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Updated seismic fragility functions for cold-formed steel framed shear walls per FEMA P-58 methodology

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ABSTRACT

Performance-based earthquake engineering (PBEE) provides a robust alternative to traditional earthquake design. PBEE enables engineers to estimate expected damage, repair costs, and economic losses due to downtime for a candidate design, potentially leading to novel new designs or retrofit solutions. With the increasing application of light-frame structural systems, such as cold-formed steel (CFS) panels, in residential and commercial construction, it is necessary to develop and employ fragility functions for these systems to enable PBEE. In this regard, a set of fragility functions was previously developed by researchers based on a series of monotonic and cyclic tests for CFS framed shear walls with wood structural panel sheathing, flat strap X-bracing, and steel sheet sheathing. Recently, the senior author has led in the development of a large database of CFS framed shear wall tests, including 617 monotonic and cyclic tests conducted in the last 20 years from 25 primary sources. Based on the wider database, the fragility functions for CFS framed shear walls are re-evaluated. The developed fragility functions provide updated knowledge for application of PBEE per the FEMA P-58 methodology and are recommended for future use.

1. Introduction

1.1. Cold-formed steel light-framed structural systems

Cold-formed steel (CFS) is used extensively in the building construction industry for nonstructural partitions and ceilings, exterior curtain walls and façade support, as well as load-bearing structures including lateral force-resisting systems. With recent advances in the understanding of CFS framing, the application of CFS is expected to expand into more complex structural systems. CFS framing members are typically thin, open, and singly symmetric, and thus, their design must be specialized to account for unique behavior and limit states [1].

A bare CFS framing panel consists of lipped channel studs and plain channel tracks attached together with self-drilling fasteners and has minimal lateral resistance. In general, there are two approaches to providing lateral resistance to CFS framing panels: strap-bracing and sheathing. In strap-braced walls, diagonal flat straps are connected to one or both faces of the wall and provide lateral resistance through truss action, and strength is limited by the tensile strength of the strap, with properly designed connections. In walls with sheathed panels, the framing undergoes shear deformation while the sheathing panels rotate. This creates a differential demand at the fasteners, providing the primary mechanism for resisting lateral loads. Currently, common CFS framed lateral (seismic) force-resisting systems can be classified into 5 categories (see Fig. 1): (1) shear walls with wood structural panels, (2) shear walls with steel sheet sheathing, (3) strap-braced walls (diagonal, tension-braced walls), (4) special bolted moment frames, and (5) proprietary products not specifically recognized by AISI S400-15 [2], including shear walls with steel sheet adhered to other sheathing materials such as gypsum board.

1.2. Seismic performance assessment of buildings based on FEMA P-58

In conventional seismic design, design criteria are based on strength and serviceability limits of structural components and systems. In this process, the uncertainties and variabilities in seismic demand and capacity of structural components are aggregated into probabilistic load and resistance factors. However, these design criteria do not specify the expected level of damage. Recent natural disasters have shown that even structures compliant with building codes can undergo significant damage, including human and financial losses [7]. Traditional seismic design codes attempt to ensure life safety, but for certain buildings, a

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lower damage can be desired (or required). For example, critical infrastructure such as a hospital may need to be kept operable after earthquakes, or school buildings (or other public buildings) may be required to stand operable to be used as post-event shelters. In this regard, the U. S. Federal Emergency Management Agency (FEMA) has focused efforts on developing and promoting performance-based seismic design in the past two decades. A key result of these efforts is the FEMA P-58, Seismic Performance Assessment of Buildings, Methodology and Implementation, which provides the necessary information and recommended procedures for developing basic building information, response quantities, fragilities, consequence data used as inputs to the methodology, and guidelines on how to implement the methodology.

1.3. Previous work on the fragility of CFS framed shear walls

As the application of CFS framed panels is increasing in the construction industry, it is essential to study the fragility of different types of CFS framed shear walls. The fragility functions of CFS framed panels can be used in performance-based earthquake engineering. CFS framed structures show significant nonlinearity at connections, in the framing, and in the behavior of sheathing materials, specifically under lateral loads. As a result, understanding and estimation of fundamental behaviors such as lateral capacity is acquired using experimental testing [8]. In this regard, monotonic and cyclic tests have been conducted on CFS framed shear walls with different sheathing materials and structural characteristics [3,4,15–24,5,6,9–14]. Based on a portion of these tests, a set of fragility functions was previously developed [25] for CFS framed shear walls with wood structural panel (WSP) sheathing, flat strap X-bracing, and steel sheet sheathing using story drift ratio as the engineering demand parameter in accordance with the reporting

Table 1
Current values of median and log standard deviation of fragility functions for CFS framed shear walls, demand parameter: story drift ratio (%).

Type of shear wall	Damage state	Median	Log Standard Deviation
WSP sheathed	DS1	0.40	0.40
	DS2	2.26	0.30
	DS3	2.67	0.25
Flat strap X-braced	DS1	1.39	0.25
	DS2	1.79	0.25
Steel sheet sheathed	DS1	1.90	0.25
	DS2	2.53	0.25

Fig. 1. Different types of CFS framed shear walls: (a) shear wall with wood structural panels (image source: [3]); (b) shear walls with steel sheet sheathing (image source: [4]); (c) strap-braced walls (image source: [5]); (d) special bolted moment frames (image source: https://csengineermag.com/); (e) an example of a proprietary shear wall with wood structural panel on the front side and gypsum board on the back side (image source: [6]).

requirements for the U.S. FEMA P-58 project [26]. Table 1 lists the parameters of these fragility functions.

1.4. Cold-formed steel framed shear wall database

Recently, the senior author of this study has led in the development of a large database of CFS framed shear wall tests, including 617 monotonic and cyclic tests conducted in the last 20 years from 25 primary sources [8]. These tests support the CFS framed shear wall provisions provided in the North American Standard for Cold-Formed Steel Structural Framing (AISI S240-15) [27], the North American Standard for Seismic Design of Cold-Formed Steel Structural Systems (AISI S400-15) [2], and the U.S. Seismic Evaluation and Retrofit of Existing Buildings (ASCE41-17) [28]. Using the database, a set of fragility functions for CFS framed shear walls with WSP sheathing, flat strap X-bracing, and steel sheet sheathing is developed based on the wider database. The fragility functions for walls with WSP sheathing are developed in four groups based on the type of wood used for sheathing: Oriented Strand Board (OSB), Douglas Fir Plywood (DFP), Canadian Softwood Plywood (CSP), and all types inclusive. While panel grade is limited to 15/32″ Structural 1 (corresponding to DFP) and 7/16″ OSB in the US and Mexico per AISI S400-15, a wider range is allowed in Canada, including all three panel types. The functions for walls with flat strap X-bracing are developed in three groups based on the steel grade of straps: 230 MPa (33 ksi), 345 MPa (50 ksi), and all inclusive. In the process of developing these fragility functions, walls with supplemental gypsum wallboard and walls with aspect ratios of 4:1 or larger are excluded.

2. Fragility functions of CFS framed walls with WSP sheathing

2.1. Background

Shear walls with WSP sheathing are one of the most common seismic force-resisting systems used with CFS framing. According to a large body of research, the primary energy dissipation mechanism for these walls consists of deformation in the member-to-sheathing connections and the wood panels [1]. AISI S400-15 [2] provides data on shear strength of Structural 1 and OSB sheathing with different fastener spacing and thickness of studs and tracks. The general layout of walls with WSP sheathing is illustrated in Fig. 2.

CFS framed walls with WSP sheathing can undergo different failure modes. The preferred failure mode per design by AISI S400-15 [2] is tilting and bearing damage at the sheathing to stud/track fastener locations. If edge distances are insufficient, fastener tear out at the panel edges will occur, resulting in more dramatic losses in load carrying capacity, as shown in Fig. 3a. With proper edge distance, tilting and bearing of the fasteners eventually evolves to fastener pull-through at large lateral drift, as shown in Fig. 3b. Other limit states such as screw shear, screw withdrawal, stud buckling, track bending, and hold-down

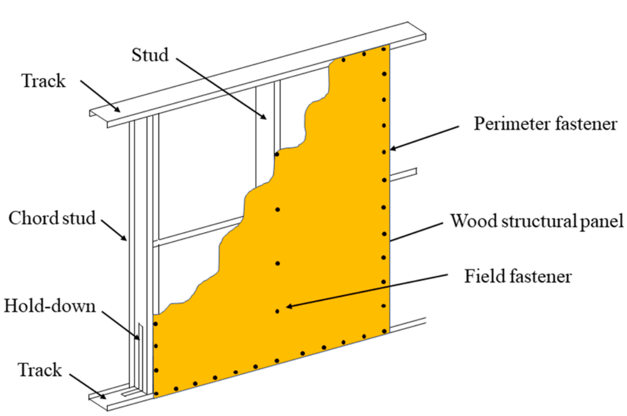

Fig. 2. CFS shear wall with WSP sheathing.

failure are intended to not occur either due to prescriptive limits established from testing or capacity protection principles (over-strength design) as detailed per AISI S400-15.

