

A Case for Rethinking ASCE 41 Performance-Based Assessment Criteria for Cold-Formed Steel

Matthew S. Speicher¹, Zhidong Zhang², and Benjamin W. Schafer²

¹ National Institute of Standards and Technology, Gaithersburg MD 20899, USA

² Johns Hopkins University, Baltimore MD 21218, USA
matthew.speicher@nist.gov

Abstract. The objective of this paper is to evaluate the impact of applying the ASCE 41-17 performance-based design provisions on a 2-story building previously designed and tested during the CFS-NEES research project. The latest version of the American Society of Civil Engineers Standard 41 (ASCE 41-17) incorporates new provisions for the assessment of cold-formed steel (CFS) systems. These provisions include component acceptance criteria derived from a database of available experimental data. Though the new set of criteria was rigorously developed, how these criteria would score a newly-designed CFS building is not well understood. Given that the vast majority of CFS buildings are designed using linear procedures, particular interest is given to comparing the linear procedures for new design (i.e., using ASCE 7-16) versus the linear procedures for existing building assessment (i.e., using ASCE 41). Findings show that although the 2-story building withstood shake table tests with intensities greater than the maximum considered earthquake, the ASCE 41 assessment indicates the building's shear walls are deficient. This highlights a disconnect between ASCE 41 and new building design per ASCE 7. To help resolve this disconnect, practical ways to account for system overstrength should be considered in future editions of ASCE 41.

Keywords: cold-formed steel, performance-based seismic design, building codes, building standards.

1 Introduction

The American Society of Civil Engineers (ASCE) 41-17 [1] is the third edition of ASCE 41, which contains a standardized performance-based seismic engineering methodology used in assessing, retrofitting, and repairing existing buildings and in designing new buildings. Major updates were made in ASCE 41-17 including a significant expansion of the cold-formed steel (CFS) provisions to reflect the best-available performance data, as outlined in Ayhan et al [2]. Of particular note, the acceptance criteria of various CFS lateral force-resisting systems were updated to match test data using a process of backbone curve development outlined in Chapter 7 of ASCE 41-17. This process is rational and, with respect to component-level performance, produces acceptance criteria values that reflect the state-of-knowledge. How-

ever, repetitively framed systems such as those constructed of CFS are known to have substantial overstrength due to the presence of sheathed gravity framing and non-structural framing (e.g., partition walls), both of which are not easily captured in component-level assessments. Therefore, without an easy way to capture these effects, it is hypothesized that ASCE 41 will give a pessimistic performance prediction, even when shake table testing demonstrates satisfactory building performance

Several studies have investigated the relationship between ASCE 41-based seismic design and conventional seismic design. It has been found that a building designed with the conventional approach (e.g., loads from ASCE 7 [3] and capacities from American Institute of Steel Construction (AISC) 341 [4] and AISC 360 [5]) often does not pass an ASCE 41 assessment, suggesting either ASCE 41 is overly conservative, or the conventional design is deficient [6-11]. To shed light on this issue for CFS systems, this paper conducts an ASCE 41 assessment of a two-story CFS-framed building designed and tested during a National Science Foundation (NSF) sponsored research project known as CFS-NEES. This building was selected because it is representative of typical CFS construction and is a case with available full-scale experimental results. Additional details of this case study can be found in [12-14].

2 Methodology

First, minor design updates were done to the CFS-NEES building to satisfy the following standards: ASCE 7-16 [15], AISI S100-16 [16], and AISI S400-15 [17]. The updated design was evaluated as an *existing building* using the linear static procedure (LSP) of ASCE 41-17. The evaluation was done for Collapse Prevention (CP) at the Basic Safety Earthquake 2E (BSE-2E) level. A *new building design* was then created to satisfy the ASCE 41-17 assessment criteria for CP at the BSE-2N level.

The LSP is the “simplest” form of an ASCE 41 Tier 3 analysis and aligns with ASCE 7’s equivalent lateral force procedure. In the LSP, an unreduced lateral load is distributed to each story, and then the force demand is compared to the product of the expected capacity and a component capacity modification (m)-factor. The m -factor accounts for the ductility at the selected structural performance level.

