NIST Technical Note 2116

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NIST policy is to use the International System of Units (metric units) in all its publications. In this report, however, information is presented in U.S. Customary Units (inch-pound), because those are the units used in the standards being evaluated.

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Abstract

The objective of this report is to assess the adequacy of new provisions in ASCE 41 for seismic assessment of cold-formed steel framed buildings. A two-story cold-formed steel (CFS) framed building that has been designed to contemporary seismic standards (ASCE 7 and AISI S400) and tested on a shake table was selected as the archetype building for this study. Shake table tests of the CFS-framed building indicated only minimal damage at earthquake levels exceeding the ASCE 7 maximum considered earthquake. Further, previously conducted incremental dynamic analyses of the CFS-framed building indicated the ASCE 7 design led to acceptable collapse margin ratios, which equates to acceptable performance. Assessment of the selected CFS-framed building is performed per the linear procedure in ASCE 41. A retrofit design, and a new design, for the same CFS-framed building are also completed per ASCE 41. The ASCE 41 assessment indicates that the building is inadequate, despite the known good performance in experimental shaking and complementary nonlinear time history analyses. The ASCE 41 retrofit requires nearly a doubling in the strength of the shear walls and the remaining elements of the seismic force resisting system. It is shown that ASCE 41's predicted demands for short period buildings, and its lack of a simple means to account for large system overstrength, are the two primary contributors to the overly-conservative predictions from the ASCE 41 provisions. These findings are intended to be used to improve future versions of ASCE 41, with a focus on CFS-framed building provisions.

Key words
earthquake engineering, performance-based design, seismic assessment, building codes, cold-formed steel

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Glossary

AISC – American Institute of Steel Construction
AISI – American Iron and Steel Institute
ASCE – American Society of Civil Engineers
BSE – basic safety earthquake
CFS – cold-formed steel
CP – collapse prevention
DE – design earthquake
ELF – equivalent lateral force
LS – life safety
MCE – maximum considered earthquake
NEES - Network for Earthquake Engineering Simulation
NIST – National Institute of Standards and Technology
NSF – National Science Foundation
OSB – oriented strand board
PBSD – performance-based seismic design

1. Introduction

Performance-based seismic design has gained traction in the U.S. building industry as an alternative way to design new buildings and retrofit existing buildings to resist seismic effects. The current standardized performance-based design methodology is contained within the American Society of Civil Engineers (ASCE) / Structural Engineering Institute (SEI) standard 41 – *Seismic Evaluation and Retrofit for Existing Buildings* – hereafter referred to as ASCE 41. Additions to ASCE 41 in the 2017 edition [1] include complete modeling and acceptance criteria for components of cold-formed steel (CFS) framed building construction. These changes were made using available test data in the literature and by following accepted practice for deriving new parameters. However, there has been limited validation of these CFS-based criteria to ensure reasonable design and assessment outcomes. Therefore, this report examines the relationship between prescriptive new building design approaches and the performance-based existing building standard (ASCE 41). This evaluation is completed by assessing a 2-story CFS building that was tested on a shake table during a previously funded National Science Foundation project. The building is evaluated using linear assessment procedures in ASCE 41. Per the assessment, retrofit schemes are proposed. Additionally, the building is redesigned, as new per ASCE 41, to understand how the building would need to change from the existing code-conforming design.

2. Background and Motivation

In 2015, NIST published a series of reports investigating the relationship between new structural steel building design standards and existing structural steel building assessment standards [2–4]. The NIST studies began by designing a suite of steel frames using the following:

- ASCE 7, *Minimum Design Loads for Buildings and Other Structures* (2010) [5].
- American Institute of Steel Construction (AISC) 360, *Specification for Structural Steel Buildings* (2010) [6].
- AISC 341, *Seismic Provisions for Structural Steel Buildings* [7].

Next, these buildings were assessed using the systematic evaluation (Tier 3) approach in ASCE 41. The evaluation included the following four levels of analysis procedures: linear static, linear dynamic, nonlinear static, and nonlinear dynamic. The basic question raised in the NIST study was, "do the standards for designing new buildings and assessing existing buildings provide consistent levels of performance?" The results from the study, in a broad stroke, show that the "newly-designed buildings," which were code compliant to ASCE 7/AISC360/AISC 341, did not pass an ASCE 41 assessment. The reports, along with follow-on studies [8–12] identified several issues that contributed to the observed outcome. This body of work has given the ASCE standards committees new information that can help inform changes to the provisions, and work is actively underway for this update cycle (next edition of ASCE 41 is due in 2023).

A possible consequence with the finding from the NIST studies is that, with overly conservative ASCE 41 assessment procedures, existing buildings may be unnecessarily retrofitted or existing buildings that would benefit from retrofitting may not receive investment due to the high cost. Further, design engineers considering using ASCE 41 as a performance-based seismic design (PBSD) alternative for the design of new buildings are likely to find the ASCE 41 design route to be overly conservative and, therefore, less desirable. These two scenarios are essentially the opposite of the implicit goal of PBSD, which is to create less restrictive and more efficient designs supported by improved predictions of performance that are closely aligned with finer-tuned desired levels of performance.

In this context, the addition of CFS system assessment criteria into the latest version of ASCE 41 motivates the same type of question: how does the new building design standard compare to the existing building assessment standard? The acceptance criteria were developed based on an extensive database of experimental test results [13]. The database was used to create backbone curves, per the generally acceptable approach outlined in ASCE 41-17 Chapter 7, for each type of system and to then calculate corresponding m-factors (linear acceptance values) and inelastic deformations (nonlinear acceptance values). However, there has been little investigation into how these acceptance criteria correspond to new building design. In other words, will a building designed with a CFS lateral system, per the latest version of the CFS seismic lateral design standard, American Iron and Steel Institute (AISI) S400 [14], pass an ASCE 41 assessment using the new acceptance criteria?

Load bearing CFS-framed buildings generally consist of repetitively framed lipped channel studs and joists. CFS-framed buildings are most commonly ledger-framed (i.e., not platform or balloon framed – see Fig. 1), whereby the joists are hung from the inboard side of the studs using a continuous ledger allowing the joist spacing to be different (commonly less than) the stud spacing (typically 24 in. on center). Lateral stiffness and strength are established through sheathing or strap-bracing segments of the wall and floor. Details of the commonly used seismic force resisting systems in CFS framing and their design and performance are summarized in a 2016 NIST report [15].

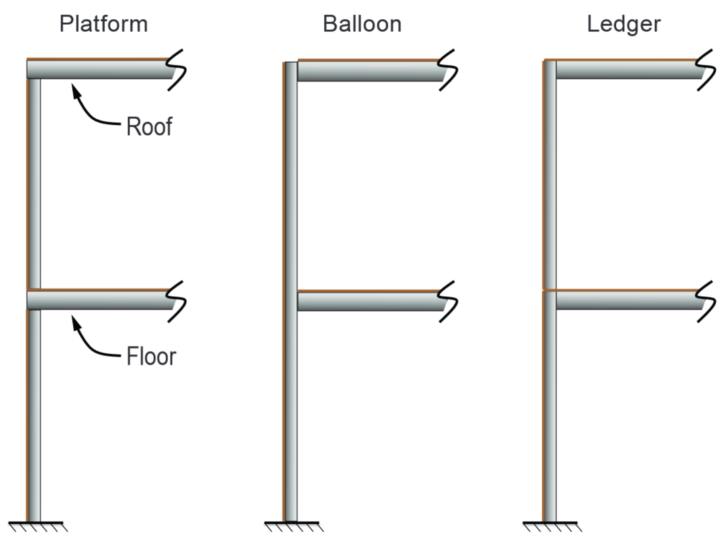


Fig. 1. Illustration of basic CFS framing types [15].

As detailed above, past studies on other hot-rolled steel and reinforced concrete systems have indicated ASCE 41 can be more conservative tan ASCE 7. Performing this comparison between design methodologies will help inform and motivate future efforts to improve both current prescriptive CFS design and performance-based CFS design and assessment. To achieve this, the 2-story CFS 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 (see Fig. 2) is selected as a case study (NSF award 1041578, NEESR-CR: Enabling Performance-Based Seismic Design of Multi-Story Cold-Formed Steel Structures, Principal Investigator: Benjamin Schafer) [16]. This 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 overall building response involved only minor damage even for seismic excitations in excess of ASCE 7's maximum considered earthquake levels [17]. Subsequent nonlinear time history analyses further demonstrated that while the building was efficiently designed with respect to ASCE7/AISI S400 (i.e., design demand/capacity ratios for the shear walls generally near 1.0), the building had substantial strength reserve and more than acceptable collapse probabilities [18, 19].

