Proceedings of the Cold-Formed Steel Research Consortium Colloquium
20-22 October 2020 (cfsrc.org)

Application of ASCE 41 to a two-story CFS building

M.S. Speicher[1], Z. Zhang[2], B.W. Schafer[3]

Abstract

The objective of this paper is to summarize the evaluation results from applying the updated performance-based seismic design provisions, ASCE 41-17, on a cold-formed steel framed building sited in a location with high seismic demands. The assessment included examination of the existing design and consideration of the retrofits required to bring the design into compliance with ASCE 41-17. The assessment of the building relies on the linear procedures and m-factors in ASCE 41-17 and follows the same basic process as the original design. Despite the fact that the studied building is compliant with ASCE 7 and AISI S400, and successfully withstood shake table testing in excess of maximum considered earthquake levels with no permanent damage and no residual drift, ASCE 41-17 finds the building to be deficient. The work highlights that, for cold-formed steel framing, even though ASCE 41 is based on the same tested shear walls that ASCE 7/AISI S400 rely upon, the component-based procedures of ASCE 41 do not easily account for the larger system overstrength and ductility that are included and validated for actual systems. Further work is needed to improve ASCE 41 to account for full system performance, this is particularly important given ASCE 41's growing role as the benchmark performance-based standard for seismic assessment and design.

1. Introduction

ASCE 41-17 [1] provides seismic design procedures for assessment, repair/retrofit, and new building design. Recent studies have highlighted critical differences between ASCE 41-based seismic design and conventional seismic design using ASCE 7 and materials standards (e.g, AISC 341) [2-11]. In the last code update cycle the ASCE 41 provisions were significantly expanded to reflect the best available data for cold-formed steel response [13]. The impact of these changes, and comparisons between ASCE 7-based design and ASCE 41-based design, do not currently exist for cold-formed steel (CFS) framed buildings. To complete such a comparison the two-story CFS framed building designed and tested during a National Science Foundation (NSF) sponsored George E. Brown Network for Earthquake Engineering Simulation (NEES) research project know as CFS-NEES (Figure 1) is selected as a case study.

The CFS-NEES building was designed to contemporary practice using ASCE 7/AISI S400 and subjected to shake table testing at the University at Buffalo in 2013. The building response was excellent, with only minor damage even for seismic excitations in excess of ASCE 7's maximum considered earthquake levels [14]. Subsequent nonlinear time history analyses further demonstrated that while the building was efficiently designed with respect to ASCE7/AISI S400 (i.e. demand/capacity ratios for the shear walls near 1.0) the building had substantial strength reserve and more than acceptable collapse probabilities [15,16].

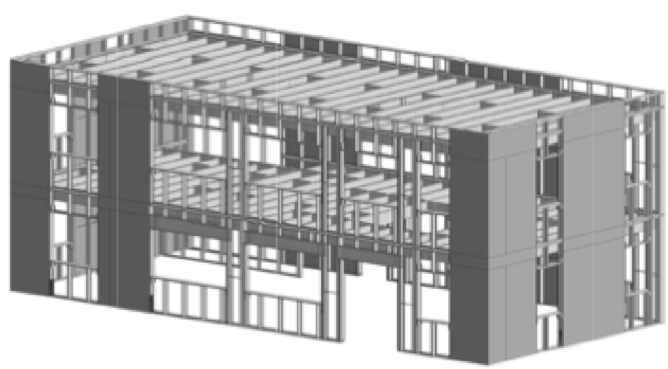

Figure 1. Isometric of framing for 2-story CFS-NEES building (sheathing depicted only on shear walls) [17].

[1] Research Structural Engineer, National Institute of Standards and Technology: matthew.speicher@nist.gov
[2] Graduate Research Assistant, Dept. of Civil and Systems Engineering, Johns Hopkins University: zhidongzhang@jhu.edu
[3] Professor, Dept. of Civil and Systems Engineering, Johns Hopkins University: schafer@jhu.edu

2. Methodology

The original CFS-NEES building design was completed per ASCE 7-05, AISI S100-07, and AISI S213-07 as detailed in [17]. The design was updated to satisfy the latest standards, ASCE 7-16 [18], AISI S100-16 [19], and AISI S400-15 [20]. Then, the updated design was evaluated as an *existing building* using the linear static procedure of ASCE 41-17. Per ASCE 41, the existing building was evaluated for life safety (LS) at the basic safety earthquake (BSE)-1E level and collapse prevention (CP) at the BSE-2E level, where the letter "E" signifies "existing." Only the LS results are provided here, see the complete report in [21] for CP results and full details of the methods, results, and discussion. After the existing building evaluation was completed, a *retrofit* that satisfies the ASCE 41 linear requirements was completed.