The shear walls are considered deformation-controlled components. The base shear, V , of the building 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 calculations in this study, $C_1 C_2$ is set equal to 1.4.

The demands on the chord studs and ties/hold-downs are determined by considering them as 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 and live load with the seismic load (Q_E) follows ASCE 41-17 Eq. (7-1), adapted here as:

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

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

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

The shear wall expected capacity per unit length, v_{ce} , is set equal to v_n , the nominal shear wall capacity per unit length, determined from AISI S400-15. Additionally, the m -factors can be considered part of the capacity of the shear wall from Table 9-9 of ASCE 41-17. CFS shear walls sheathed with oriented strand board (OSB), considered as primary components, have m -factors of 2.5 for LS and 3.3 for CP.

The chord studs are considered force-controlled, therefore lower-bound strengths are used. 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 [12].

The linear acceptance criteria check for shear walls follows the requirements for deformation-controlled components in ASCE 41-17. With the demand and capacity determined, the linear procedure acceptance criteria for the shear walls is:

$$(v_{ud}/v_{ce})/\kappa < m \quad (4)$$

where κ is the knowledge factor taken as 1.0. 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:

$$(P_{UF}/P_{CL} + M_{UF}/M_{CL})/\kappa \leq 1.0 \quad (5)$$

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) and M_{UF1} is the first-order demand. The acceptance criteria check for ties/hold-downs is:

$$(T_{UF}/T_{CL})/\kappa \leq 1.0 \quad (6)$$

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 Building Evaluation and Discussion

The CFS-NEES building's LSP assessment results for the CP at the BSE-2E are shown in Table 1. Shear walls with $v_{ud}/v_{ce} > m$ fail the assessment and are designated with bold and underline. All the shear walls fail the assessment except L2N2.

Following the same procedures as done in the evaluation, a new lateral design was created to satisfy current ASCE 41 requirements. The gravity design of [14] was assumed to be adequate. For the new building design only the shear walls, chord studs, story-to-story chord stud ties, and foundation-to-1st story hold-down anchorages were redesigned. For simplicity, and to aid comparison across the designs, it was decided to keep the shear wall lengths and locations consistent with the original building, and to use the same lateral system and sheathing (11 mm OSB) as in the original design. The design freedom is somewhat limited by the available systems in AISI S400-15.

The results of the assessment are provided for CP at the BSE-2N in Table 2. The 2nd story shear walls required 2-sided, 11 mm OSB, #8 min fasteners @ 152 mm o.c. (on-center) with 1.37 mm (54 mil) studs. The 1st story shear walls required 2-sided, 11 mm OSB, #10 min fasteners @ 152 mm o.c., with 1.73 mm (68 mil) studs.

The efficiency of the design may be judged, in part, by the utilization ratios provided in Table 2. For construction efficiency it was decided to keep the shear wall configurations uniform, which also influences the structural efficiency. The shear wall length and aspect ratio were largely determined by the architectural openings, which results in some shear walls being more highly utilized than others.

For this case study, ASCE 41 indicates that the ASCE 7-designed building needs retrofitted. ASCE 41's m -factors are based on a database of cyclically-tested shear walls [2] and are a direct measure of expected behavior. In contrast, ASCE 7's design is based on system performance factors, R , C_d , and Ω_0 , which are based more on experience and judgment than on direct testing [18]. In the case of the studied CFS-NEES building, testing of the *entire* building system was conducted and indicated behavior far better than ASCE 41's prediction. Even at excitations greater than the ASCE 7 maximum considered earthquake, minimal damage occurred [19]. Thus, the true behavior was also significantly better than the minimum requirement in ASCE 7.

Research has shown that repetitively framed buildings, such as the CFS-NEES building, have significant overstrength that may even exceed the amount attributed at Ω_0 levels [20]. Examination of the ASCE 7 seismic response modification factors using the FEMA P695 [21] procedure for the CFS-NEES building indicated that if only the shear walls were considered (as is essentially done in ASCE 41 if one ignores gravity and non-structural wall contributions to lateral capacity), then the collapse probabilities are unacceptable. In contrast, if the shear walls and all the unsheathed gravity framing were considered, then the collapse probabilities were acceptable –

Table 1. Linear static procedure assessment results of the shear walls (SW).