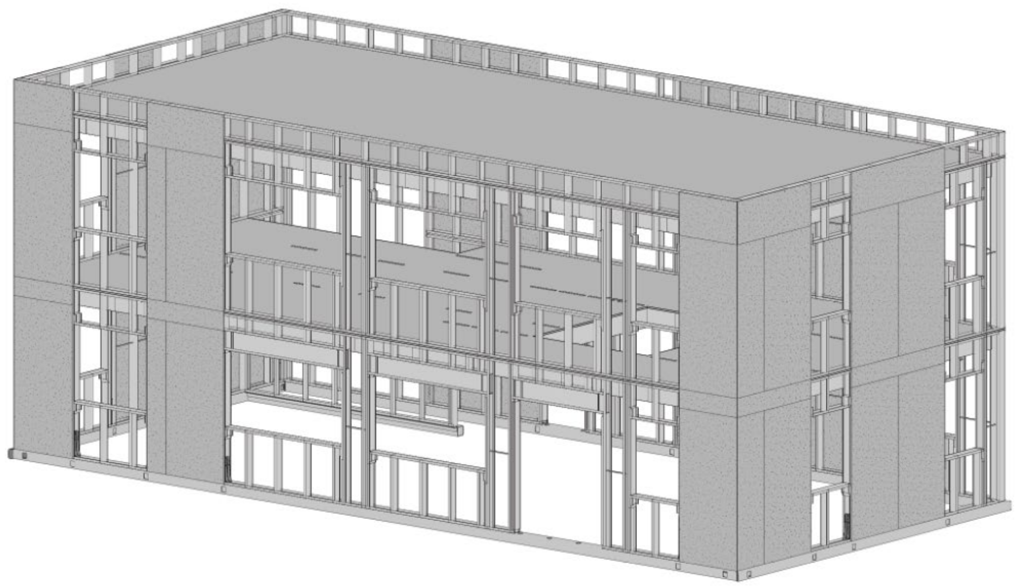


Fig. 2. Isometric of framing for 2-story CFS-NEES building (sheathing depicted only on shear walls) [16].

3. Methodology

3.1. Existing Building Evaluation

The first task in this study was to evaluate the 2-story CFS-NEES building. The original building design was completed per ASCE 7-05, AISI S100-07, and AISI S213-07 as detailed in [16]. The design was updated to satisfy the latest standards, ASCE 7-16 [20], AISI S100-16, and AISI S400-15. Then, the updated design was evaluated as an *existing building* using the linear static procedure of ASCE 41-17. Consistent with current practice, the primary lateral force-resisting system was considered, but secondary effects from the gravity framing system and non-structural elements were ignored. Although ignoring these secondary effects is typical in CFS design practice, and is the only approach consistent with current design codes, the actual response of the tested building included significant contributions from these secondary systems [17, 18].

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."

For these existing building evaluations, only the oriented strand board (OSB) sheathed shear wall capacities were assessed. The chord studs and floor-to-floor ties, as well as hold-down anchorages, are capacity-protected in AISI S400 and thus are sized once the shear wall expected capacities are determined. Figure 3 provides a typical shear wall detail as well as the floor-to-floor ties and hold-down anchorage in the CFS-NEES building.

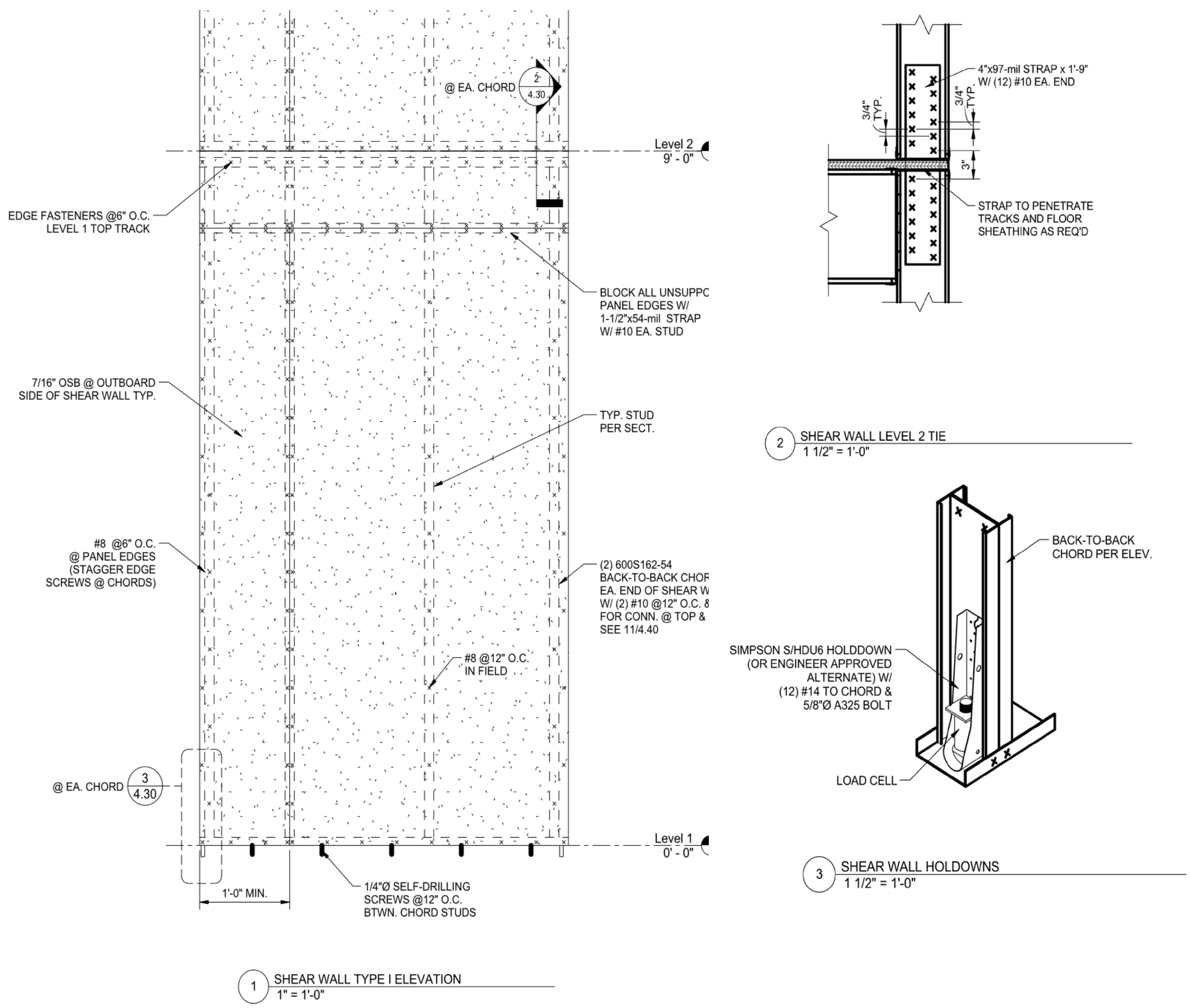


Fig. 3. Typical shear wall elevation, floor-to-floor tie, and anchorage hold-down in CFS-NEES building [16].

3.2. Existing Building Retrofit

After the existing building evaluation was completed, the next task was to design a retrofit that would satisfy the ASCE 41 linear requirements. Practical options for retrofitting the CFS-framed OSB sheathed shear walls include increasing the number of fasteners in the sheathing, or for single-sided shear walls applying an additional side of sheathing. Replacing the sheathing material is also possible, but generally less practical. Of course, if a shear wall is retrofitted, the corresponding chord studs and ties/hold-downs need to be redesigned to accommodate the new expected capacity. Practical options for accomplishing these upgrades are discussed, but the complete designs are not carried out.

3.3. New Building Design

The final task in this project was to create a new design as if the building has not yet been built. This allows freedom not available during the previous task and is expected to produce a more efficient design. For example, stud sizes can now be easily changed, which is a key advantage in the new design versus the retrofit. Even though the new design did allow more freedom, the new design maintained the same layout and number of shear walls as the orginal design, thus only changing stud size, sheathing, fastener speacing, and number of faces sheathed in a wall.

3.4. 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.

In contrast, new building design (per ASCE 7) uses lateral forces reduced by a response modification factor, *R*, and then compares these demands to nominal capacities multiplied by a resistance (reduction) factor, ϕ. The response modification factor, *R*, is comprised of two parts: overstrength (Ω_o) and ductility ($\sim R/\Omega_o$). ASCE 41's *m*-factor is conceptually similar to the ductility portion of *R*. The following subsections outline the approach used to obtain the demands, capacities, and acceptance criteria used in this study.