2.1 General Approach for ASCE 41 Assessment

ASCE 41 has several different assessment options, from a tier 1 evaluation, which includes a "checklist" cursory style screening, to a tier 3 evaluation, which consists of varying degrees of engineering analysis, with the most complex being the nonlinear dynamic procedure. For this study, the linear static procedure is used, which is the "simplest" form of a tier 3 analysis. The linear static procedure aligns well with the equivalent static force procedure used in traditional design and involves applying an unreduced lateral load, distributed at each story, and then comparing the force demand to the product of the expected capacity and a component capacity modification (*m*)-factor that accounts for the ductility at the selected structural performance level.

2.2 Demand

The first step taken for the ASCE 41 linear assessment is to calculate the demands on the shear walls. The shear walls are considered deformation-controlled components. The base shear of the building that the shear walls must carry, V, is calculated from ASCE 41-17 Equation 7-21:

$$V = C_1 C_2 S_a W \quad (1)$$

where C_1 is a modification factor relating expected maximum inelastic displacements to displacements obtained from linear elastic response; C_2 is the modification factor representing the effects of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum response; S_a is the response spectrum acceleration at the fundamental period of the building; and W is the effective seismic weight of the building. For the assessment in this study, the approximate value for the product of C_1C_2 is employed from ASCE 41-17 Table 7-3 and is equal to 1.4.

The base shear is distributed to each floor as a static force and the story shears are then distributed to the two sides of the building (1/2 to each side) and then to each shear wall along a side of the building based on their calculated relative stiffness. These individual shear wall demands are used as the deformation-controlled component demands per unit length, v_{ud}, for the assessment.

The demands on the chord studs and ties/hold-downs are determined by treating them as force-controlled components. The "capacity design" approach in ASCE 41 is employed in which the expected capacity of the shear wall is used to determine the maximum forces that can be delivered to the force-controlled components. The required axial load, P_r, and the required moment, M_r, are generated assuming the shear wall is carrying its expected capacity in combination with the appropriate gravity load. The chord studs are subjected to eccentric loads, primarily due to gravity loads framing into the interior flange of the stud from the ledger.

For linear procedures, the combination of actions resulting from dead load (Q_D) and live load (Q_L) with the seismic load (Q_E) follows per ASCE 41-17 Eq. (7-1), adapted here as follows:

$$Q = Q_E + 1.1(Q_D + Q_L) \quad (2)$$

where Q_D is the action resulting from the dead load and Q_L is the action resulting from the live load. Further, Q_L is defined as 25 % of the unreduced live load from ASCE 7. The maximum axial forces in the ties and hold-downs are determined similarly – considering the expected capacity of the shear wall, and considering the case of counteracting loads where ASCE 41-17 Eq. 7-2 holds, adapted here as:

$$Q = Q_E + 0.9(Q_D) \quad (3)$$

2.3 Capacity

The shear wall expected capacity per unit length, v_{ce}, is:

$$v_{ce} = \phi v_n \quad (4)$$

where ϕ is set to 1.0 and v_n is the nominal shear wall capacity per unit length. The nominal shear wall capacity is determined from AISI S400-15. Additionally, the *m*-factors in ASCE 41 can be considered part of the capacity of the shear wall. The *m*-factors are found in ASCE 41-17 Table 9-9 for CFS components. CFS shear walls sheathed with oriented strand board (OSB), considered as primary components, have *m*-factors of 2.5 for life safety (LS) and 3.3 for collapse prevention (CP).

The chord studs are considered force-controlled components, therefore lower-bound strengths are used in the assessment. The lower-bound axial (P_{CL}) and flexural strength (M_{CL}) for the chord studs as specified in ASCE 41-17 Section 9.3.2.3.2 and result in P_{CL}=0.94P_n and M_{CL}=0.94M_n as detailed in [21].