2.2. Definition of damage states

The engineering demand parameter used to define damage states is story drift ratio (%), which is defined as the ratio of the amount of horizontal drift of the wall specimen to the height of the wall. DS1 is defined as the story drift ratio corresponding to the end of the elastic behavior of the wall specimen, which is at 40% of peak load. The observed failure modes at DS1 include minor bearing damage at fasteners, minor fastener tilting, screw head pull-through of sheathing, and cosmetic damage at wall face or screw heads. DS2 is defined as the story drift ratio corresponding to the peak load in the wall specimen (maximum capacity of wall). At DS2, the wall experiences a significant amount of permanent damage and a significant degradation in initial stiffness, if unloaded. The failure modes include noticeable bearing/tilting damage at fasteners, fastener pull-through at multiple locations, fastener tear-out at sheathing panel edges at multiple locations, and permanent rotation of sheathing. DS3 is defined as the story drift ratio corresponding to the 80% of post-peak load in the wall specimen. At DS3, the wall specimen undergoes unrecoverable damage, including deformations associated with DS2 in addition to other failure modes such as detachment of sheathing from studs and tracks for significant portion of sheathing perimeter, wood bearing at panel to fastener interface, local elastic buckling of studs at fastener locations, global buckling of studs/tracks, shear failure of fasteners, and fastener withdrawal from studs. The damage states defined above for updated fragility functions are the same as those used in the previous study with some improvement on their description. Accordingly, the repair procedures for all three damage states are similar to those reported in the previous study.

For cyclic tests, the average absolute values from both negative and positive cycles are taken. Fig. 4 depicts a generic example of how displacement values corresponding to damage states can be obtained from data for a wall specimen (data from Test 3C of [3]).

2.3. Fragility functions

The procedure to develop fragility functions is in accordance with the fragility reporting requirements of P-58 [26]. To obtain the mean and dispersion parameters of the fragility functions, Method A of [26] has been used where all the specimens failed at observed values of EDP. In this paper, the fragility functions are idealized using the lognormal distribution. Before analyzing the data, outliers were excluded using the Peirce's Criterion [29]. The developed functions were then tested for goodness-of-fit at 5% significance level using the Lilliefors test [30]. Fig. 5 illustrates the development of the fragility function for DS3 of walls with WSP sheathing.

The fragility functions for CFS framed walls with WSP sheathing are developed regardless of the spacing of the sheathing fasteners and in four groups based on the type of wood used for the sheathing, i.e., OSB, DFP, CSP, and all inclusive. These functions are useful for damage assessment when the fastener spacing is unknown and for different types of wood sheathing (known or unknown). To develop these fragility functions, monotonic and cyclic tests from the database are used. The fragility functions for walls with WSP sheathing are shown in Figs. 6-9. The current fragility curves (based on 217 tests) are also plotted for comparison with the updated curves developed in this study from the wider database of tests

The values for the median and logarithmic standard deviation of the story drift ratio for the updated fragility functions for walls with WSP sheathing are listed in Table 2. It is noteworthy to highlight that DFP is close in performance to the Structural 1 wood panels according to the US specifications in AISI S400-15 [2].

(a) (b)

Fig. 3. Examples of failure modes of walls with WSP sheathing: (a) sheathing tear-out at corners; (b) wood bearing failure (image). Source: [3]

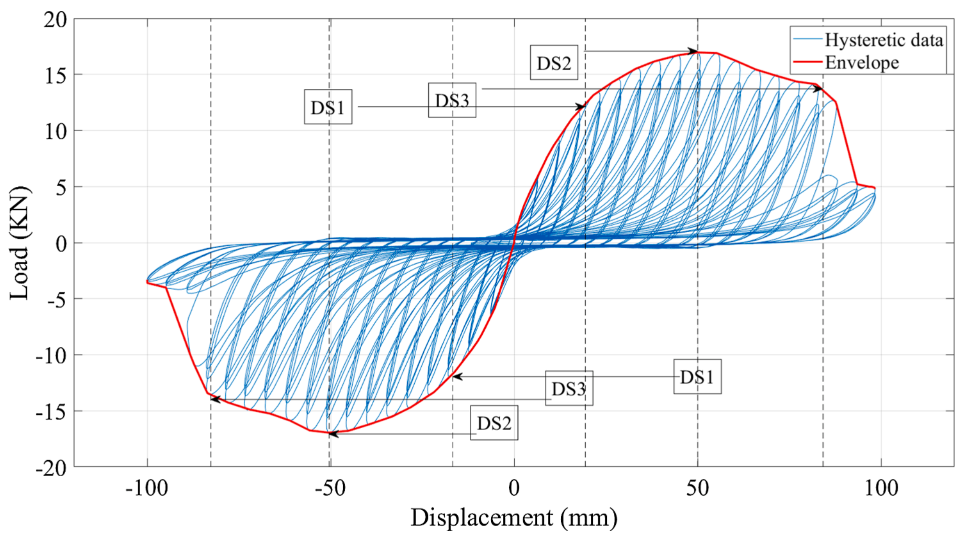

Fig. 4. Example of obtaining the displacements corresponding to the damage states for a test specimen.

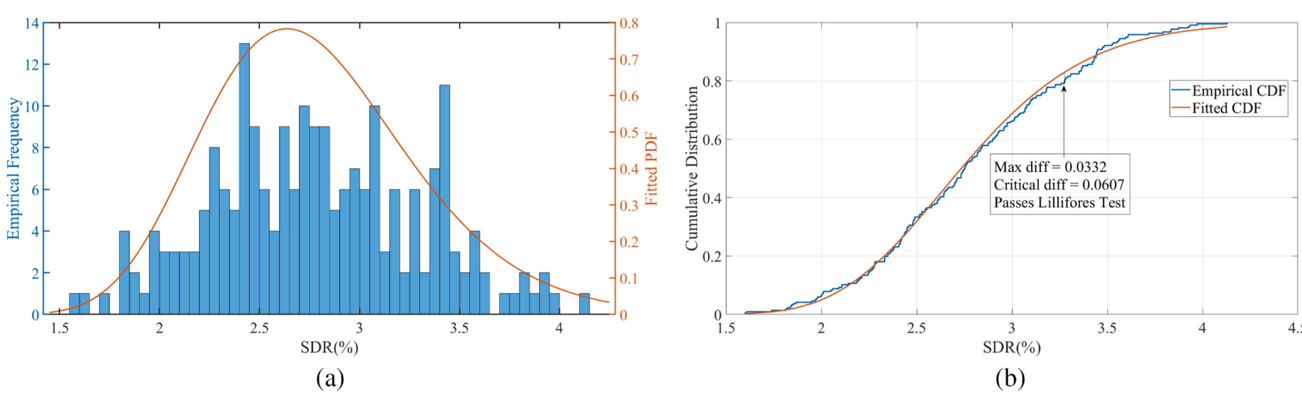

Fig. 5. Probabilistic (a) and cumulative distribution (b) corresponding to DS3 for all WSP sheathed walls.

3. Fragility functions of CFS framed walls with steel sheet sheathing

3.1. Background

Application of steel sheets with different thicknesses as sheathing over CFS framing provides a high shear resistance while being incombustible, to compare with wood panel sheathing. AISI S400-15 [2] provides the shear strength of CFS framed walls with 0.018-inch (0.46-mm) to 0.033-inch (0.76-mm) steel sheet sheathing and different screw spacing and framing thickness. This is an area of active research, and further studies are being conducted to expand knowledge and data to thicker steel sheets and framing members. The results of these studies will lead to higher strength steel sheet sheathed shear walls that can be used in mid-rise buildings [1]. The primary energy dissipation mechanisms in these walls are through the structural member-to-sheathing connections and yielding of the steel sheet. The general layout of CFS framed walls with steel sheet sheathing is illustrated in Fig. 10.

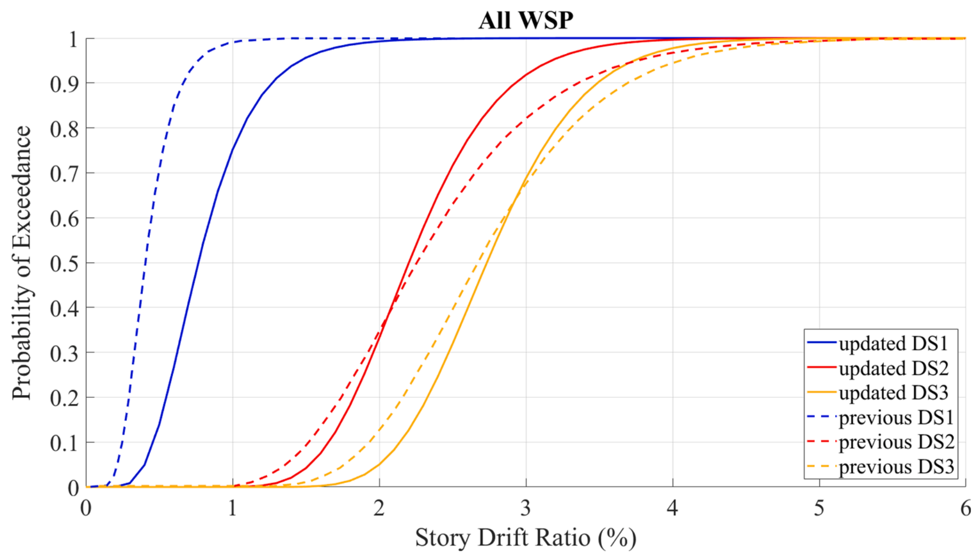

Fig. 6. Updated fragility curves for CFS framed walls with all WSP sheathing.