3.4.1. 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 is calculated from ASCE 41-17 Equation 7-21:

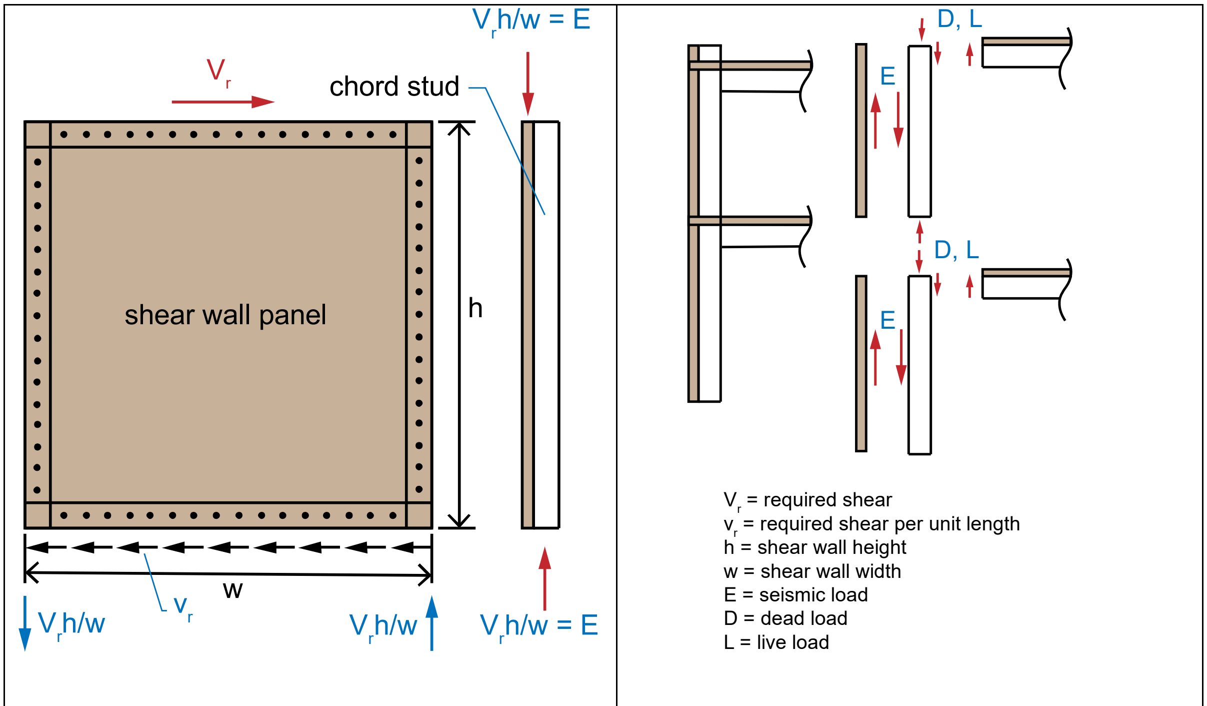
$$V = C_1 C_2 S_a W \tag{1}$$

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 using ASCE 41-17 Equation 7-24. This force distribution is identical to that used in the design using ASCE 7-16 Equation 12.8-11. 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. Note, the lateral forces are not reduced by R, as is the case in the conventional ASCE 7 design.

The demands on the chord studs and ties/hold-downs are determined by treating them as force-controlled components. ASCE 41 gives two options for determining the demands on force-controlled components. The first approach is to use the combination of seismic forces (as used to determine required shear wall capacity) and the gravity loads. The unreduced seismic forces are reduced by dividing by the product of C_1, C_2, and J, where C_1 and C_2 are described above, and J is the force-delivery reduction factor taken as the smallest demand-capacity ratio of the components in the load path delivering force to the component in question. This load combination is then compared with the lower-bound strength of the respective force-controlled component.

The second approach is a "capacity design" approach 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 concept being that the desired energy dissipation occurs in the deformation-controlled shear wall and the boundary members, and that connections for the shear wall must be designed to ensure nearly elastic response up to the expected strength of the deforming shear wall to enable the shear wall to perform in a ductile manner over repeated cycles. 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 often subjected to eccentric loads, primarily due to gravity loads framing into the interior flange of the stud from the ledger, and a smaller amount in each story from sheathing demands originating at the attached flange face, which in total creates a moment on the stud. A free body diagram illustrating the force calculation for the chord studs is shown in Fig. 4.



(a) simplified conversion of seismic shear wall demand to chord stud demand	(b) demands on chord studs considering dead (D) and live (L) load from joists and seismic (E) load from sheathing

Fig. 4. Free body diagrams for (a) shear wall contribution to chord studs and (b) dead, live, and earthquake loading to first and second story chord studs.

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

$$Q = Q_E + 1.1(Q_D + Q_L) \tag{2}$$

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 in a similar manner – considering the expected capacity of the shear wall, and considering the case of counteracting loads where ASCE 41-17 Equation 7-2 holds, adapted here as:

$$Q = Q_E + 0.9(Q_D) \tag{3}$$

3.4.2. Capacity

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

$$v_{ce} = \phi v_n \tag{4}$$

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. Specifically, AISI S400-15 Table E1.3-1 provides strength in lbf per linear ft (plf) based on sheathing type, fastener spacing and size, and stud and track thickness. Adjustment is required for narrow aspect ratio shear walls: if shear wall height, h, divided by width, w, is greater than 2.0, the strength is reduced by the multiplicative factor $2(w / h)$.

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 are calculated based on the nominal strength from AISI S100 (e.g. P_n), but with ϕ set to 1.0 and the yield stress, F_y, determined from lower-bound material properties. Per ASCE 41-17 Section 9.2.2.5.2, the lower-bound yield stress $F_{yL} = 0.85F_{ye}$, where F_{ye} is the expected yield stress. The expected yield stress is defined as R_yF_{yn}, where $R_y = 1.1$ from AISI S400-15 Table A3.2-1, and F_{yn} is the nominal yield stress; thus $F_{yL} = (0.85)(1.1)F_{yn} = 0.94F_{yn}$. It is conservative (by 6 % or less) to use P_{CL}=0.94P_n (or M_{CL}=0.94M_n) for the lower-bound strength where P_n is calculated based on F_{yn}, and this simplification is taken herein.

3.4.3. Acceptance Criteria Check

The 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:

$$m\kappa v_{ce} > v_{ud} \tag{5}$$

where m is the m-factor and κ is the knowledge factor (taken as 1.0 in this report). Equation (5) is equivalent to ASCE 41-17 Equation 7-36. For convenience, Equation (5) can be rearranged and written as:

$$\frac{v_{ud}}{\kappa v_{ce}} < m \tag{6}$$

The results are presented in the form of Equation (6), where the demand-to-capacity ratio must be less than m to pass the acceptance criteria.

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:

$$\frac{1}{\kappa}\left(\frac{P_{UF}}{P_{CL}} + \frac{M_{UF}}{M_{CL}}\right) \leq 1.0 \tag{7}$$

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} should include second order effects and may be approximated as $B_1 M_{UF1}$ where B_1 is the approximate moment magnifier (Equation C1.2.1.1-3 in AISI S100-16 [14]) and M_{UF1} is the first-order demand. An equivalent equation, based on the maximum force delivered from the shear walls, can be used for the ties/hold-downs. The acceptance criteria check for ties/hold-downs is:

$$\frac{1}{\kappa}\left(\frac{T_{UF}}{T_{CL}}\right) \leq 1.0 \tag{8}$$

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. As force-controlled elements, the lower-bound capacity for the ties and hold-downs is taken as 0.94 times the nominal strength, as detailed previously.

4. Results

The results of the ASCE 41 assessments are presented in this section. The first set of results are for the evaluation of the building as an existing building. The next set of results show the retrofit options of the existing building and the associated assessment. Finally, the last set of results show a new lateral design created considering the building as a new design with the same general layout.

The fundamental demand that drives the seismic design of a building is the induced lateral forces. For the CFS-NEES building the lateral demands for the equivalent lateral force (ELF) procedure in ASCE 7-16 are compared with those for ASCE 41-17 in Table 1. Although ASCE 7 and ASCE 41 utilize different design philosophies, the comparison illustrates that even when accounting for ductility (m in ASCE 41, R/Ω_o in ASCE 7) and considering all performance and hazard levels, ASCE 41 demands are greater than ASCE 7 demands for this location and building system.

Table 1. Summary of lateral seismic demands for ELF/linear procedures in ASCE 7 and ASCE 41 for the CFS-NEES building site and design system, where V is the seismic base shear.