2.4 Acceptance Criteria Check

The linear acceptance criteria check for the shear walls follows the requirements for deformation-controlled components in ASCE 41. With the demand and capacity determined, the linear procedure acceptance criteria for the shear walls is:

$$\frac{v_{ud}}{\kappa v_{ce}} < m \tag{5}$$

where κ is assumed as 1.0 herein. The acceptance criteria check for the chord studs and ties/hold downs follow the requirements for force-controlled components in ASCE 41. The acceptance criteria for the chord studs can be written as the following interaction equation:

$$\kappa\left(\frac{P_{UF}}{P_{CL}} + \frac{M_{UF}}{M_{CL}}\right) \leq 1.0 \tag{6}$$

where P_{CL} is the lower-bound capacity of the chord stud in compression, M_{CL} is the lower-bound capacity of the chord stud in flexure, P_{UF} is the maximum axial load that can be developed in the chord stud due to the shear wall reaching its expected capacity (in combination with dead and live load), and M_{UF} is the flexural load resulting from eccentricity in the loads being delivered to the chord stud. Note, M_{UF} includes second order effects and may be approximated as B_1M_{UF1} where B_1 is the approximate moment magnifier (Equation C1.2.1.1-3 in AISI S100-16 [19]) and M_{UF1} is the first-order demand. The acceptance criteria check for ties/hold-downs is:

$$\kappa\left(\frac{T_{UF}}{T_{CL}}\right) \leq 1.0 \tag{7}$$

where T_{CL} is the lower-bound tension or compression capacity and T_{UF} is the demand arising from the shear wall reaching its expected capacity.

3. Existing Building Evaluation per ASCE 41-17

Per ASCE 41-17 the CFS-NEES building's linear static procedure assessment results for the life safety (LS) performance level at the BSE-1E earthquake hazard level are shown in Table 1.

Shear walls with $v_{ud}/v_{ce} > m$ fail the assessment and are designated with bold and underline. For the 2nd story, 6 out of 10 shear walls fail the assessment. For the first story, 9 out of 10 shear walls fail the assessment.

Table 1. Linear static assessment results of the shear walls considering life safety (LS) at the BSE-1E earthquake hazard level, where plf is pounds per linear foot.

2nd Story				m-factor
Shear wall[a]	v_{ud} plf (kN/m)	v_{ce} plf (kN/m)	v_{ud} / v_{ce}	
L2S1	2039(29.76)	622(9.08)	**<u>3.28</u>**	2.5
L2S2	2623(38.28)	700(10.2)	**<u>3.75</u>**	2.5
L2S3	2039(29.76)	622(9.08)	**<u>3.28</u>**	2.5
L2N1	1684(24.58)	700(10.2)	2.41	2.5
L2N2	1152(16.81)	700(10.2)	1.65	2.5
L2W1	1408(20.55)	622(9.08)	2.26	2.5
L2W2	1408(20.55)	622(9.08)	2.26	2.5
L2W3	2595(37.87)	700(10.2)	**<u>3.71</u>**	2.5
L2E1	1755(25.61)	700(10.2)	**<u>2.51</u>**	2.5
L2E2	2362(34.47)	700(10.2)	**<u>3.37</u>**	2.5
1st Story				m-factor
Shear wall[a]	v_{ud} plf (kN/m)	v_{ce} plf (kN/m)	v_{ud} / v_{ce}	
L1S1	3465(50.57)	733(10.7)	**<u>4.73</u>**	2.5
L1S2	4434(64.71)	825(12.0)	**<u>5.37</u>**	2.5
L1S3	3465(50.57)	733(10.7)	**<u>4.73</u>**	2.5
L1N1	2860(41.74)	825(12.0)	**<u>3.47</u>**	2.5
L1N2	1946(28.40)	825(12.0)	2.36	2.5
L1W1	2401(35.04)	733(10.7)	**<u>3.27</u>**	2.5
L1W2	2401(35.04)	733(10.7)	**<u>3.27</u>**	2.5
L1W3	4383(63.97)	825(12.0)	**<u>5.31</u>**	2.5
L1E1	2979(43.48)	825(12.0)	**<u>3.61</u>**	2.5
L1E2	4002(58.40)	825(12.0)	**<u>4.85</u>**	2.5

Note: **<u>bold and underline</u>** component fails assessment. **a**. shear walls are identified by level one (L1) or two (L2) by face of the building north (N), south (S), east (E), and west (W) - the south (long) and east (short) walls are shown in Figure 1, and finally shear wall number 1, 2, or 3.