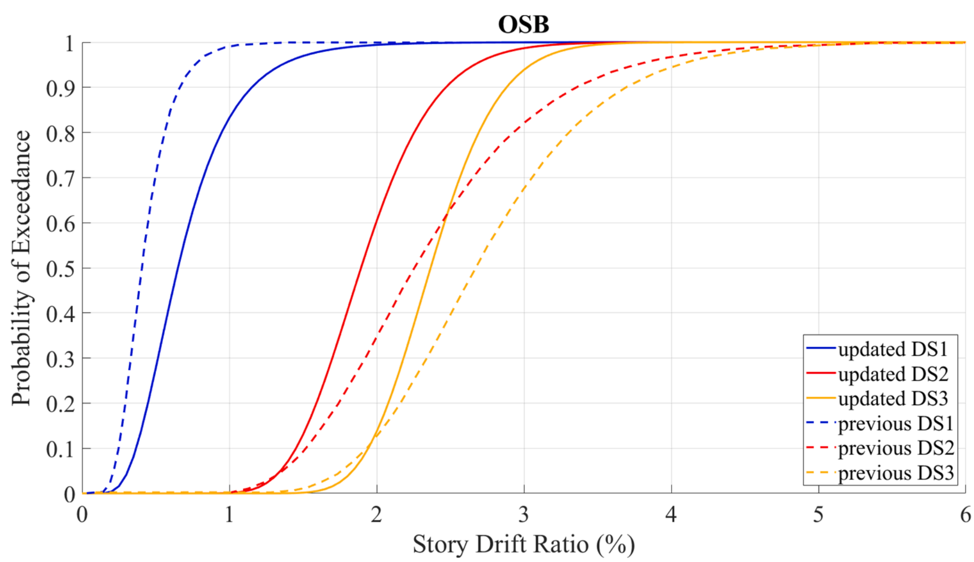

Fig. 7. Updated fragility curves for CFS framed walls with OSB sheathing.

CFS framed walls with steel sheet sheathing can undergo different failure modes. The preferred failure mode per design by AISI S400-15 [2] is yielding in the steel sheet followed by bearing damage at the sheathing to stud/track fastener locations. Buckling in the thin steel sheets is a prominent feature of the response even at low deformation (Fig. 11a). Although ideally the stud and track do not deform, many of the tests included in the AISI S400-15 standard (and the database) include some bending deformation in the attached stud flange and track, which can cause significant deformations at post-peak as shown in Fig. 11b. As the bearing damage progresses at the fasteners, eventually edge tear-out at the panel edges will occur, resulting in more dramatic losses in load carrying capacity and redistribution of load within the wall. Other properly designed failure modes, such as screw shear, screw withdrawal, chord stud buckling, and hold-down failure, are intended to not occur, either due to prescriptive limits established from testing or capacity protection principles (over-strength design) as detailed in AISI S400-15.

3.2. *Definition of damage states*

Similar to walls with WSP sheathing, the engineering demand parameter used to define damage states is story drift ratio (%). DS1 is defined as the drift ratio corresponding to the end of the elastic behavior of the wall specimen, which is at 40% of peak load. This damage state is introduced based on the wider database of tests to address minor damages. The observed failure modes at DS1 include visual buckling of steel sheet, small wavelength buckling along perimeter, minor bearing damage at perimeter fasteners, minor fastener tilting, and minor screw head pull-through of sheathing. The repair procedure is refastening the structural wall panels. DS2 is defined as the story drift ratio corresponding to the peak load in the wall specimen. At DS2, the wall experiences a significant amount of permanent damage and a significant degradation in initial stiffness, if unloaded. The failure modes include significant buckling of steel sheet, bearing damage at multiple fasteners, fastener pull-through at multiple locations, fastener pull-out from framing members, and block shear rupture of steel sheathing at panel edges. The repair for DS2 is the same as those corresponding to damage state 1 from the previous study. DS3 is defined as the story drift ratio

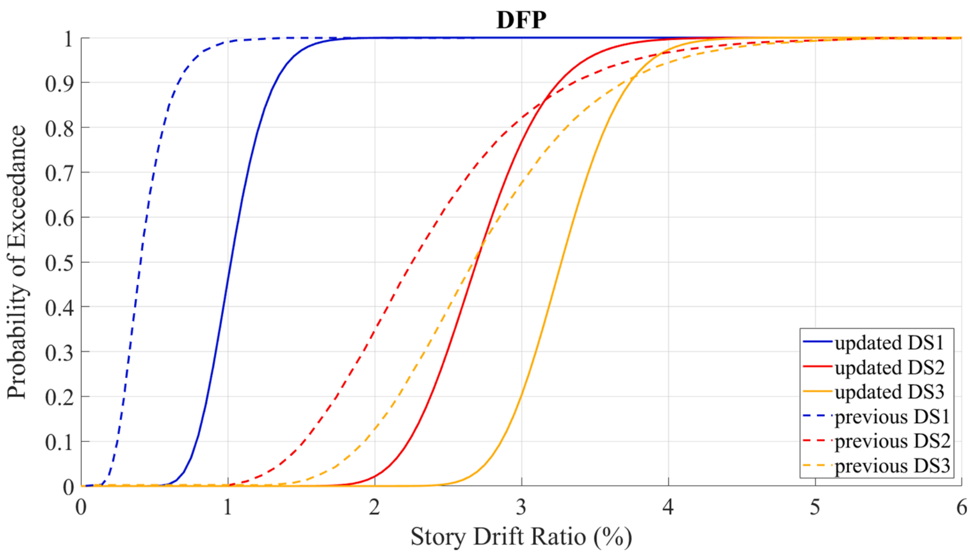

Fig. 8. Updated fragility curves for CFS framed walls with DFP sheathing.

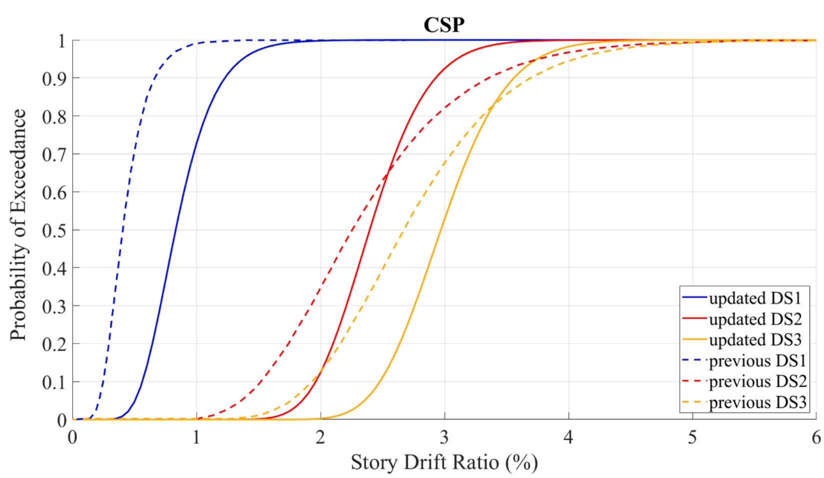

Fig. 9. Updated fragility curves for CFS framed walls with CSP sheathing.

Table 2
Updated parameters of the fragility functions for CFS framed walls with WSP sheathing, demand parameter: story drift ratio (%).

Wood Type	Damage State	Median	Log SD	# Samples	Lilliefors Test*
All	DS1	0.77	0.39	224	Fails**
	DS2	2.20	0.22	224	Fails**
	DS3	2.73	0.19	219	Passes
OSB	DS1	0.65	0.45	95	Passes
	DS2	1.89	0.21	95	Passes
	DS3	2.37	0.15	92	Passes
DFP***	DS1	1.02	0.20	26	Passes
	DS2	2.69	0.15	26	Passes
	DS3	3.27	0.10	26	Passes
CSP/ Plywood	DS1	0.83	0.30	103	Passes
	DS2	2.40	0.16	103	Passes
	DS3	2.97	0.14	101	Passes

* At 5% significance level.
** Passes the test at 10% significance level.
*** Equivalent to Structural 1 wood panels per AISI S400-15 provisions for the US.

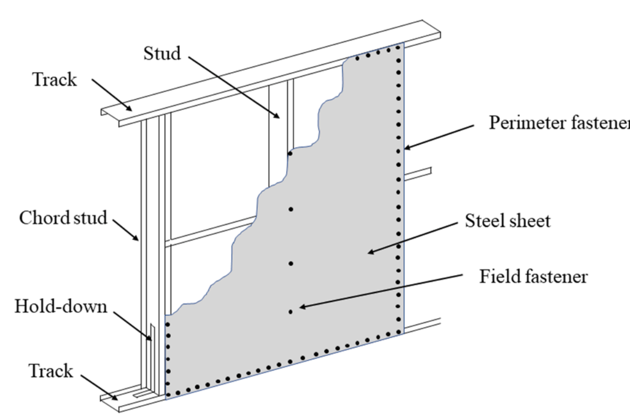

Fig. 10. CFS framed shear wall with steel sheet sheathing.

corresponding to the 80% of post-peak load in the wall specimen. At DS3, the wall specimen undergoes unrecoverable damages, including fastener pull-through, fastener bearing/edge tear-out, detachment of sheathing from studs and track for a significant portion of sheathing perimeter, and bending of stud and track flanges. The repair for DS3 is

(a) (b)

Fig. 11. Examples of failure modes of walls with steel sheet sheathing: (a) buckling of steel sheet; (b) twisting of stud (image).
Source: [11]

the same as those for damage state 2 of the previous study.

3.3. *Fragility functions*

The fragility functions for CFS framed walls with steel sheet sheathing are developed independent of the spacing of the sheathing fasteners. To develop these fragility functions, monotonic and cyclic tests from the database are used. The updated fragility curves for walls with steel sheet sheathing, based on 101 tests, are shown in Fig. 12. The current fragility curves, based on 6 tests, are also plotted for comparison.

The values for the median and logarithmic standard deviation of the story drift ratio for the updated fragility functions for walls with steel sheet sheathing are listed in Table 3.

