frames. *Engineering Journal*. 15(3): p. 82-91.

[19] Baker JW (2015): Efficient Analytical Fragility Function Fitting Using Dynamic Structural Analysis. *Earthquake Spectra*. 31(1): p. 579-599. https://doi.org/10.1193/021113eqs025m.

[20] Towns J, Cockerill T, Dahan M, Foster I, Gaither K, Grimshaw A, Hazlewood V, Lathrop S, Lifka D, Peterson GD, Roskies R, Scott JR, Wilkins-Diehr N (2014): XSEDE: Accelerating Scientific Discovery. *Computing in Science & Engineering*. 16(5): p. 62-74. https://doi.org/10.1109/MCSE.2014.80.

[21] Maison BF, Speicher MS (2016): Loading Protocols for ASCE 41 Backbone Curves. *Earthquake Spectra*. 32(4): p. 1-20. https://doi.org/10.1193/010816EQS007EP.