4. Retrofit Design Evaluation per ASCE 41-17

Each shear wall was individually retrofitted to pass the ASCE 41 assessment. The easiest retrofit option was to increase the number of fasteners. The original fastener spacing was 6 in. (150 mm), therefore for practical purposes a 3 in. (75 mm) fastener spacing was first investigated. If a 3 in. (75 mm) spacing did not give the necessary capacity, double sheathing (i.e. sheathing on both sides of the wall) was the next option examined. If with double-sided sheathing the capacity was sufficient to relax back from 3 in. fastener spacing to 6 in. (150 mm) fastener spacing, then this was done. After iterating through the different options, each shear wall was retrofitted and Table 2 summarizes the results for life safety (LS) at the BSE-1E level.

Table 2. CFS-NEES retrofit for shear walls for life safety (LS) at the BSE-1E earthquake hazard level, where *s* is the fastener spacing.

	Retrofit				
SW	OSB Sheathing in (mm)	sides	*s* in (mm)	v_{ud} / v_{ce}	m-factor
2nd Story					
L2S1	7/16(11)	**2**	6(150)	1.64	2.5
L2S2	7/16(11)	**2**	6(150)	1.86	2.5
L2S3	7/16(11)	**2**	6(150)	1.64	2.5
L2N1	7/16(11)	1	6(150)	2.41	2.5
L2N2	7/16(11)	1	6(150)	1.65	2.5
L2W1	7/16(11)	1	6(150)	1.85	2.5
L2W2	7/16(11)	1	6(150)	1.85	2.5
L2W3	7/16(11)	**2**	6(150)	2.06	2.5
L2E1	7/16(11)	**2**	6(150)	1.26	2.5
L2E2	7/16(11)	**2**	6(150)	1.69	2.5
1st Story					
L1S1	7/16(11)	**2**	**3(75)**	1.26	2.5
L1S2	7/16(11)	**2**	**3(75)**	1.43	2.5
L1S3	7/16(11)	**2**	**3(75)**	1.26	2.5
L1N1	7/16(11)	1	**3(75)**	1.85	2.5
L1N2	7/16(11)	1	**3(75)**	1.26	2.5
L1W1	7/16(11)	1	**3(75)**	1.73	2.5
L1W2	7/16(11)	1	**3(75)**	1.73	2.5
L1W3	7/16(11)	**2**	**3(75)**	1.43	2.5
L1E1	7/16(11)	**2**	**3(75)**	0.96	2.5
L1E2	7/16(11)	**2**	**3(75)**	1.30	2.5

Note: **bold** indicates changes from original design (7/16” (11 mm) OSB on 1 side with fasteners spaced at 6 in. (152 mm) o.c.).

Required changes for the 1st story shear wall retrofit are significant – the South and East wall lines require double-sided sheathing as does the longest shear wall on the West facing wall line, L1W3. All 1st story shear walls need additional fasteners placed between all existing fasteners to decrease the fastener spacing down to 3 in. (75 mm) The 2nd story shear walls require double-sided sheathing in the same locations as the 1st story, but the existing 6 in. (150 mm) fastener spacing is adequate. The required retrofits would be costly; however, they do not require an increase in shear wall length, thus practically they could be accomplished.

Given the increased capacity of the shear walls due to decreasing the fastener spacing and/or adding sheathing, the chords studs need to be evaluated to determine if they have sufficient capacity to carry the forces created when the shear walls are loaded to their new expected capacity. At the life safety BSE-1E hazard level the existing 2nd story chord studs are adequate for the retrofit, but none of the existing 1st story chord studs are adequate. Retrofit options are possible, but none are without complication. Retrofit designs consisting of adding one or two additional studs to the chord studs are provided for the life safety BSE-1E hazard level in Table 3. The results of the interaction equation are also provided in the tables. In the reported retrofit designs an interaction expression as high as 1.05 was allowed. At the life safety BSE-1E hazard level adding one additional stud (for a total of 3) is found to be sufficient.

Table 3. Linear static procedure assessment results of the chord studs considering expected capacities from shear walls retrofitted to meet life safety (LS) at the BSE-1E earthquake hazard level.