Table 3
Updated parameters of fragility functions for CFS framed walls with steel sheet sheathing, demand parameter: story drift ratio (%).

Damage State	Median	Log SD	# Samples	Lilliefors Test*
DS1	0.56	0.60	101	Passes
DS2	1.60	0.30	101	Passes
DS3	2.46	0.22	84	Passes

* at 5% significance level.

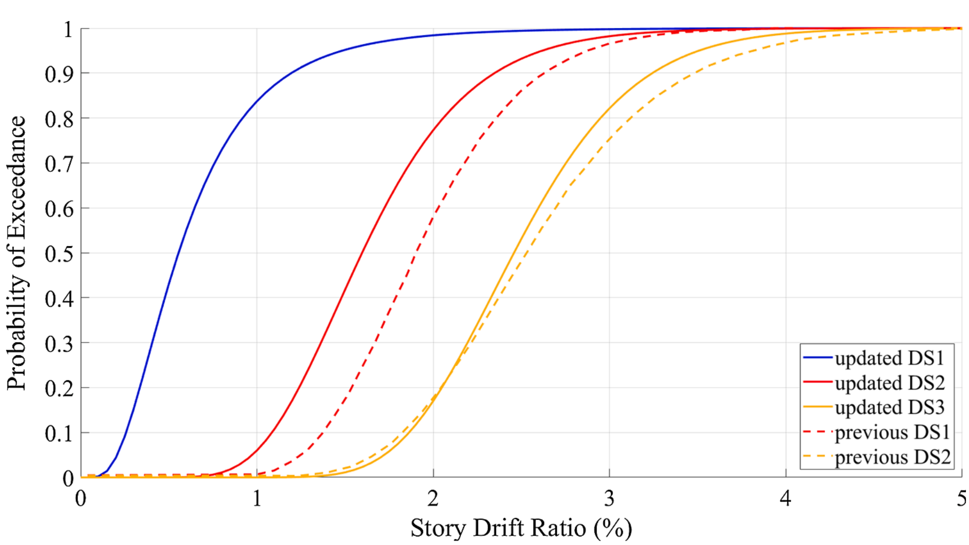

Fig. 12. Updated fragility curves for CFS framed walls with steel sheet sheathing.

4. Fragility functions of CFS framed walls with flat strap X-bracing

4.1. Background

CFS framed walls with flat strap X-Bracing are commonly used in CFS light frame buildings. The energy dissipation mechanism in these walls is yielding of the straps. As the expected strength of straps is extensively studied, the capacity-based design for walls with flat straps is relatively straightforward. The general layout of a wall with flat strap bracing is illustrated in Fig. 13.

To ensure capacity protection, e.g., net section fracture in the tension straps not occurring prior to yielding of the strap gross cross-section, specific limitations have been placed on the strap connections. The limitations for capacity protection were initially introduced in AISI S213-07 [31]; however, the design strength of certain elements and connections were allowed to be taken as the nominal strength. In this regard, AISI S400-15 [2] places stringent provisions regarding capacity protection of elements and connections in CFS framed shear walls due to observations in testing regarding a number of undesirable limit states including: fracture of the flat strap in the net section at the stud to strap connection (Fig. 14a), anchorage failures (Fig. 14b), and local bending of the chord stud and/or track at the strap to stud connection. The most noticeable feature of tested specimens with flat strap X-bracing is the unloading and buckling of the compression strap; however, this has little influence on the behavior. Yielding of the flat strap is visually difficult to observe when the wall is deformed, but a plumb wall which has yielded will have slack straps due to elongation—only upon drifting the wall back out to previous deformations does the strap again become taught.

4.2. Definition of damage states

Similar to walls with WSP and steel sheet sheathing, the engineering demand parameter used to define damage states is story drift ratio (%). DS1 is defined as the story drift ratio corresponding to the end of the elastic behavior of the wall specimen, which is at 40% of peak load. This damage state is introduced based on the wider database of tests to address minor damages. The observed failure modes at DS1 include visual buckling of flat strap in compression, minor elongation of tension strap, and minor bearing damage at strap to stud fasteners, if screwed. The repair for DS1 consists of removal of building contents and refastening the structural wall panels. DS2 is defined as the story drift ratio corresponding to the peak load in the wall specimen. At DS2, the wall experiences a significant amount of permanent damage and a significant degradation in initial stiffness, if unloaded. The failure modes include buckling of flat strap in compression, local buckling of the chord stud, noticeable elongation of tension strap, and bearing damage at strap to stud fasteners, if screwed. The repair for DS2 is the same as those for damage state 1 from the previous study. DS3 is defined as the story drift ratio corresponding to the 80% of post-peak load in the wall specimen. At DS3, the wall specimen undergoes unrecoverable damages, including necking and yielding in tension strap accompanied with large elongation. For older walls without proper capacity protection (i.e., all walls constructed before 2007 and potentially those constructed between 2007 and 2015), undesirable failure modes can occur, such as strap net section failure at connection, anchorage failure, and bending of stud and track flanges. The repair for DS3 is the same as those for damage state 2 of the previous study.

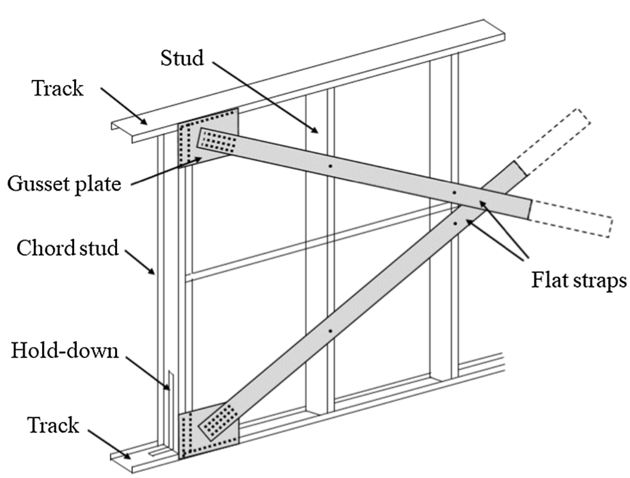

Fig. 13. CFS framed shear wall with flat strap X-bracing (fastened detail shown, welded gusset plates and strap is also common).

4.3. Fragility functions

The fragility functions for CFS framed walls with flat strap X-bracing are developed in three groups based on the grade of steel used for the straps: 230 MPa (33 ksi), 345 MPa (50 ksi), and all inclusive. The developed fragility functions are useful for damage assessment whether the strap material grade is known or unknown. To develop these fragility functions, monotonic and cyclic tests from the database are used. The updated fragility curves for walls with one-sided flat strap X-bracing are shown in Figs. 15–17. Initially, the developed fragility functions for DS2 and DS3 in all three cases were crossing. These functions were corrected based on the procedures provided in Section 3.4 of [26]. Additional work with the data to further limit the selected specimens based on whether or not the details meet current AISI S400-15 requirements [2] is completed in the following section to further separate the results.

The values for the median and logarithmic standard deviation of the story drift ratio for the updated fragility functions for walls with flat strap X-bracing are listed in Table 4.

5. Fragility functions based on shear wall tests that meet AISI S400-15 specifications

AISI S400-15 [2] implements capacity protection for all its seismic force resisting systems. The detailing and member design requirements are intended to ensure the designated energy dissipating mechanism occurs in the systems. The fragility functions introduced in Sections 2, 3, and 4 are developed based on all available shear wall data. In many cases, tested shear walls do not meet the current requirements of AISI S400-15 and as a result may provide a biased estimation of CFS framed shear walls designed to contemporary standards. To examine this effect, the fragility functions for walls with WSP sheathing, steel sheet sheathing, and flat strap X-bracing are regenerated for only those walls that meet the detailing and capacity protection requirements per AISI S400-15.

5.1. CFS framed walls with WSP sheathing per AISI S400-15

CFS framed walls sheathed with WSP are expected to withstand seismic loads through deformation in the connection between the wood panel sheathing and the CFS members. To ensure capacity protection in these walls, two types of requirements have to be met per AISI S400-15 [2]. First, the chord studs shall be designed according to capacity design principles to resist the maximum anticipated seismic loads, and second, prescriptive system detailing requirements have to be met. To ensure capacity protection, the nominal axial strength of chord studs has to be greater than their respective expected strength (see Fig. 18).