2 nd story				1 st story				m-factor
SW	v_{ud} (kN/m)	v_{ce} (kN/m)	v_{ud}/v_{ce}	SW	v_{ud} (kN/m)	v_{ce} (kN/m)	v_{ud}/v_{ce}	
L2S1	45.3	9.1	<u>4.99</u>	L1S1	77.0	10.7	<u>7.19</u>	3.3
L2S2	58.2	10.2	<u>5.70</u>	L1S2	98.5	12.0	<u>8.18</u>	3.3
L2S3	45.3	9.1	<u>4.99</u>	L1S3	77.0	10.7	<u>7.19</u>	3.3
L2N1	37.4	10.2	<u>3.66</u>	L1N1	63.5	12.0	<u>5.28</u>	3.3
L2N2	25.6	10.2	2.50	L1N2	43.2	12.0	<u>3.59</u>	3.3
L2W1	31.3	9.1	<u>3.44</u>	L1W1	53.3	10.7	<u>4.98</u>	3.3
L2W2	31.3	9.1	<u>3.44</u>	L1W2	53.3	10.7	<u>4.98</u>	3.3
L2W3	57.6	10.2	<u>5.64</u>	L1W3	97.3	12.0	<u>8.09</u>	3.3
L2E1	39.0	10.2	<u>3.82</u>	L1E1	66.2	12.0	<u>5.50</u>	3.3
L2E2	45.3	9.1	<u>4.99</u>	L1E2	88.9	12.0	<u>7.38</u>	3.3

Note: **bold and underline** component fails assessment. 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), and finally shear wall number 1, 2, or 3. More details on the shear walls can be found in [14].

Table 2. Performance summary of new design for shear walls, ties, and chord studs.

2 nd story and story-to-story tie						1 st story and hold-down anchorage				
SW ^a	$\frac{v_{ud}}{v_{ce}}$	$\frac{v_{ud}}{v_{ce}}/m$	Chord ^b		Tie ^c	Conn. ^c		Chord ^c		HD ^f
			$\frac{P_{UF}}{P_{CL}} + \frac{M_{UF}}{M_{CL}}$	$\frac{T_{UF}}{T_{CL}}$	$\frac{T_{UF}}{T_{CL}}$	$\frac{P_{UF}}{P_{CL}} + \frac{M_{UF}}{M_{CL}}$	$\frac{T_{UF}}{T_{CL}}$			
L2S1	2.27	0.69	0.62	0.76	0.84	L1S1	2.95	0.89	0.69	0.73
L2S2	2.56	0.78	0.70	0.85	0.95	L1S2	3.31	1.00	0.89	0.82
L2S3	2.27	0.69	0.59	0.76	0.84	L1S3	2.95	0.89	0.69	0.73
L2N1	1.67	0.50	0.65	0.80	0.89	L1N1	2.16	0.66	0.54	0.78
L2N2	1.13	0.34	0.70	0.83	0.92	L1N2	1.45	0.44	0.59	0.80
L2W1	1.58	0.48	0.52	0.78	0.87	L1W1	2.07	0.63	0.42	0.75
L2W2	1.58	0.48	0.52	0.78	0.87	L1W2	2.07	0.63	0.42	0.75
L2W3	2.53	0.77	0.59	0.87	0.97	L1W3	3.27	0.99	0.84	0.83
L2E1	1.73	0.53	0.59	0.87	0.97	L1E1	2.25	0.68	0.52	0.84
L2E2	2.32	0.70	0.59	0.43	0.96	L1E2	3.01	0.91	0.75	0.83
max	2.56	0.78	0.70	0.87	0.97		3.31	1.00	0.89	0.84

a. 2nd story shear wall: 2-sided, 11 mm OSB, #8 min @ 76 mm o.c., $t=1.37$ mm (54 mil) min,

b. 2nd story chord stud: back-to-back 600S162-54,

c. Tie consists of 2.46 mm (97 mil) 345 MPa strap 122 cm wide and 53 cm long, with 12 staggered #10 screws at each end to studs,

d. 1st story shear wall: 2-sided, 11 mm OSB, #10 min @ 76 mm o.c., $t=1.73$ mm (68 mil) min,

e. 1st story chord stud: back-to-back 600S200-97,

f. Hold-down anchorage consists of 2 S/HDU9 Simpson hold-downs

suggesting ASCE 7 response modification factors (R and Ω_0) are justified. Moreover, if the final building, with sheathing, non-structural walls, and finishes, 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 [19]. Essentially, for this case, and likely this type of building system, ASCE 41's lack of an "easy switch"

to account for system overstrength in the linear assessment procedure is an important reason for the linear analysis providing the pessimistic predictions of performance.