	ASCE 7-16			ASCE 41-17											
				LS / BSE-1E			CP / BSE-2E			LS / BSE-1N			LS / BSE-2N		
	$R = 6.5$ $\Omega_o = 3$			$C_1C_2 = 1.4$ $m = 2.5$			$C_1C_2 = 1.4$ $m = 3.3$			$C_1C_2 = 1.4$ $m = 2.5$			$C_1C_2 = 1.4$ $m = 3.3$		
	V	$\frac{V}{R}$	$\frac{V\Omega_o}{R}$	V	$\frac{V}{C_1C_2}$	$\frac{V}{m}$	V	$\frac{V}{C_1C_2}$	$\frac{V}{m}$	V	$\frac{V}{C_1C_2}$	$\frac{V}{m}$	V	$\frac{V}{C_1C_2}$	$\frac{V}{m}$
	(kips)														
Roof level	42	6	19	59	42	24	90	64	27	73	52	29	109	78	33
Floor level	29	4	13	41	29	16	62	44	19	51	36	20	76	54	23
Base Shear	71	11	33	100	71	40	152	108	46	123	88	49	185	132	56

Note, selected response modification coefficients are for CFS-framed building with OSB sheathed shear walls

4.1. Evaluation of Existing Building

As mentioned previously, the original building design was completed per ASCE 7-05, AISI S100-07, and AISI S213-07 as detailed in [16]. The design was updated to satisfy the latest standards: ASCE 7-16, AISI S100-16, and AISI S400-15. With respect to ASCE7-16 vs. ASCE7-05, the seismic response modification coefficients are unchanged (R=6.5, Ω_o=3, C_d=4) while the design base shear for the site decreased by 1.4 % and all other loads, load combinations, seismic response modification coefficients, etc. remained the same. With respect to capacity, the shear wall available strength is unchanged in AISI S400-15 from AISI S213-07. Chord studs and other capacity-based (force-controlled) elements received an updated treatment in AISI S400-15 from AISI S213-07, but the end result was no net change and chord studs may be understood as designed for the expected strength of the shear wall (but not greater than seismic loads at Ω_o levels). Minor member capacity changes from AISI S100-07 to AISI S100-16 have no net change on the chord stud, joist, and track selections in the CFS-NEES building. The average utilization ratio (required strength/available strength) in the new design is 92 % and 64 % for the first story and second story shear walls, respectively (see Table 2). Thus, from the standpoint of current design, the CFS-NEES building represents an efficient and realistic contemporary design.

Table 2. Details of shear walls in CFS-NEES building updated per ASCE 7-16/AISI S400-15 design.

Shear walls sheathed with 7/16 in. OSB (exterior face only), w long, attached at spacing s							
2nd Story				1st Story			
Shear wall	w (ft)	s (in.)	v_r / ϕv_n	Shear wall	w (ft)	s (in.)	v_r / ϕv_n
South face (joists perpendicular to shear walls)							
L2S1	4	6	0.64	L1S1	4	6	0.92
L2S2	5	6	0.63	L1S2	5	6	0.91
L2S3	4	6	0.64	L1S3	4	6	0.92
North face (joists perpendicular to shear walls)							
L2N1	12	6	0.41	L1N1	12	6	0.59
L2N2	8	6	0.35	L1N2	8	6	0.50
West face (joists parallel to shear walls)							
L2W1	4	6	0.51	L1W1	4	6	0.73
L2W2	4	6	0.51	L1W2	4	6	0.73
L2W3	7	6	0.58	L1W3	7	6	0.83
East face (joists parallel to shear walls)							
L2E1	6	6	0.51	L1E1	6	6	0.74
L2E2	8	6	0.58	L1E2	8	6	0.83

Notes: 1. in calculation of v_n the w = 4 ft shear walls are deemed narrow and adjusted for aspect ratio
2. chord studs: back-to-back 600S162-54, field studs 600S162-33 2nd story, 600S162-54 1st story.
3. member naming convention per the Steel Stud Manufacturers Association (SSMA) identification code [21] as described in Appendix A.

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 is shown in Table 3. 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.

The linear static procedure assessment results for the collapse prevention (CP) performance level at the BSE-2E earthquake hazard level is shown in Table 4. Despite the more relaxed m-factor at the collapse prevention (CP) level (m=2.5 for LS vs. 3.3 for CP) the results indicate worse performance than at the life safety (LS) level (BSE-1E) assessment. For the 2nd story, 9 out of 10 shear walls fail the assessment. For the lower story, all the shear walls fail the assessment.

In effect, a building that was deemed adequate per ASCE 7-16/AISI S400-15 is deemed inadequate per ASCE 41-17.

Table 3. Linear static procedure assessment results of the shear walls considering life safety (LS) at the BSE-1E earthquake hazard level, where v is shear per unit length.

2nd Story				1st Story				m-factor
Shear wall	v_{ud} (plf)	v_{ce} (plf)	v_{ud} / v_{ce}	Shear wall	v_{ud} (plf)	v_{ce} (plf)	v_{ud} / v_{ce}	
L2S1	2039	622	**3.28**	L1S1	3465	733	**4.73**	2.5
L2S2	2623	700	**3.75**	L1S2	4434	825	**5.37**	2.5
L2S3	2039	622	**3.28**	L1S3	3465	733	**4.73**	2.5
L2N1	1684	700	2.41	L1N1	2860	825	**3.47**	2.5
L2N2	1152	700	1.65	L1N2	1946	825	2.36	2.5
L2W1	1408	622	2.26	L1W1	2401	733	**3.27**	2.5
L2W2	1408	622	2.26	L1W2	2401	733	**3.27**	2.5
L2W3	2595	700	**3.71**	L1W3	4383	825	**5.31**	2.5
L2E1	1755	700	**2.51**	L1E1	2979	825	**3.61**	2.5
L2E2	2362	700	**3.37**	L1E2	4002	825	**4.85**	2.5

Note: **bold and underline** indicates component that fails assessment.

Table 4. Linear static procedure assessment results of the shear walls considering collapse prevention (CP) at the BSE-2E earthquake hazard level, where v is shear per unit length

2nd Story				1st Story				m-factor
Shear wall	v_{ud} (plf)	v_{ce} (plf)	v_{ud} / v_{ce}	Shear wall	v_{ud} (plf)	v_{ce} (plf)	v_{ud} / v_{ce}	
L2S1	3103	622	**4.99**	L1S1	5273	733	**7.19**	3.3
L2S2	3991	700	**5.70**	L1S2	6747	825	**8.18**	3.3
L2S3	3103	622	**4.99**	L1S3	5273	733	**7.19**	3.3
L2N1	2563	700	**3.66**	L1N1	4352	825	**5.28**	3.3
L2N2	1753	700	2.50	L1N2	2962	825	**3.59**	3.3
L2W1	2142	622	**3.44**	L1W1	3653	733	**4.98**	3.3
L2W2	2142	622	**3.44**	L1W2	3653	733	**4.98**	3.3
L2W3	3948	700	**5.64**	L1W3	6670	825	**8.09**	3.3
L2E1	2671	700	**3.82**	L1E1	4533	825	**5.50**	3.3
L2E2	3103	622	**4.99**	L1E2	6089	825	**7.38**	3.3

Note:

4.2. Retrofit of Existing Building

Since the previous section shows the building designs do not pass the ASCE 41 criteria, this section examines retrofit options.

4.2.1. Shear Walls

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., therefore for practical purposes a 3 in. fastener spacing was first investigated. If a 3 in. 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. fastener spacing, then this was done. Note, the 2nd story shear walls have 33 mil field studs and 54 mil chord studs, as was done in the original design per Madsen et al. [16] the nominal strength is based on the 33 mil field studs; however the expected strength for force-controlled elements is based on the 54 mil chord studs. In addition, AISI S400-15 does not provide a capacity for 7/16 in. OSB attached to 33 mil studs at 3 in. spacing, therefore the value provided for 4 in. spacing is employed.

After iterating through the different options, each shear wall was retrofitted based on each performance level and hazard level combination. The resulting retrofitted shear walls are presented in Table 5 for life safety (LS) at the BSE-1E level. 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. The 2nd story shear walls require double-sided sheathing in the same locations as the 1st story, but the existing 6 in. fastener spacing is adequate.

Results of retrofitted shear walls are presented in Table 6 for the collapse prevention (CP) performance level with the BSE-2E demand. In this example, the retrofit is slightly less onerous than the LS-level retrofit. For the 1st story, shear walls on the South face as well as the longer shear walls on the West and East face, L1W3 and L1E2, require double-sided sheathing. Again, all the 1st story shear walls require 3 in. fastener spacing except L1N2 which can be left with its original 6 in. spacing. For the 2nd story, the same shear walls as the 1st story require double-sided sheathing and, with the exception of the walls on the south face, at least one wall on each face requires additional fasteners to be installed to decrease the fastener spacing to 3 in.

The required retrofits for either the life safety (LS) or collapse prevention (CP) level would be costly; however, they do not require an increase in shear wall length, thus practically they could be accomplished.

Table 5. CFS-NEES retrofit design details for shear walls for life safety (LS) at the BSE-1E earthquake hazard level.

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

Note: **bold** indicates changes from original design.

Table 6. CFS-NEES retrofit for shear walls for collapse prevention (CP) at the BSE-2E earthquake hazard level.