1st Story			
SW	Existing Chord Stud	Retrofit Chord Stud	Int'n
L1S1	(2) 600S162-54	**(3) 600S162-54**	1.00
L1S2	(2) 600S162-54	**(3) 600S162-54**	1.03
L1S3	(2) 600S162-54	**(3) 600S162-54**	0.99
L1N1	(2) 600S162-54	**(3) 600S162-54**	0.96
L1N2	(2) 600S162-54	**(3) 600S162-54**	0.94
L1W1	(2) 600S162-54	**(3) 600S162-54**	0.82
L1W2	(2) 600S162-54	**(3) 600S162-54**	0.82
L1W3	(2) 600S162-54	**(3) 600S162-54**	1.01
L1E1	(2) 600S162-54	**(3) 600S162-54**	1.01
L1E2	(2) 600S162-54	**(3) 600S162-54**	1.01

Note: **bold** indicates changes from original design. Interaction allowed up to 1.05 by engineering judgment

The retrofit design requires increased capacity of the shear walls and this also potentially influences the existing story-to-story ties and the hold-down anchorage. The demands, T_{UF}, consider the load combination for counteracting loads. At the life safety BSE-1E hazard level the existing 1st-to-2nd story ties are adequate, but none of the foundation-to-1st story hold-downs are adequate.

Retrofit of the foundation-to-1st story hold-down can be completed relatively simply if a second hold-down (added to the opposite face of the stud) is adequate for the demand. It is possible to place hold-downs side by side as well, thus having as many as 4 commercial hold-downs connected to a built-up chord stud. Non-commercial options using heavy angles are also possible for higher demands. As higher capacity hold-downs are employed, one must note that the anchor bolt sizes typically increase, requiring additional redesign for the retrofit. Capacity of the underlying foundation, particularly with multiple anchors in close proximity, may further limit the available tensile capacity and require additional, more costly and more complex, retrofit. Where possible it is recommended to simply double up the existing S/HDU 6 hold-downs. However, this is not adequate for all the hold-downs in the South walls and East walls and in the L1W3 West wall. For these cases 2 x S/HDU9 hold-downs are specified. These hold-downs have 64 % more strength than the S/HDU6 when connected to 54 mil studs, but require a 7/8 in. (22 mm) anchor bolt.

6. Discussion

For the studied CFS-framed building, ASCE 41-17 provides a more pessimistic estimation of the seismic response than ASCE 7-16. ASCE 41's m-factors are based on direct shear wall tests (as described in [13]) and are ostensibly a more direct and rational gauge of expected behavior than the R and Ω_o factors of ASCE 7, which are based more on experience and judgment than on direct testing [23]. However, in the case of the studied CFS-NEES building, direct testing of the entire building system was conducted and indicated behavior far better than ASCE 7's prediction – even at excitations in excess of the ASCE 7 maximum considered earthquake (MCE)-level, minimal damage occurred [14]. Thus, the true behavior is better than ASCE 7-16's prediction, and far better than ASCE 41-17's prediction.

Subsequent analysis indicated that repetitively framed buildings, such as the CFS-NEES building, have significant overstrength, even more than the amount attributed at Ω_o levels [15]. Examination of the ASCE 7 seismic response modification factors using the FEMA P695 [23] procedure for the CFS-NEES building indicated that if only the shear walls were considered (as essentially is done in ASCE 41 if gravity and non-structural wall contributions to lateral capacity are ignored), then the collapse probabilities are unacceptable. In contrast, if the shear walls and all the gravity framing (unsheathed) were considered, then the collapse probabilities were acceptable – suggesting ASCE 7 response modification factors (R and Ω_o) are justified. Moreover, if the final building, with sheathing, non-structural walls, and finish systems, was considered, then the collapse probabilities were acceptable by an even wider margin and the structural analysis was in line with the shake table test results [16]. Essentially, for this building, and likely this building system type, ASCE 41's lack of an "easy switch" to account for system overstrength in the linear assessment procedure is an important reason that it's linear analysis method provides such pessimistic predictions of performance.

The use of nonlinear static or nonlinear dynamic procedures could provide further insight on the predicted behavior of the building. However, the use of nonlinear procedures is not expected to change the fundamental findings herein: ASCE 41 predicts higher demands than ASCE 7, especially for short period buildings, and does not readily provide a means to easily include system overstrength, thus resulting in conservative assessment outcomes. One proviso on this conclusion, if the gravity and non-structural wall elements are modeled as being meaningfully capable of resisting lateral demands and a rational approach can be adopted for their strength and stiffness degradation, then it is possible, within the ASCE 41 framework, to include the system overstrength. However, where ASCE 7 allows the engineer to include this overstrength effect through a single Ω_o factor, ASCE 41 would require explicit modeling, with significant uncertainty in the parameters, to include the same phenomena.