The expected strength (probable resistance) of CFS framed shear walls with WSP sheathing can be calculated according to Section E1.3 of AISI S400-15 [2], as follows: the nominal shear strength per unit length for seismic and other in-plane loads, v_n, can be obtained from Table E1.3-1 of AISI S400-15 [2]. For Type 1 shear walls, defined as “Wall designed to resist in-plane lateral forces that is fully sheathed and

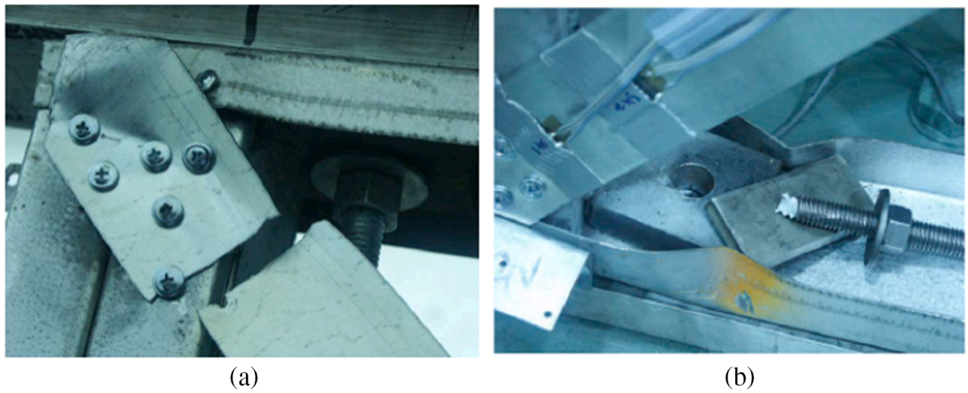

Fig. 14. Potential undesirable failure modes in flat strapped shear walls: (a) improperly detailed connection leading to net section failure; (b) improperly sized rod leading to anchorage failure (image).
Source: [23]

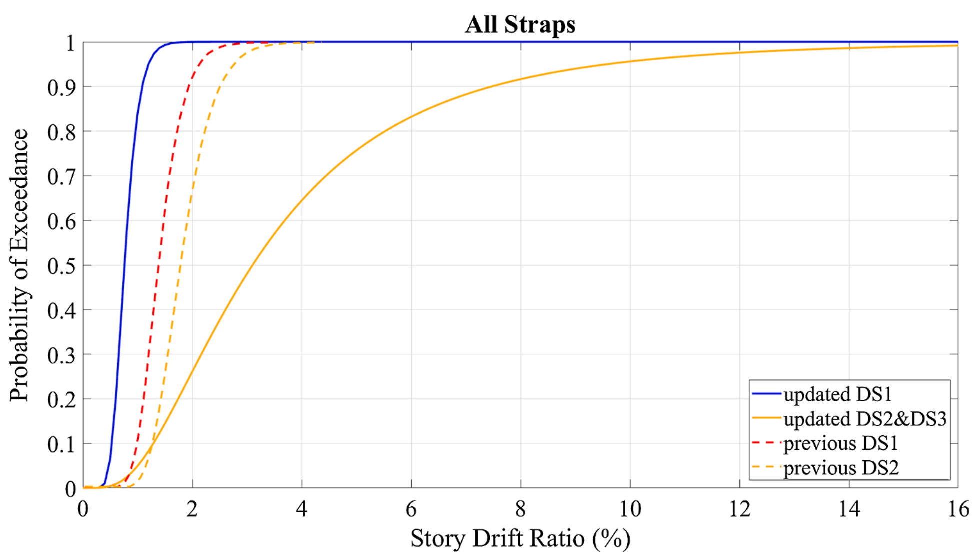

Fig. 15. Updated fragility curves for CFS framed walls with X-bracing (all steel grades inclusive).

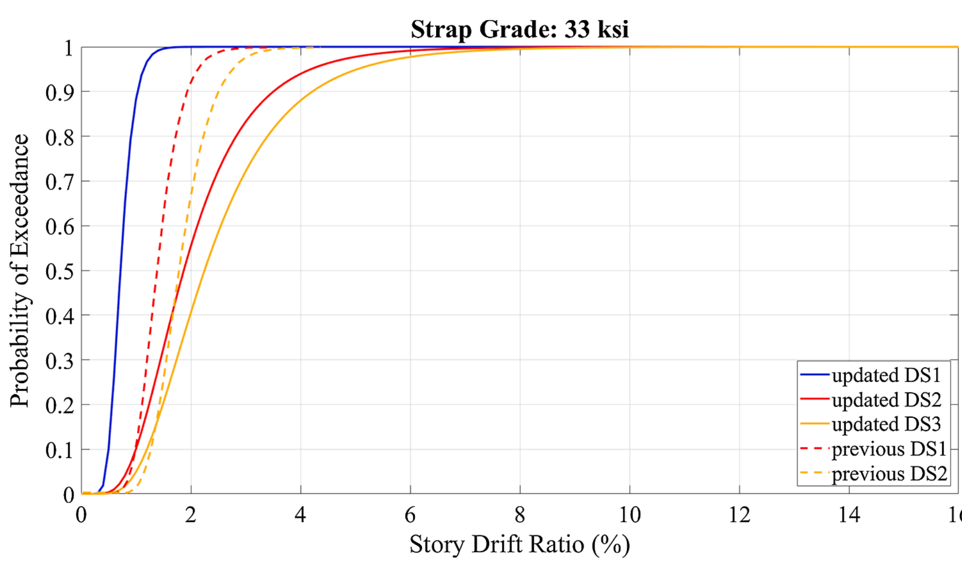

Fig. 16. Updated fragility curves for CFS framed walls with X-bracing, steel grade: 230 MPa (33 ksi).

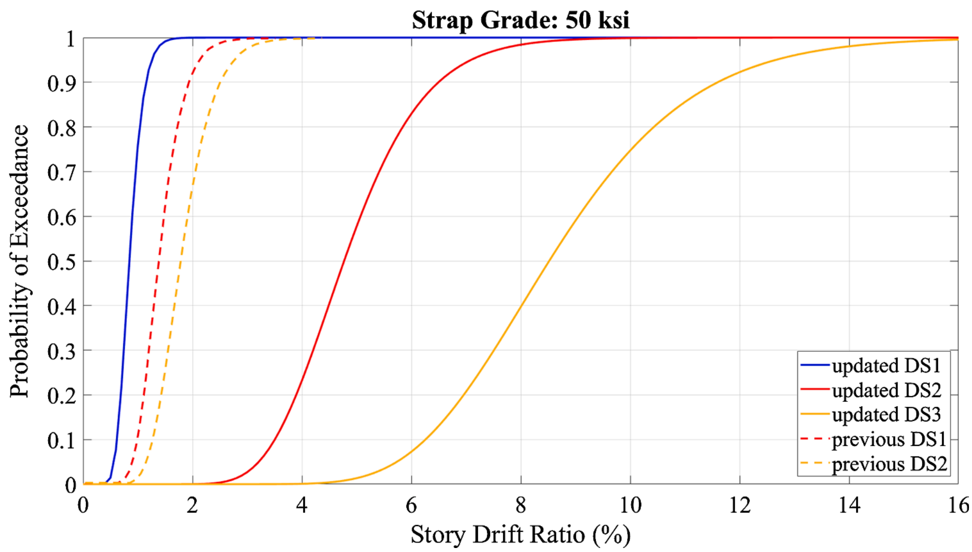

Fig. 17. Updated fragility curves for CFS framed walls with X-bracing, steel grade: 345 MPa (50 ksi).

Table 4
Updated parameters of fragility functions for CFS framed walls with flat strap X-bracing, demand parameter: story drift ratio (%).

Steel Grade	Damage State	Median	Log SD	# Samples	Lilliefors Test *
All	DS1	0.76	0.28	85	Passes
	DS2	3.21	0.67	85	Fails **
	DS3	3.21	0.67	39	Passes
33 ksi	DS1	0.71	0.28	55	Passes
	DS2	1.95	0.50	55	Passes
	DS3	2.29	0.50	28	Passes
50 ksi	DS1	0.85	0.23	30	Fails **
	DS2	4.97	0.25	30	Fails **
	DS3	7.55	0.25	11	Passes

* at 5% significance level.
** passes the test at 10% significance level.

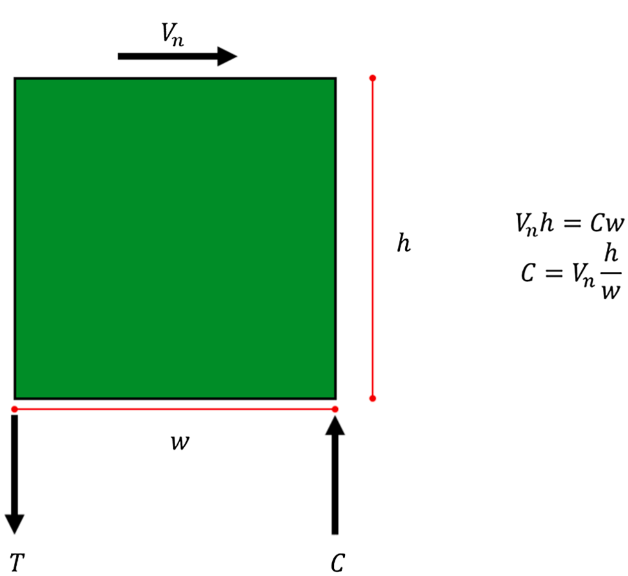

Fig. 18. Force diagram of a CFS framed wall under seismic loading.

that is provided with hold-downs and anchorage at each end of the wall segment" [2, p. 6], the nominal strength for shear, V_n, is calculated based on the height-to-length-aspect ratio (h/w):

$$V_n = \begin{cases} v_n w & h/w \le 2 \\ v_n w(2h/w) & 2 < h/w \le 4 \end{cases} \quad (1)$$

where h is the height of the wall, and w is the length of the wall.

Equation (1) gives the nominal strength for shear walls sheathed with WSP on one side of the wall. If the wall has sheathing with the same material and fastener spacing on both sides, the nominal shear strength is obtained by adding the strength from the two opposite faces together. For walls with more than a single sheathing material or fastener configuration, the overall nominal strength shall be calculated per Section E1.3.1.1.3 of AISI S400-15 [2]. The expected strength, P_r, can be determined from the nominal strength, as follows:

$$P_r = \Omega_E V_n \frac{h}{w} \quad (2)$$

where Ω_E is the expected strength factor, which is 1.8 in the US and Mexico for walls sheathed with OSB or Structural I (equivalent to DFP) panels. In Canada, Ω_E shall be 1.33 for walls sheathed with DFP or OSB wood panels, and 1.45 for walls with CSP panels, according to Section E1.3.3 of AISI S400-15 [2]. CSP wood panels are not allowed in the US and Mexico to be used as structural panels.