	Original			Retrofit				
SW	Sheathing	sides	s (in.)	Sheathing	sides	s (in.)	v_{ud} / v_{ce}	m-factor
2nd Story								
L2S1	7/16" OSB	1	6	7/16" OSB	**2**	6	2.49	3.3
L2S2	7/16" OSB	1	6	7/16" OSB	**2**	6	2.85	3.3
L2S3	7/16" OSB	1	6	7/16" OSB	**2**	6	2.49	3.3
L2N1	7/16" OSB	1	6	7/16" OSB	1	**3**	3.08	3.3
L2N2	7/16" OSB	1	6	7/16" OSB	1	6	1.95	3.3
L2W1	7/16" OSB	1	6	7/16" OSB	1	**3**	3.27	3.3
L2W2	7/16" OSB	1	6	7/16" OSB	1	**3**	3.27	3.3
L2W3	7/16" OSB	1	6	7/16" OSB	**2**	6	2.40	3.3
L2E1	7/16" OSB	1	6	7/16" OSB	1	**3**	2.94	3.3
L2E2	7/16" OSB	1	6	7/16" OSB	**2**	**3**	1.96	3.3
1st Story								
L1S1	7/16" OSB	1	6	7/16" OSB	**2**	**3**	1.93	3.3
L1S2	7/16" OSB	1	6	7/16" OSB	**2**	**3**	2.17	3.3
L1S3	7/16" OSB	1	6	7/16" OSB	**2**	**3**	1.93	3.3
L1N1	7/16" OSB	1	6	7/16" OSB	1	**3**	3.04	3.3
L1N2	7/16" OSB	1	6	7/16" OSB	1	6	2.95	3.3
L1W1	7/16" OSB	1	6	7/16" OSB	1	**3**	2.70	3.3
L1W2	7/16" OSB	1	6	7/16" OSB	1	**3**	2.70	3.3
L1W3	7/16" OSB	1	6	7/16" OSB	**2**	**3**	2.14	3.3
L1E1	7/16" OSB	1	6	7/16" OSB	1	**3**	2.95	3.3
L1E2	7/16" OSB	1	6	7/16" OSB	**2**	**3**	1.97	3.3

Note: **bold** indicates changes from original design.

4.2.2. Chord Studs

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. The demands P_{UF} and M_{UF} are determined per Section 3.4.1 and the capacity P_{CL} and M_{CL} are determined per Section 3.4.2. The interaction is evaluated per Equation (7) with κ assumed to be 1.0. At both the life safety BSE-1E hazard level and the collapse prevention BSE-2E hazard level, the existing 2nd story chord studs are adequate for the retrofit, but none of the existing 1st story chord studs are adequate.

Given the existing 1st story shear wall chord studs (back-to-back 600S162-54's) are insufficient, retrofit options are possible, but none are without complication. The simplest option would be to add stud(s) adjacent to the existing chord studs. Existing walls have sheathing on only one side, so adding stud(s) inside the wall would be the obvious option –

these studs would need to be connected to the existing chord studs and to the sheathing. Adding studs outboard of the existing chord studs is also possible, but the sheathing would need to be replaced and extended. Replacement of an existing stud with a thicker option requires temporarily removing the gravity load during the retrofit – possible, but costly. If simple retrofit options are impractical, another consideration is adding a new shear wall in another part of the building to reduce the demands on each existing shear wall.

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 7 and for the collapse prevention BSE-2E hazard level in Table 8. The results of the Equation (7) 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; however, for the collapse prevention BSE-2E level several shear walls require two additional studs (for a total of 4) on the East wall and one on the North wall.

Table 7. 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.

2nd Story				1st Story			
SW	Existing Chord Stud	Retrofit Chord Stud	Int'n	SW	Existing Chord Stud	Retrofit Chord Stud	Int'n
L2S1	(2) 600S162-54	No change	0.54	L1S1	(2) 600S162-54	**(3) 600S162-54**	1.00
L2S2	(2) 600S162-54	No change	0.56	L1S2	(2) 600S162-54	**(3) 600S162-54**	1.03
L2S3	(2) 600S162-54	No change	0.52	L1S3	(2) 600S162-54	**(3) 600S162-54**	0.99
L2N1	(2) 600S162-54	No change	0.58	L1N1	(2) 600S162-54	**(3) 600S162-54**	0.96
L2N2	(2) 600S162-54	No change	0.57	L1N2	(2) 600S162-54	**(3) 600S162-54**	0.94
L2W1	(2) 600S162-54	No change	0.52	L1W1	(2) 600S162-54	**(3) 600S162-54**	0.82
L2W2	(2) 600S162-54	No change	0.52	L1W2	(2) 600S162-54	**(3) 600S162-54**	0.82
L2W3	(2) 600S162-54	No change	0.52	L1W3	(2) 600S162-54	**(3) 600S162-54**	1.01
L2E1	(2) 600S162-54	No change	0.52	L1E1	(2) 600S162-54	**(3) 600S162-54**	1.01
L2E2	(2) 600S162-54	No change	0.52	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

Table 8. Linear static procedure assessment results of the chord studs considering expected capacities from shear walls retrofitted to meet collapse prevention (CP) at the BSE-2E earthquake hazard level.

2nd Story				1st Story			
SW	Existing Chord Stud	Retrofit Chord Stud	Int'n	SW	Existing Chord Stud	Retrofit Chord Stud	Int'n
L2S1	(2) 600S162-54	No change	0.55	L1S1	(2) 600S162-54	**(3) 600S162-54**	1.01
L2S2	(2) 600S162-54	No change	0.57	L1S2	(2) 600S162-54	**(3) 600S162-54**	1.04
L2S3	(2) 600S162-54	No change	0.52	L1S3	(2) 600S162-54	**(3) 600S162-54**	1.00
L2N1	(2) 600S162-54	No change	0.80	L1N1	(2) 600S162-54	**(4) 600S162-54**	0.78
L2N2	(2) 600S162-54	No change	0.57	L1N2	(2) 600S162-54	**(3) 600S162-54**	0.60
L2W1	(2) 600S162-54	No change	0.70	L1W1	(2) 600S162-54	**(3) 600S162-54**	0.91
L2W2	(2) 600S162-54	No change	0.46	L1W2	(2) 600S162-54	**(3) 600S162-54**	0.91
L2W3	(2) 600S162-54	No change	0.52	L1W3	(2) 600S162-54	**(3) 600S162-54**	1.01
L2E1	(2) 600S162-54	No change	0.53	L1E1	(2) 600S162-54	**(4) 600S162-54**	0.75
L2E2	(2) 600S162-54	No change	0.68	L1E2	(2) 600S162-54	**(4) 600S162-54**	0.84

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

4.2.3. Ties and Hold-Downs

The retrofit design requires increased capacity of the shear walls and this also potentially influences the existing story-to-story ties (Fig. 3b) and the hold-down anchorage (Fig. 3c). The demands, T_{UF}, are determined per Section 3.4.1 and must consider the load combination for counteracting loads. The capacities, T_{CL}, are determined per Section 3.4.2. 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. At the collapse prevention BSE-2E hazard level one 1st-to-2nd story tie is inadequate and again all hold-downs are inadequate as detailed in Table 9 and Table 10.

Retrofit for the story-to-story ties can be completed relatively simply with the addition of an additional strap on the opposite face from the existing strap. The original design also considered the compressive capacity of the strap in an effort to avoid localized damage of the studs in bearing against the OSB floor [16]. For higher demands, this approach becomes more and more impractical and the introduction of a bearing plate at the OSB floor level becomes necessary. Alternatives to using straps for the ties exist, such as the addition of continuous tie rods as explained in the NIST Techbrief [15], this approach may be necessary for higher demands but introduces its own retrofit challenges.

Retrofit of the foundation-to-1st story hold-down can be also 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 re-design 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.

The only tie requiring retrofit occurs at the collapse prevention (CP) performance level (BS-2E earthquake hazard level) on the chord studs of the L2E2 shear wall. It is recommended to simply double up the strap and connector on these chord studs. All other ties are adequate at both the life safety (LS) and collapse prevention (CP) performance levels.

In contrast to the ties, all the hold-downs require retrofit to handle the increased demands. 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. anchor bolt. Full details are provided in Table 9 and Table 10.

Table 9. Linear static procedure assessment results of the ties and hold-downs considering expected capacities from shear walls retrofitted to meet life safety (LS) at the BSE-1E earthquake hazard level.