While numerous differences exist between ASCE 7-16 and ASCE 41-17, it is worth parsing out how much of the difference between the two is a function of: (i) R vs m , and (ii) the basic earthquake hazard approximation. ASCE 41-17 has adopted the same basic hazard levels for new design as found in ASCE 7-16. For a building with a period falling in the response spectrum plateau region (as is the case for the two-story CFS-NEES building) the design base shear, V_{DE} , per ASCE 7-16 is:

$$V_{DE} = \frac{(2/3)S_{MS}}{R}W = \frac{S_{DS}}{R}W \quad (7)$$

where W is the weight of the building and S_{MS} is the risk-targeted maximum considered earthquake (MCE_R) as defined in ASCE 7-16. Ostensibly ASCE 41-17 at the BSE-1N level is at the same hazard level as the DE; however, the base shear for this case, V_{BSE-1N} , is calculated using the following:

$$V_{BSE-1N} = C_1C_2 \frac{S_{DS}}{4/(5.6 - \ln(100\beta))}W \quad (8)$$

which is the resulting equation obtained by substituting the spectral acceleration S_a from the plateau section (i.e., $S_a = S_{YS} / B_I$ as defined in Section 2.4 of ASCE 41-17) into Eq. (1), where β is the damping ratio. For the CFS-NEES building, C_1C_2 equals 1.4 and if one assumes β of 2 %, then:

$$V_{BSE-1N} = 1.72S_{DS}W \quad (9)$$

Comparing Eq. (9) of ASCE 41 to Eq. (7) of ASCE 7 with R set to unity, even though the spectral acceleration is the same (S_{DS}), the demands from ASCE 41 are significantly greater than those from ASCE 7 for the same hazard level. Note, for buildings with 5 % damping and longer fundamental periods causing $C_1C_2 = 1.0$, this difference between ASCE 41 and ASCE 7 diminishes and $V_{BSE-1N} = RV_{DE}$. Thus, the manner that short height (short period) buildings are handled in ASCE 41 (specifically Table 7-3 in ASCE 41-17) contributes to design differences in the two standards.

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. There is a stipulation on this conclusion: if the gravity and non-structural wall elements are modelled 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_0 factor, to include the same phenomena the approach in ASCE 41 would require explicit modelling, with significant uncertainty in the parameters.

4 Conclusions

A two-story cold-formed steel (CFS) framed building, previously designed to ASCE 7 and successfully tested on a shake table in the laboratory, was examined to determine necessary changes if ASCE 41 is adopted for assessment. This building fails when assessed as an existing building per ASCE 41, suggesting retrofit is required. A new design of the two-story CFS 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 finding is not justified by the experimentally and numerically validated performance of the building and is caused by two primary factors. First, the basic seismic demands are significantly greater in ASCE 41 than in ASCE 7, especially for short-period structures. Second, large system overstrength, common in repetitively framed structures, is not easily accounted for in the linear procedures of ASCE 41 but is accounted for in ASCE 7 via a simple factor. 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 common design. The evaluation and new design conducted in this paper highlight that, since chord studs, anchorage, and ties are typically designed for the expected strength of the shear walls, the most efficient designs have shear walls that are highly utilized (i.e., with demand-to-capacity ratios that are as close to the ASCE 41 m -factors as possible). 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 CFS-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 buildings.

5 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: Seismic Evaluation and Retrofit of Existing Buildings. ASCE/SEI 41-17, American Society of Civil Engineers, Reston, VA (2017).
2. Ayhan, D., Madsen, R.L., Schafer, B.W.: Progress in the Development of ASCE 41 for Cold-Formed Steel. Proceedings of the 23rd International Specialty Conference on Cold-Formed Steel Structures, pp 417–432 Baltimore, MD (2016).
3. ASCE: Minimum Design Loads for Buildings and Other Structures. ASCE/SEI 7-10, American Society of Civil Engineers, Reston, VA (2010).