The nominal axial strength, P_n, of chord studs can be obtained from Table III-2 of AISI D100-17, Cold-Formed Steel Design – Vol. 1 [32]. The nominal axial strength of chord studs has to be greater than their respective expected strength, $P_n > P_r$.

The dimensional and system requirements per AISI S400-15 [2] for CFS framed shear walls with WSP sheathing are specified in Sections E1.3 and E1.4 of AISI S400-15. The requirements considered in this study based on the available details from the shear wall test references are as follows: wall studs and tracks are ASTM A1003 Structural Grade 33 steel for members with a designation thickness of 33 and 43 mils and ASTM A1003 Structural Grade 50 steel for members with a designation thickness equal to or greater than 54 mils. In no case, shall the height-to-length aspect ratio (h/w) exceed 4:1. The length of Type I shear walls shall not be less than 24 in. (610 mm). The sheathing screw size should meet the minimum requirements listed in Table E1.3-1 of AISI S400-15 [2]. For studs (field studs and chord studs), the minimum flange width is 1–5/8 in. (41.3 mm), the minimum web depth is 3–1/2 in. (89 mm), the minimum edge stiffener is 3/8 in. (9.5 mm), and the maximum stud spacing is 24 in. (610 mm) on center. For tracks, the minimum flange width is 1–1/4 in. (31.8 mm), and the minimum web depth is 3–1/2 in. (89 mm). The sheathing fastener distance from panel edges is not less than 3/8 in. (9.5 mm) in the US and Mexico, and 1/2 in. (12.5 mm) in Canada. Field fasteners of panels are installed not longer than 12 in. (305 mm) on center. All sheathing edges should be attached to structural members or panel blocking. From 261 tests, 41 meet AISI S400-15 requirements for the US and Mexico, and 88 meet the requirements for Canada.

The updated fragility curves for CFS framed walls with WSP sheathing per AISI S400-15 specifications (separately for the US and Canada) are shown in Figs. 19-22 and compared with those developed regardless of the AISI S400-15 requirements.

The values for the median and logarithmic standard deviation of the story drift for the updated fragility functions for walls with different WSPs are listed in Tables 5 and 6 for US/Mexico-specific and Canada-specific AISI S400-15 requirements, respectively.

5.2. CFS framed walls with steel sheet sheathing per AISI S400-15

CFS framed walls sheathed with steel sheets are expected to withstand seismic loads through deformation in the connection between the steel sheet sheathing and the CFS members. The requirements for capacity protection are the same as those for walls with WSP sheathing. The expected strength factor, Ω_E, for these walls is 1.8 in the US and Mexico and 1.4 in Canada per Section E2.3.3 of AISI S400-15 [2]. An additional requirement for walls sheathed with steel sheet is stud blocking which is specified in Table E2.3-1 of AISI S400-15. From 179 tests, 92 meet AISI S400-15 requirements for the US and Mexico, and 50 meet the requirements for Canada.

The updated fragility curves for CFS framed walls with steel sheet sheathing per AISI S400-15 specifications (separately for the US and Canada) are shown in Fig. 23 and compared with those developed regardless of the AISI S-400 specifications.

The values for the median and logarithmic standard deviation of the story drift for the updated fragility functions for walls with steel sheet sheathing are listed in Tables 7 and 8 for US/Mexico-specific and Canada-specific AISI S400-15 requirements, respectively.

5.3. CFS framed walls with flat strap X-bracing per AISI S400-15

CFS framed walls braced with flat straps are expected to withstand seismic loads through tension yielding along the strap bracing. To ensure capacity protection, similar to walls sheathed with WSP or steel sheet, the chord studs shall be designed according to capacity design principles to resist the maximum anticipated seismic loads. The expected strength of CFS framed shear walls with flat strap X-bracing can be calculated according to Section E3.3 of AISI S400-15 [2]. The nominal shear strength, V_n, shall be determined using Equation (3):

$$V_n = T_n \frac{w}{\sqrt{h^2 + w^2}} \tag{3}$$

where $T_n = A_g F_y$ is the yielding nominal strength of the straps, A_g is the cross area of the strap, and F_y is the yield stress of the strap. If a wall has flat straps with the same material and fastener spacing on both sides, the nominal shear strength is obtained by adding the strength from the two opposite faces together. The expected strength, P_r, can be determined from the nominal strength, as follows:

$$P_r = R_y V_n \frac{h}{w} \tag{4}$$

where R_y is determined based on the value of the yield stress of the strap per Table A3.2-1 of AISI S400-15 [2]. The nominal axial strength of chord studs has to be greater than their respective expected strength, $P_n > P_r$.

The system requirements to ensure desirable failure modes are different from those for walls with WSP and steel sheet sheathing. The connection should be configured such that the straps meet the criteria of Equations (5) and (6):

$$(R_t F_u)/(R_y F_y) \geq 1.2 \tag{5}$$

$$R_t A_n F_u > R_y A_g F_y \tag{6}$$

where R_t is determined based on the value of the yield stress of the strap per Table A3.2-1 of AISI S400-15 [2], F_u is the tensile strength of the strap, and A_n is the net area of the flat strap.

Other requirements are as follows: wall studs, tracks, and flat straps are ASTM A1003 Structural Grade 33 steel for members with a designation thickness of 33 and 43 mils and ASTM A1003 Structural Grade 50 steel for members with a designation thickness equal to or greater than 54 mils. In no case, shall the height-to-length aspect ratio (h/w) exceed 4:1. The length of Type I shear walls shall not be less than 24 in. (610 mm). AISI S400-15 does not place specific requirements for Canada. From 85 tests, 49 meet these requirements.

The updated fragility curves for CFS framed walls with flat strap X-bracing per AISI S400-15 specifications are shown in Figs. 24–26 and

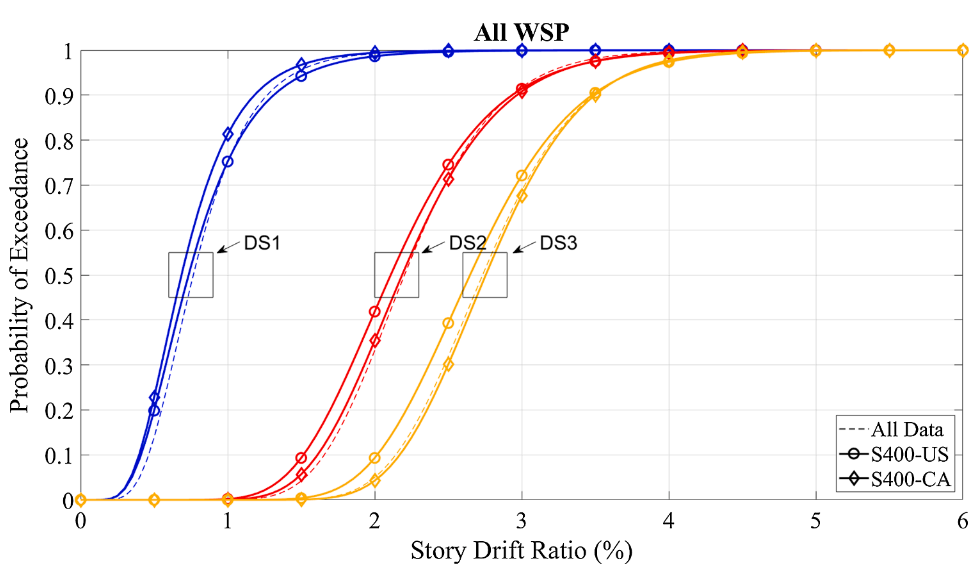

Fig. 19. Updated fragility curves for CFS framed walls with all WSP sheathing per AISI S400-15.

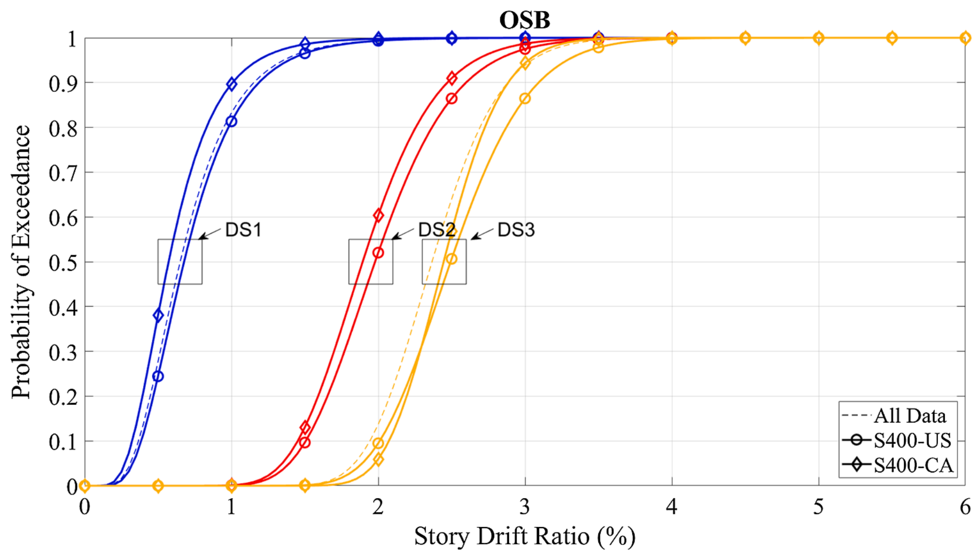

Fig. 20. Updated fragility curves for CFS framed walls with OSB sheathing per AISI S400-15.