1st to 2nd Story Tie					Foundation to 1st Story Hold-down			
SW	Existing Tie (Strap)	Retrofit Tie	Strap T_{UF}/T_{CL}	Connector T_{UF}/T_{CL}	SW	Existing Hold-down	Retrofit Hold-down	T_{UF}/T_{CL}
L2S1	97-mil 12#10 each end[1]	No Change	0.66	0.74	L1S1	S/HDU6	**(2) S/HDU9**	0.80
L2S2	97-mil 12#10 each end[1]	No Change	0.74	0.83	L1S2	S/HDU6	**(2) S/HDU9**	0.90
L2S3	97-mil 12#10 each end[1]	No Change	0.66	0.74	L1S3	S/HDU6	**(2) S/HDU9**	0.80
L2N1	97-mil 12#10 each end[1]	No Change	0.30	0.33	L1N1	S/HDU6	**(2) S/HDU6**	0.64
L2N2	97-mil 12#10 each end[1]	No Change	0.33	0.37	L1N2	S/HDU6	**(2) S/HDU6**	0.68
L2W1	97-mil 12#10 each end[1]	No Change	0.33	0.37	L1W1	S/HDU6	**(2) S/HDU6**	0.66
L2W2	97-mil 12#10 each end[1]	No Change	0.33	0.37	L1W2	S/HDU6	**(2) S/HDU6**	0.66
L2W3	97-mil 12#10 each end[1]	No Change	0.76	0.85	L1W3	S/HDU6	**(2) S/HDU9**	0.91
L2E1	97-mil 12#10 each end[1]	No Change	0.76	0.85	L1E1	S/HDU6	**(2) S/HDU9**	0.91
L2E2	97-mil 12#10 each end[1]	No Change	0.76	0.84	L1E2	S/HDU6	**(2) S/HDU9**	0.91

1. Existing floor-to-floor tie detail 4"x97-mil STRAP x 1'-9", 12#10 each end

Table 10. Linear static procedure assessment results of the ties and hold-downs considering expected capacities from shear walls retrofitted to meet collapse prevention (CP) at the BSE-1E earthquake hazard level.

1st to 2nd Story Tie					Foundation to 1st Story Hold-down			
SW	Existing Tie (Strap)	Retrofit Tie	Strap T_{UF}/T_{CL}	Connector T_{UF}/T_{CL}	SW	Existing Holddown	Retrofit Holddown	T_{UF}/T_{CL}
L2S1	97-mil 12#10 each end[1]	No Change	0.66	0.74	L1S1	S/HDU6	**(2) S/HDU9**	0.80
L2S2	97-mil 12#10 each end[1]	No Change	0.74	0.83	L1S2	S/HDU6	**(2) S/HDU9**	0.90
L2S3	97-mil 12#10 each end[1]	No Change	0.66	0.74	L1S3	S/HDU6	**(2) S/HDU9**	0.80
L2N1	97-mil 12#10 each end[1]	No Change	0.42	0.47	L1N1	S/HDU6	**(2) S/HDU6**	0.72
L2N2	97-mil 12#10 each end[1]	No Change	0.33	0.37	L1N2	S/HDU6	**(2) S/HDU6**	0.45
L2W1	97-mil 12#10 each end[1]	No Change	0.44	0.49	L1W1	S/HDU6	**(2) S/HDU6**	0.73
L2W2	97-mil 12#10 each end[1]	No Change	0.44	0.49	L1W2	S/HDU6	**(2) S/HDU6**	0.73
L2W3	97-mil 12#10 each end[1]	No Change	0.76	0.85	L1W3	S/HDU6	**(2) S/HDU9**	0.91
L2E1	97-mil 12#10 each end[1]	No Change	0.49	0.55	L1E1	S/HDU6	**(2) S/HDU9**	0.50
L2E2	97-mil 12#10 each end[1]	**2 x original**	0.50	0.55	L1E2	S/HDU6	**(2) S/HDU9**	1.01

1. Existing floor-to-floor tie detail 4"x97-mil STRAP x 1'-9", 12#10 each end

4.3. Design of New Building

Following the same basic procedures as the evaluation and retrofit, a new lateral design was performed for the CFS-NEES building using current ASCE 41 requirements. The gravity design of Madsen et al. [16] was assumed to be adequate. Thus, 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. Consistent with the evaluation work, two performance levels were considered: life safety (LS) and collapse prevention (CP). These two performance levels were assessed at the earthquake hazard levels consistent with new design, therefore LS at BSE-1N and CP at BSE-2N. The increased lateral demands that are associated with these performance and hazard levels are provided in Table 1.

For simplicity, and to aid comparison across the designs, it was decided to try to keep the shear wall lengths and locations the same as in the original building. (In the original design, shear wall locations were largely driven by architectural demands, and thus it was not desired to change this aspect.) Further, it was also decided to try to use the same lateral system and sheathing (7/16" OSB) as the original design. Thus, the design choices reduced to the following: single- or double-sided shear walls, fastener spacing, stud/framing thickness, chord stud dimensions and number, strap dimensions and number of connectors for story-to-story ties, and type and number of foundation-to-1st story hold-downs. Even with this relatively constrained design problem, the process is more iterative than may be expected.

Gravity design requires 600S162-33 mil field studs for the 2nd story and similar 54 mil field studs for the 1st story. Following the original CFS-NEES shear wall layout, single-sided shear walls are possible with 7/16" OSB fastened @ 3 in. on center (o.c.) on the 2nd story and 2 in. o.c. for the 1st story. Single-sided shear walls are preferred from the standpoint of providing services in the wall cavity. However, back-to-back chord studs are inadequate at 33 mils for the 2nd story and 54 mils for the 1st story. If the stud thickness is increased to improve the adequacy of the chord studs, the shear wall capacity increases – leading to a larger expected demand on the already inadequate chord studs. Double-sided sheathing decreases the eccentric demands on the chord studs, and since the chord studs are the limiting factor this benefit is significant.

Iteration of the shear wall and chord stud design for the LS performance level at the BSE-1N earthquake hazard level is summarized in Table 11. The design freedom is somewhat limited by the available (tabled) systems in AISI S400-15. For the 2nd story, it is found that either single-sided 7/16" OSB sheathing with fasteners @ 3 in. o.c. or double-sided 7/16" OSB sheathing with fasteners @ 6 in. o.c. are adequate so long as the chord studs (back-to-back 600S162) are increased to 54 mil thickness. For the 1st story, double-sided 7/16" OSB perimeter fastened at 6 in. o.c. with back-to-back 600S162-97 mil studs for the chord stud are required. The chord stud demands are significant, even in just a two-story building, requiring a 97 mil stud when gravity framing only requires a 54 mil stud at the same location. The use of the thicker 97 mil chord stud is outside of the scope of AISI S400-15, which limits thickness to 68 mil in this case. However, the 97 mil stud is selected here to maintain the use of OSB sheathed shear walls in the design. Testing shows inadequate performance is achieved with 97 mil studs if fasteners are inappropriately selected, i.e. #8 or smaller [22], in addition in the same standard (AISI S400-15) Canada allows 97 mil studs for OSB sheathed shear walls and both the U.S. and Canada allow 97 mil studs for steel sheet sheathed shear walls. Thus, this relaxation of the AISI S400-15 provisions is implemented in this report.

Table 11. Summary of new design iterations for shear walls and chord studs at the life safety (LS) performance level with the BSE-1N earthquake hazard level

			Shear wall			Chord Stud[b]	
Story	sides	sheathing	s (in)	t (mils)	utilization[c]	Adequate?	Int'n[d]
2nd	1	7/16" OSB	4[a]	33	NG	-	-
	2	7/16" OSB	6	33	92%	NG	1.44
	1	7/16" OSB	3	43	83%	NG	1.07
	2	7/16" OSB	6	43	78%	NG	1.13
	1	**7/16" OSB**	**3**	**54**	**91%**	**OK**	**0.55**
	2	**7/16" OSB**	**6**	**54**	**68%**	**OK**	**0.69**
1st	1	7/16" OSB	2	54	92%	NG	3.25
	2	7/16" OSB	4	54	77%	NG	2.29
	1	7/16" OSB	3	68	94%	NG	2.08
	2	7/16"OSB	6	68	88%	NG	1.43
	1	7/16" OSB	3	97	94%	NG	1.24
	2	**7/16"OSB**	**6**	**97**	**88%**	**OK**	**0.91**

a. AISI S400-15 does not provide strength at 3 in. o.c., 4 in o.c. is the tightest spacing for 33 mil studs
b. Chord studs are back-to-back 600S162 with thickness to match t (mils)
c. max$[(v_{ud}/v_{ce})/m]$ for shear walls d. max$[P_{UF}/P_{CL} + M_{UF}/M_{CL}]$ for chord studs

Design of the story-to-story ties and the hold-down anchorages proceeds relatively directly once the shear wall and chord studs are selected. The ties consist of a strap extending from stud-to-stud, which are checked for yielding in the gross section and fracture in the net section. In addition, the length of the strap is controlled by the level of shear transfer and the required number of fasteners – i.e., the connection strength. The hold-down is selected from Simpson Strong-Tie's S/HDU hold-down components – but could be separately designed if a non-proprietary solution is desired.