4. AISC: Specification for Structural Steel Buildings. ANSI/AISI 360-10. American Institute of Steel Construction, Chicago, IL (2010).
5. AISC: Seismic Provisions for Structural Steel Buildings. ANSI/AISC 341. American Institute of Steel Construction, Chicago, IL (2010).
6. Harris, J.L., Speicher, M.S.: 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 (2015). <http://dx.doi.org/10.6028/NIST.TN.1863-1>
7. Speicher, M.S., Harris, J.L.: Collapse Prevention Seismic Performance Assessment of New Eccentrically Braced Frames using ASCE 41. *Engineering Structures* 117:344–357 (2016).
8. Speicher, M.S., Harris, J.L.: Collapse prevention seismic performance assessment of new special concentrically braced frames using ASCE 41. *Engineering Structures* 126:652–666 (2016). <https://doi.org/10.1016/j.engstruct.2016.07.064>
9. Speicher, M.S., Harris, J.L.: Collapse Prevention seismic performance assessment of new buckling-restrained braced frames using ASCE 41. *Engineering Structures* 164:274–289 (2018). <https://doi.org/10.1016/j.engstruct.2018.01.067>
10. Harris, J.L., Speicher, M.S.: Assessment of Performance-Based Seismic Design Methods in ASCE 41 for New Steel Buildings: Special Moment Frames. *Earthquake Spectra* 34(3):977–999 (2018). <https://doi.org/10.1193/050117EQS079EP>
11. Sattar, S.: Evaluating the consistency between prescriptive and performance-based seismic design approaches for reinforced concrete moment frame buildings. *Engineering Structures*. 174:919-931 (2018). <https://doi.org/10.1016/j.engstruct.2018.07.080>
12. Speicher, M.S., Olivares, I., Schafer, B.W.: Seismic Evaluation of a 2-Story Cold-Formed Steel Framed Building using ASCE 41-17. NIST Technical Note 2116. National Institute of Standards and Technology, Gaithersburg, MD (2020).
13. Speicher, M.S., Zhang, Z., Schafer, B.W.: Application of ASCE 41 to a two-story CFS building, Proceedings of the Cold-Formed Steel Research Consortium (CFSRC) Colloquium, October 20-21, Baltimore, MD (2020).
14. Madsen, R.L., Nakata, N., Schafer, B.W.: CFS-NEES Building Structural Design Narrative. (2013). Available at <http://jhir.library.jhu.edu/handle/1774.2/40584>
15. ASCE: Minimum Design Loads for Buildings and Other Structures. ASCE/SEI 7-16. American Society of Civil Engineers, Reston, VA (2017).
16. AISI: North American Specification for the Design of Cold-Formed Steel Structural Members, AISI S100-16, American Iron and Steel Institute, Washington, D.C. (2016).
17. AISI: North American Standard for Seismic Design of Cold-Formed Steel Structural Systems, AISI S400-15, American Iron and Steel Institute, Washington, D.C. (2015).
18. FEMA: Quantification of Building Seismic Performance Factors, Department of Homeland Security, Washington, D.C. (2009).
19. Peterman, K.D., Stehman, M.J.J., Madsen, R.L., Buonopane, S.G., Nakata, N., Schafer, B.W.: Experimental Seismic Response of a Full-Scale Cold-Formed Steel-Framed Building. I: System-Level Response. *Journal of Structural Engineering* 142(12):04016127. (2016). [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0001577](https://doi.org/10.1061/(ASCE)ST.1943-541X.0001577)
20. Leng, J., Peterman, K.D., Bian, G., Buonopane, S.G., Schafer, B.W.: Modeling seismic response of a full-scale cold-formed steel-framed building. *Engineering Structures* 153:146–165 (2017). <https://doi.org/10.1016/j.engstruct.2017.10.008>
21. Leng, J., Buonopane, S.G., Schafer, B.W.: 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 (2020). <https://doi.org/10.1002/eqe.3245>