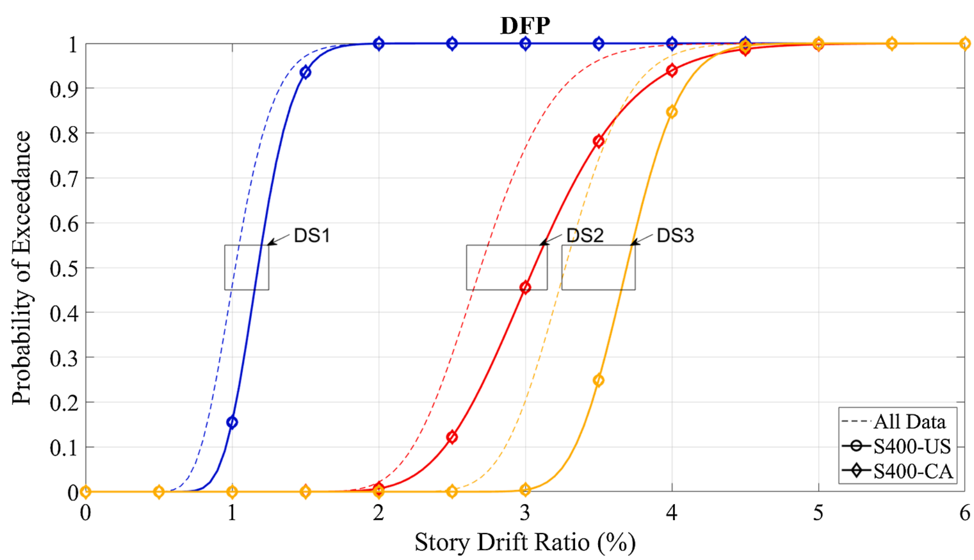

Fig. 21. Updated fragility curves for CFS framed walls with DFP sheathing per AISI S400-15.

compared with those developed regardless of AISI S-400-15 specifications.

The values for the median and logarithmic standard deviation of the story drift for the updated fragility functions for walls with flat strap X-bracing per AISI S400-15 are listed in Table 9.

6. Discussion

FEMA P-58 is the result of a decade of effort to promote performance-based earthquake engineering. The final products of this effort provide the methodology as well as basic building information, response quantities, fragilities, and data to be used as input to the methodology. To encourage practical implementation, FEMA provides a set of fragility and consequence data for common structural systems and building occupancies, along with a set of spreadsheet tools and the Performance Assessment Calculation Tool (PACT) to assist engineers in performing probabilistic computations and accumulation of losses. Since its release in 2012, the P-58 methodology has been used in practice and implemented into other engineering tools, such as the Resilient Design Initiative (REDi), the U.S. Resiliency Council (USRC) rating system, and the Seismic Performance Prediction Program (SP3) software. The fragility functions provided in this paper are intended to replace current fragility functions for damage assessment of buildings with CFS framed shear walls. These updated functions give an improved estimate for the probability that a given CFS framed shear wall will incur a given damage states at different story drift ratios.

The fragility functions developed based on the entire database include a limited number of wall specimens that did not meet all current AISI S400-15 criteria. Therefore, these functions provide a conservative estimate of damage for shear walls designed according to AISI S400-15 and are believed to provide a reasonable estimate of past designs. To provide a better estimation for new design walls, those wall specimens that did not meet the AISI S400-15 requirements were excluded, and the fragility functions were recreated. The second set of fragility functions provide a more accurate estimate for how new design CFS framed shear walls undergo different damage states under seismic loads.

As illustrated in Section 5, meeting the AISI S400-15 requirements has diverse effects on the fragility functions for different damage states. Those wall specimens that do not meet AISI S400-15 requirements are likely to undergo undesirable failure modes and consequent

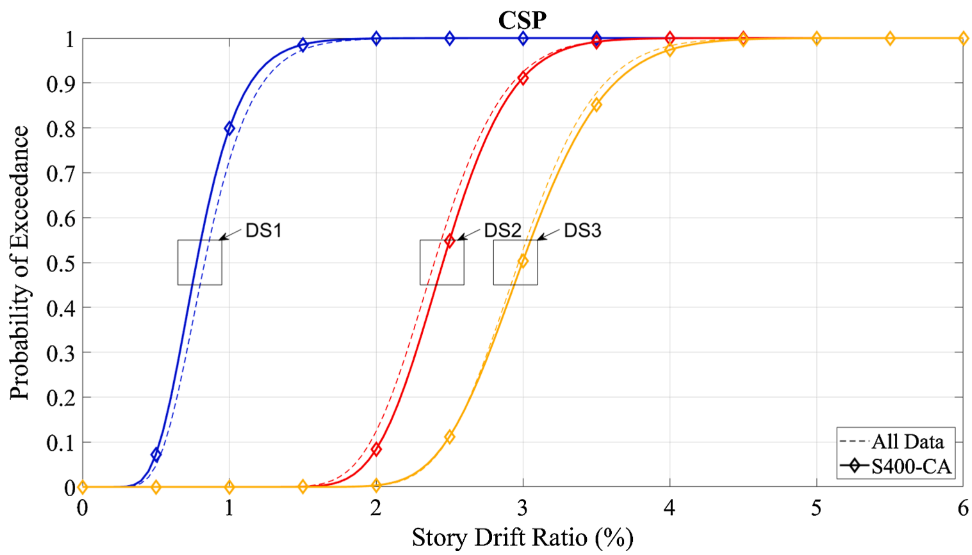

Fig. 22. Updated fragility curves for CFS framed walls with CSP sheathing per AISI S400-15.

Table 5
Updated parameters of the fragility functions for CFS framed walls with WSP sheathing per AISI S400-15 (US and Mexico), demand parameter: story drift ratio (%).

Wood Type	Damage State	Median	Log SD	# Samples	Lilliefors Test *
All	DS1	0.73	0.45	41	Passes
	DS2	2.11	0.26	41	Passes
	DS3	2.65	0.21	39	Passes
OSB	DS1	0.68	0.44	35	Passes
	DS2	1.98	0.21	35	Passes
	DS3	2.49	0.17	33	Passes
DFP **	DS1	1.18	0.16	6	Passes
	DS2	3.06	0.17	6	Passes
	DS3	3.69	0.08	6	Passes
CSP	DS1	–	–	–	–
	DS2	–	–	–	–
	DS3	–	–	–	–

* at 5% significance level.
** equivalent to Structural I wood panels per AISI S400-15 provisions for the US.

Table 6
Updated parameters of the fragility functions for CFS framed walls with WSP sheathing per AISI S400-15 (Canada), demand parameter: story drift ratio (%).

Wood Type	Damage State	Median	Log SD	# Samples	Lilliefors Test *
All	DS1	0.69	0.42	88	Passes
	DS2	2.19	0.24	88	Passes
	DS3	2.76	0.19	88	Passes
OSB	DS1	0.57	0.44	44	Passes
	DS2	1.89	0.21	44	Passes
	DS3	2.45	0.13	44	Fails **
DFP	DS1	1.18	0.16	6	Passes
	DS2	3.06	0.17	6	Passes
	DS3	3.69	0.08	6	Passes
CSP	DS1	0.78	0.30	38	Passes
	DS2	2.46	0.15	38	Passes
	DS3	3.00	0.15	38	Passes

* at 5% significance level.
** passes the test at 10% significance level.

deformations. However, the corresponding effects do not directly translate to a certain predictable change in the fragility. What AISI S400-15 designates is the type of damage (i.e., failure mode) and not the intensity of damage. In fact, some of the non-conforming wall specimens showed same or even a higher ductility (mainly from the chord stud bending capacity) than conforming wall specimens. This implies that the non-conforming walls do not necessarily undergo the defined damage states at lower drift ratio than conforming walls. Therefore, what the fragility functions that are developed based on AISI S400-15 conforming tests provide is a higher probabilistic confidence in the predicted level of damage associated with the defined damage states.

7. Application of updated fragility functions based on FEMA P-58 methodology

Two case studies are carried out to demonstrate the differences between the current fragility functions for CFS framed shear walls and updated functions provided in this study. In these case studies, the repair cost of shear walls is estimated using the current and the updated fragility functions based on the FEMA P-58 methodology.

7.1. Case study I: CFS-NEES

CFS-NEES is a completed NSF-funded project (NEESR-CR: Enabling Performance-Based Seismic Design of Multi-Story Cold-Formed Steel Structures) with the goal of generating the required knowledge to enhance seismic performance of CFS framed buildings. The project included Johns Hopkins University, Bucknell University, and Devco Engineering and was performed in collaboration with the American Iron and Steel Institute and other sponsors [33].

Two full-scale two-story CFS framed buildings were tested under 141 different excitations on the shake tables at the Structural Engineering and Earthquake Simulation Laboratory (SEESL) at the University of Buffalo (see Fig. 27). The structural system of the building consisted of an all-steel design with CFS framed gravity walls and CFS framed shear walls sheathed with OSB panels. Building dimensions were 23 ft. by 50 ft. in plan and 19 ft. in height, designed as a functioning office building with the hypothetical location at Orange County, California. The first building (Phase 1) only consisted of the lateral force-resisting system and the gravity system (no nonstructural components). The second building (Phase 2) had the same structural system while framed with nonstructural elements (exterior OSB panels, exterior weatherproofing, interior drywall, interior partition walls, and stairways).