The results of the final design – broken out for each shear wall and the newly designed components are provided for the life safety (LS) performance level at the BSE-1N hazard level in Table 12 and the collapse prevention (CP) performance level at the BSE-2N hazard level in Table 13. For both performance level / hazard level combinations, the following designs are found to be adequate and selected:

- 2nd story shear walls: 2-sided, 7/16" OSB, #8 min @ 6 in. o.c., t=54 mil,
- 2nd story chord studs: back-to-back 600S162-54,
- tie: 4" wide 97 mil 50 ksi strap 1'-9" long, connected with 12 staggered #10 screws at each end of strap to studs,
- 1st story shear walls: 2-sided, 7/16" OSB, #10 min @ 6 in. o.c., t=68 mil
- 1st story chord studs:
    - Life safety (LS) performance level: back-to-back 600S162-97 studs
    - Collapse prevention (CP) performance level: back-to-back 600S200-97 studs
- Hold-down anchorage: 2 S/HDU9 Simpson hold-downs

The efficiency of the design may be judged, in part, by the utilization ratios provided in Table 11 (LS at BSE-1N) and Table 12 (CP at the BSE-2N). For construction efficiency it was decided to keep the shear wall configurations uniform – this influences the structural efficiency. The shear wall length and aspect ratio were largely determined by the architectural openings which results in some shear walls (e.g. L1S2) being more highly utilized than others (e.g. L1N2). Since essentially the same design was adequate for both the life safety (LS) and collapse prevention (CP) assessments, it is not a surprise that the utilization ratios at the CP level are higher (compare Table 13 with Table 12). In fact, at the CP level the 1st story shear wall L1S2 is exactly at an m-factor of 3.3 (ratio of 1.0), and thus greater efficiencies are not possible.

Table 12. Performance summary of new design for shear walls, ties, and chord studs at the life safety (LS) performance level with the BSE-1N earthquake hazard level.

2nd story and story-to-story tie						1st story and holddown anchorage				
			Chord[b]	Tie[c]	Conn.[c]				Chord[e]	HD[f]
SW[a]	$\frac{v_{ud}}{v_{ce}}$	$\frac{v_{ud}/v_{ce}}{m}$	$\frac{P_{UF}}{P_{Cl}}+\frac{M_{UF}}{M_{CL}}$	$\frac{T_{UF}}{T_{CL}}$	$\frac{T_{UF}}{T_{CL}}$	SW[d]	$\frac{v_{ud}}{v_{ce}}$	$\frac{v_{ud}/v_{ce}}{m}$	$\frac{P_{UF}}{P_{Cl}}+\frac{M_{UF}}{M_{CL}}$	$\frac{T_{UF}}{T_{CL}}$
L2S1	1.52	0.61	0.61	0.76	0.84	L1S1	1.97	0.79	0.62	0.73
L2S2	1.71	0.68	0.69	0.85	0.95	L1S2	2.21	0.88	0.73	0.82
L2S3	1.52	0.61	0.58	0.76	0.84	L1S3	1.97	0.79	0.61	0.73
L2N1	1.11	0.44	0.65	0.80	0.89	L1N1	1.44	0.58	0.79	0.78
L2N2	0.75	0.30	0.69	0.83	0.92	L1N2	0.97	0.39	0.91	0.70
L2W1	1.06	0.42	0.52	0.78	0.87	L1W1	1.37	0.55	0.71	0.75
L2W2	1.06	0.42	0.52	0.78	0.87	L1W2	1.37	0.55	0.71	0.75
L2W3	1.69	0.67	0.59	0.87	0.97	L1W3	2.18	0.87	0.69	0.83
L2E1	1.16	0.46	0.59	0.87	0.97	L1E1	1.50	0.60	0.77	0.84
L2E2	1.55	0.62	0.59	0.43	0.96	L1E2	2.01	0.80	0.63	0.83
max	**1.71**	**0.68**	**0.69**	**0.87**	**0.97**		**2.21**	**0.88**	**0.91**	**0.84**

a. 2nd story shear wall: 2-sided, 7/16" OSB, #8 min @ 6 in. o.c., t=54 mil min, **b.** 2nd story chord stud: back-to-back 600S162-54, **c.** Tie consists of 97 mil 50 ksi strap 4' wide and 1'9" long, with 12 staggered #10 screws at each end to studs, **d.** 1st story shear wall: 2-sided, 7/16" OSB, #10 min @ 6 in. o.c., t=68 mil min, **e.** 1st story chord stud: back-to-back 600S162-97, **f.** Hold-down anchorage consists of 2 S/HDU9 Simpson hold-downs

Table 13. Performance summary of new design for shear walls, ties, and chord studs at the collapse prevention (CP) performance level with the BSE-2N earthquake hazard level.

2nd story and story-to-story tie						1st story and holddown anchorage				
			Chord[b]	Tie[c]	Conn.[c]				Chord[e]	HD[f]
SW[a]	$\frac{v_{ud}}{v_{ce}}$	$\frac{v_{ud}/v_{ce}}{m}$	$\frac{P_{UF}}{P_{Cl}}+\frac{M_{UF}}{M_{CL}}$	$\frac{T_{UF}}{T_{CL}}$	$\frac{T_{UF}}{T_{CL}}$	SW[d]	$\frac{v_{ud}}{v_{ce}}$	$\frac{v_{ud}/v_{ce}}{m}$	$\frac{P_{UF}}{P_{Cl}}+\frac{M_{UF}}{M_{CL}}$	$\frac{T_{UF}}{T_{CL}}$
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, 7/16" OSB, #8 min @ 6 in. o.c., t=54 mil min, **b.** 2nd story chord stud: back-to-back 600S162-54, **c.** Tie consists of 97 mil 50 ksi strap 4' wide and 1'9" long, with 12 staggered #10 screws at each end to studs, **d.** 1st story shear wall: 2-sided, 7/16" OSB, #10 min @ 6 in. o.c., t=68 mil min, **e.** 1st story chord stud: back-to-back 600S**200**-97, **f.** Hold-down anchorage consists of 2 S/HDU9 Simpson hold-downs

5. 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 [17]. 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 [18]. Examination of the ASCE 7 seismic response modification factors using the FEMA P695 [24] 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), the collapse probabilities are unacceptable. In contrast, if the shear walls and all the gravity framing (unsheathed) were considered, 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, 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 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.

From a practical standpoint, the impact of ASCE 41-17's conservatism is captured in the design changes approximated in Table 14: double-sided shear walls, thicker studs, and more hold-downs. The cost of ASCE 41-17's conservatism for CFS-framed buildings may be estimated by first noting 43 % of the perimeter walls are shear walls and, coincidentally, 43 % of the structural framing is in the walls (as opposed to the floors and roof). If the new shear walls are assumed to cost twice the original walls (see Table 14) and the framing costs for walls and floors is approximately equal per area (reasonable for this small footprint building, with a small number of stories), the rough percentage increase in the cost of the structure for adopting ASCE 41-17 over ASCE 7-16 even at the BSE-1E hazard level is at least $0.43 \times 0.43 = 0.18 = 18$ %.

Table 14. Summary of key design changes between ASCE7-16 and ASCE41-17 for CFS-NEES building (new design).

	ASCE7-16	ASCE41-17	Cost
Typ. Shear wall	1-sided OSB 6 in. spacing 54 mil	2-sided OSB 6 in. spacing 97 mil	> 2 x $ (Constrains intro. of services)[a]
Typ. Chord Stud	600S162-54	600S162-97	~ 2 x $
Typ. Anchorage	S/HDU 6	S/HDU 9 x 2	> 2 x $

a. Limits available access for building services (e.g., water, electrical, and gas)

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 methods is a function of R vs m, and how much is a function of differences in the basic earthquake hazard approximation. For ASCE 7-10 and ASCE 41-13, the spectral acceleration at the BSE-2E earthquake hazard level was closest to that in ASCE 7 and any differences were site specific. However, ASCE 41-17 has adopted the same basic hazard levels for new design as found in ASCE 7-16. If the fundamental period of the building is in the plateau region of the response spectrum as illustrated in Fig. 5, the ASCE 41-17 hazard levels and ASCE 7-16 design levels can be summarized as given in Table 15 where the BSE-1N level in ASCE 41-17 is the same as the design earthquake (DE) level in ASCE 7-16. For a building period 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:

$$V_{DE} = \frac{(2/3)S_{MS}}{R}W = \frac{S_{DS}}{R}W \tag{9}$$

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

$$V_{BSE-1N} = C_1C_2\frac{S_{DS}}{4/\left(5.6-\ln(100\beta)\right)}W \tag{10}$$

which is the resulting equation obtained by substituting the spectral acceleration from the plateau section (i.e., $S_a = S_{XS} \;/\; B_1$ as defined in Section 2.4 of ASCE 41-17, where B_1 is the denominator in Equation (10) above) into Equation (1), where β is the damping ratio. For the CFS-NEES building, C_1C_2 (for definition see Section 3.4.1) equals 1.4 and if one assumes β (damping) equals 2 %, then:

$$V_{BSE-1N} = 1.72S_{DS}W \tag{11}$$

If one compares ASCE 41, Equation (11), to ASCE 7, Equation (9) with R set to unity, even though the spectral acceleration is the same (S_{DS}), ASCE 41's demands are significantly greater than ASCE 7's for the same considered hazard level.