7. Conclusions

A two-story cold-formed steel framed building, previously designed to ASCE 7 and successfully tested on shake tables in the laboratory, was examined to determine necessary changes if ASCE 41 is adopted for assessment. The two-story cold-formed steel framed building, designed to satisfy ASCE 7, fails when assessed as an existing building per ASCE 41. Retrofit of the two-story cold-formed steel framed building such that it meets the criteria of ASCE 41 essentially requires doubling the capacity of the seismic force resisting system beyond that of ASCE 7. This doubling in capacity is not justified by the experimentally and numerically validated performance of the building. Two primary factors contribute to the conservative nature of ASCE 41's predictions: (1) the basic seismic demands are significantly greater in ASCE 41 than in ASCE 7, especially for short period structures, and (2) large system overstrength, common in repetitively-framed structures, is accounted for in ASCE 7, but not easily in the linear procedures of ASCE 41. Though overstrength may be addressed in ASCE 41 by the higher tier analysis methods (i.e., nonlinear methods), for normal low-rise CFS buildings, this level of effort may not be a realistic option. For ASCE 41 to realize its performance-based design vision and for society to benefit from the flexibility afforded by such frameworks, the basic predicted seismic response for cold-formed steel framed buildings needs to be more closely aligned with reality as demonstrated by shake table tests. Thus, improvements in both demand and capacity procedures for ASCE 41 are needed for this class of building.

8. Acknowledgments and Disclaimers

The contributions of Ivana Olivares are gratefully acknowledged. Certain commercial entities, equipment, or materials may be identified in this document in order to describe an experimental procedure or concept adequately. Such identification is not intended to imply recommendation

or endorsement by the National Institute of Standards and Technology, nor is it intended to imply that the entities, materials, or equipment are necessarily the best available for the purpose.

References

[1] ASCE (2017) Seismic Evaluation and Retrofit of Existing Buildings (American Society of Civil Engineers).

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

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

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

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

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

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

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

[13] Ayhan D, Rob L. Madsen, Ben W. Schafer (2016) Progress in the Development of ASCE 41 for Cold-Formed Steel. Proceedings of the 23rd International Specialty Conference on Cold-Formed Steel Structures (Baltimore, MD), pp 417–432.

[14] Peterman Kara D., Stehman Matthew J. J., Madsen Rob L., Buonopane Stephen G., Nakata Narutoshi, Schafer Benjamin W. (2016) Experimental Seismic Response of a Full-Scale Cold-Formed Steel-Framed Building. I: System-Level Response. Journal of Structural Engineering 142(12):04016127. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001577

[15] Leng J, Peterman KD, Bian G, Buonopane SG, Schafer BW (2017) Modeling seismic response of a full-scale cold-formed steel-framed building. Engineering Structures 153:146–165. https://doi.org/10.1016/j.engstruct.2017.10.008

[16] Leng J, Buonopane SG, Schafer BW (2020) Incremental dynamic analysis and FEMA P695 seismic performance evaluation of a cold-formed steel–framed building with gravity framing and architectural sheathing. Earthquake Engineering & Structural Dynamics 49(4):394–412. https://doi.org/10.1002/eqe.3245

[17] Rob L. Madsen, N. Nakata, Ben W. Schafer (2013) CFS-NEES Building Structural Design Narrative. Available at http://jhir.library.jhu.edu/handle/1774.2/40584

[18] ASCE (2017) Minimum Design Loads for Buildings and Other Structures (American Society of Civil Engineers).

[19] AISI S100 (2016). Specification for the Design of Cold-Formed Steel Structural Members. American Iron and Steel Institute. Washington, DC.

[20] AISI S400 (2015). North American Standard for Seismic Design of Cold-Formed Steel Structural Systems. American Iron and Steel Institute. Washington, DC.

[21] Speicher, M., Olivares, I., Schafer, B.W. (2020). Seismic Evaluation of a 2-Story Cold-Formed Steel Framed Building using ASCE 41-17. NIST Technical Note 2116, Gaithersburg, MD https://doi.org/10.6028/NIST.TN.2116

[22] FEMA (2009) Quantification of Building Seismic Performance Factors (Department of Homeland Security).