Note, for buildings with 5 % damping and a larger number of stories, and thus longer period causing $C_1C_2 \rightarrow 1$, this difference between ASCE 41 and ASCE 7 diminishes and $V_{BSE-1N} \rightarrow RV_{DE}$. Thus, the manner that short height (short period) buildings are handled in ASCE 41 (specifically Table 7-3 in ASCE 41-17) significantly contributes to design output differences in the two standards.

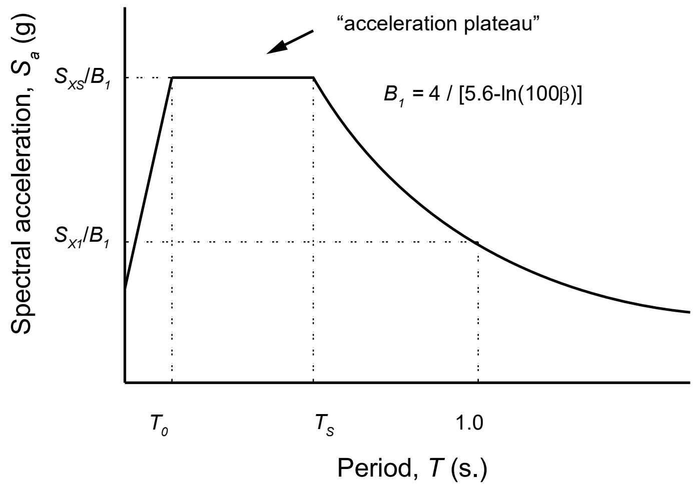


Fig. 5. Generalized acceleration response spectrum.

Table 15. Hazard levels and response spectra values for ASCE 41-17 and ASCE 7-16 including specific values at the CFS-NEES building location.

Standard	Hazard Level	Detail of Hazard	Design Short Period Acceleration Response, S_{XS} or S_{DS}
ASCE 41-17	BSE-1E	20 % probability of exceedance in 50 years	0.75
ASCE 41-17	BSE-2E	5 % probability of exceedance in 50 years	1.14
ASCE 41-17	BSE-1N	2/3 × BSE-2N = 2/3 × MCE	0.93
ASCE 7-16	DE	2/3 × MCE	0.93
ASCE 41-17	BSE-2N	$S_{XS} = S_{MS}$, per ASCE 7 Section 11.4	1.38
ASCE 7-16	MCE	S_{MS}, risk-targeted MCE, i.e., MCE_R	1.38

In ASCE 41 the hazard levels for existing buildings (BSE-1E and BSE-2E) presumably provide some relief compared with those for new design. As Table 15 shows, the spectral acceleration for the BSE-1E hazard (0.75 g) is indeed reduced from that of new design (0.93 g for both BSE-1N in ASCE 41 and DE in ASCE7). However, once the acceleration is amplified in ASCE 41 by C_1C_2 and given that the acceptance criteria only account for ductility (compare m vs R) the end result is that the ASCE 7 design fails the assessment at the BSE-1E level (as illustrated herein). This is particularly pronounced for short period buildings with high system overstrength.

To further compare ASCE 41 to ASCE 7 for the broader set of lateral systems used in CFS shear wall construction, it's helpful to note that ASCE 41-17 supports the same lateral systems as AISI S400-15: OSB and plywood sheathed shear walls, steel sheet sheathed shear walls, and strap-braced walls. Steel sheet sheathed and strap-braced shear walls are both capable of delivering similar nominal capacities as the OSB sheathed shear walls used in the original CFS-NEES building. The ASCE 7-16 seismic response modification coefficients and the ASCE 41-17 m-factors and C_1C_2 factors at the life safety (LS) and collapse prevention (CP) performance levels for these additional systems are provided in Table 16. From Table 16, one can observe that only strap-braced walls (without gypsum boards, or with the gypsum isolated from the panel through resilient channels or other means) in ASCE 41-17 provide a potentially more liberal solution than ASCE 7-16 (e.g., strap-brace m = 4.4 for life safety compared to OSB sheathing m = 2.5). In all other cases, similar to OSB sheathing, it should be anticipated that ASCE 41-17's requirements will be more stringent than ASCE 7-16's requirements.

Table 16. Comparison of seismic response modification factors and m-factors for common CFS seismic force resisting systems.

	ASCE7-16			ASCE41-17			
	R	Ω_o	$\frac{R}{\Omega_o}$	LS m	CP m	LS C_1C_2	CP C_1C_2
OSB Sheathing	6.5	3	2.2	2.5	3.3	1.4	1.4
Struct 1. Plywood Sheathing	6.5	3	2.2	1.9	2.4	1.1	1.4
Steel Sheet Sheathing	6.5	3	2.2	1.6	1.9	1.1	1.1
Strap-braced	4	2	2.0	4.4	4.9	1.4	1.4
Strap-braced with gypsum	4	2	2.0	1.8	2.4	1.1	1.4

The use of nonlinear static or nonlinear dynamic procedures could provide further insight into 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, 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.

6. Summary and 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, or new design, 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 and is expected to increase cost of the building's structure by at least 18 %. The two primary factors contribute to the conservative nature of ASCE 41's predictions are: (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. The retrofit and redesign conducted herein 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 which 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 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.

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Appendix A Supplemental Information

Figure 6 illustrates the naming convention for CFS members. Standard studs and tracks are identified by the Steel Stud Manufacturers Association (SSMA) identification code [21]. The identification codes are formed by a four-part identification code:

1. Depth in 1/100th inches. For studs, the depth is the outside depth. For tracks, the depth is the inside depth (the depth of the stud the track fits over).
2. Style: S =Stud (C-Section with Lips), T = Track (C-Section without Lips)
3. Flange Width in 1/100th inches.
4. Minimum base material thickness (95 % of design thickness) in 1/1000th inches

For example, a section with the designation 600S162-54 is a stud (C-section with lips), with a depth of 6 inches, a flange width of 1 5/8 inches, and a minimum thickness of 0.054 inches.

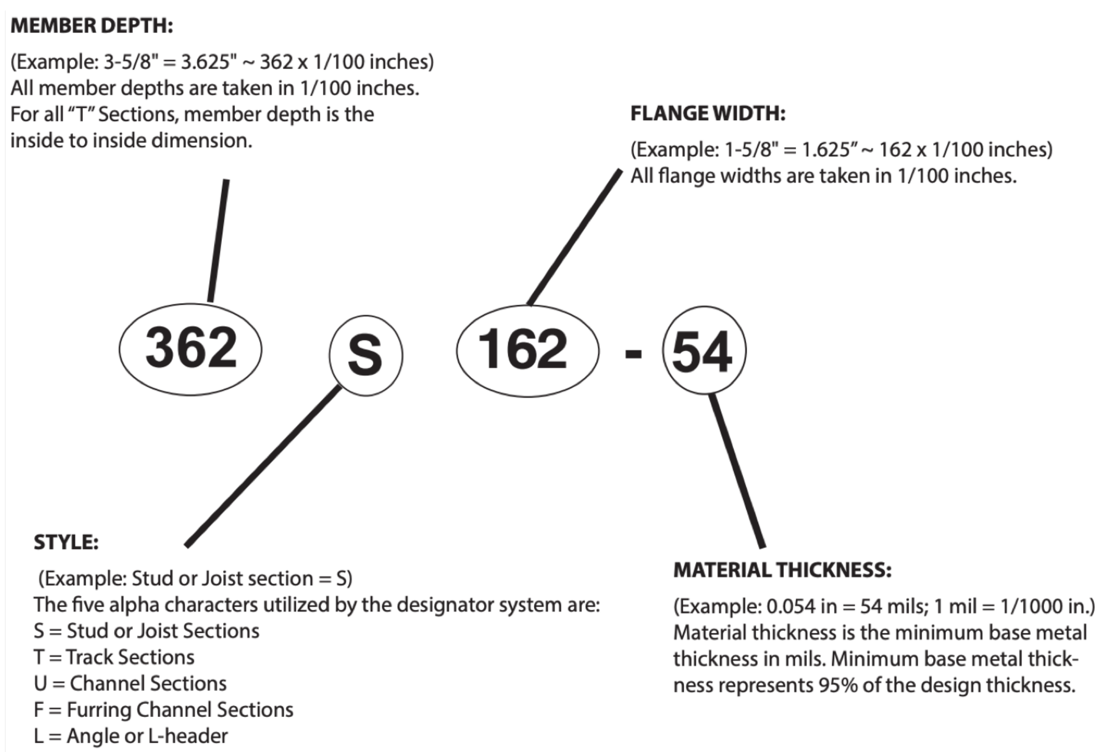

Fig. 6. CFS member naming convention per SSMA [21].