The repair costs of shear walls for the phase 2 building (fully sheathed structural with interior nonstructural and exterior DensGlass) are calculated for three seismic scenarios based on the 1994 Northridge

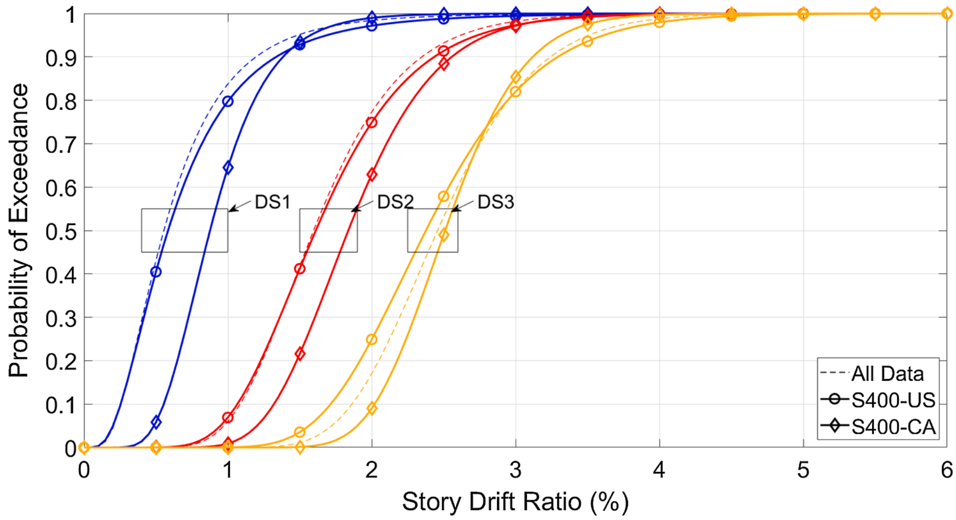

Fig. 23. Updated fragility curves for CFS framed walls with steel sheet sheathing per AISI S400-15.

Table 7
Updated parameters of the fragility functions for CFS framed walls with steel sheet sheathing per AISI S400-15 (US and Mexico), demand parameter: story drift ratio (%).

Damage State	Median	Log SD	# Samples	Lilliefors Test*
DS1	0.58	0.65	76	Fails**
DS2	1.61	0.32	76	Passes
DS3	2.38	0.25	63	Passes

* at 5% significance level.
** passes the test at 10% significance level.

Table 8
Updated parameters of the fragility functions for CFS framed walls with steel sheet sheathing per AISI S400-15 (Canada), demand parameter: story drift ratio (%).

Damage State	Median	Log SD	# Samples	Lilliefors Test*
DS1	0.88	0.36	50	Passes
DS2	1.84	0.26	50	Passes
DS3	2.51	0.17	40	Passes

* at 5% significance level.

earthquake using ground motion data from the Canoga Park records (CNP) and Rinaldi Receiving Station (RRS). Table 10 lists the details of the scenarios.

Figs. 28–30 show the histogram of repair costs based on 200 realizations. The average repair costs, rounded to nearest 1000, calculated using the current and updated fragility functions are listed in Table 11.

The results show that the updated fragility functions estimate less damage than what the current functions provide for low, medium, and high intensity earthquake scenarios. In addition, the updated fragility functions for OSB sheathed walls give higher damage, and consequently, higher repair costs than the updated functions for all walls with WSP sheathing. In scenario 1, which is the least intense earthquake scenario, according to the test report, the building experienced a maximum inter-story drift ratio of 0.2%. As shown in Fig. 28, the distributions of the cost variable are highly right-skewed and ranges between 0 to $11,000 with an average of $2000 based on current fragility functions and $1000 based on the updated functions for OSB-only sheathed walls. The updated fragility functions for all WSP walls estimate a much lower repair cost which is rounded to zero.

For scenario 2 which corresponds to the design basis earthquake, the maximum inter-story drift ratio in the building was 0.48%. The distribution of the cost variable obtained using current fragility functions

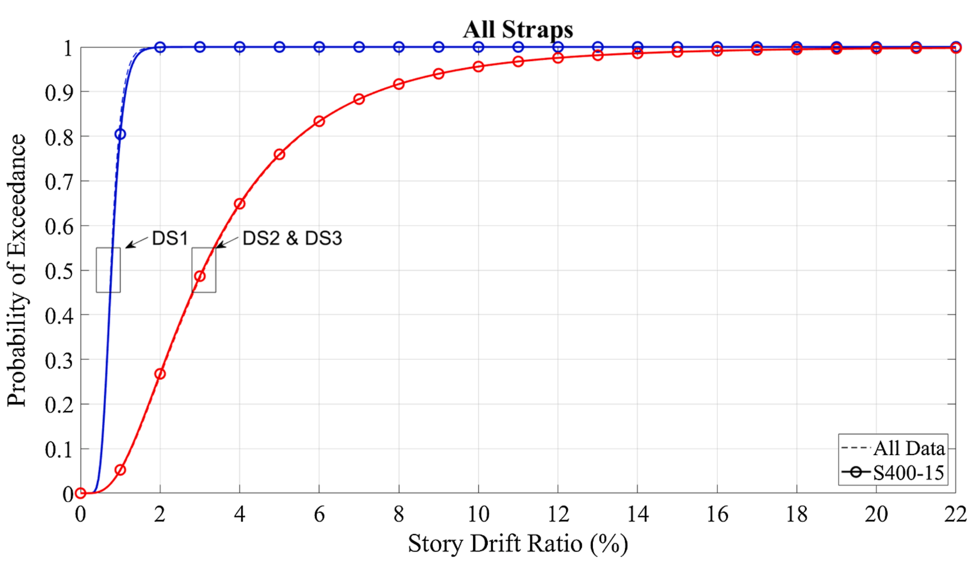

Fig. 24. Updated fragility curves for CFS framed walls with flat strap X-bracing per AISI S400-15 (all steel grades inclusive).

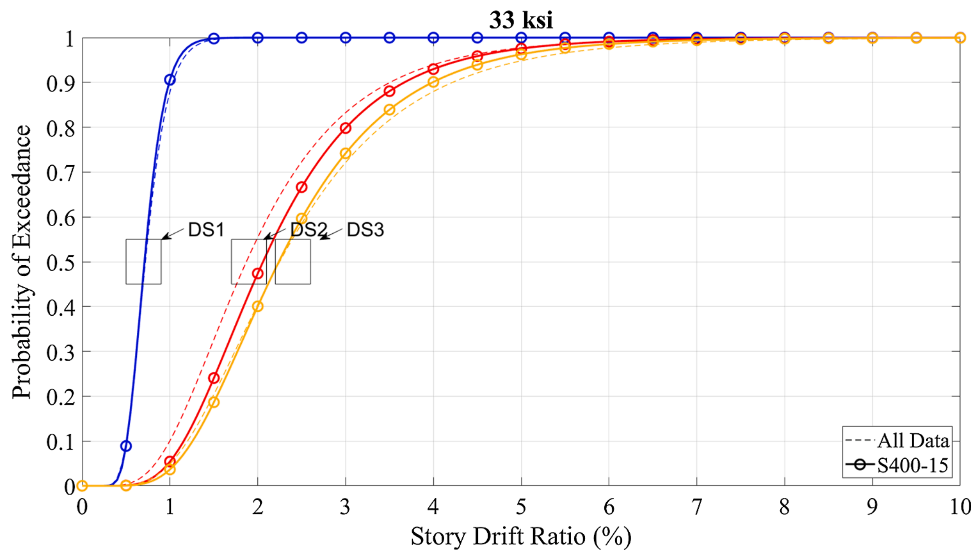

Fig. 25. Updated fragility curves for CFS framed walls with X-bracing per AISI S400-15, steel grade: 230 MPa (33 ksi).

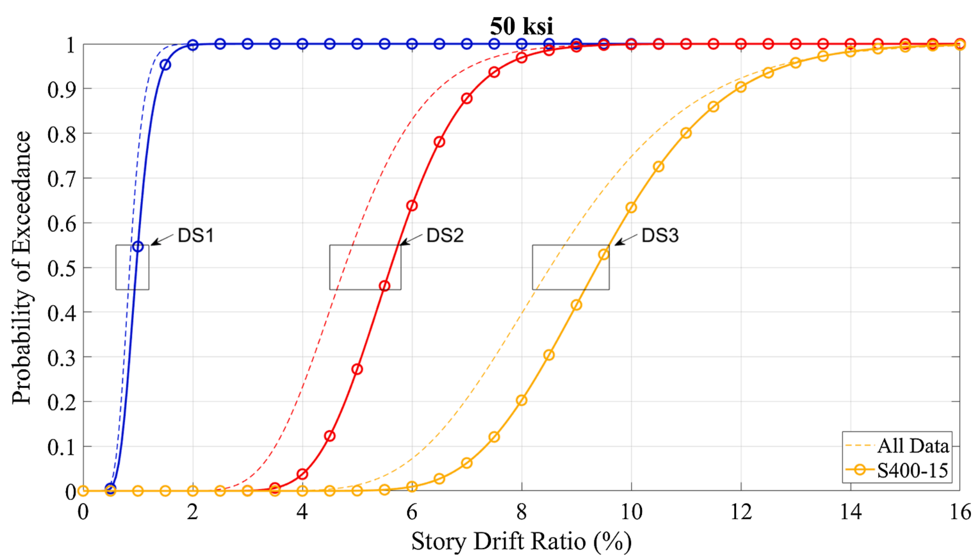

Fig. 26. Updated fragility curves for CFS framed walls with X-bracing per AISI S400-15, steel grade: 345 MPa (50 ksi).

Table 9
Updated parameters of the fragility functions for CFS framed walls with flat strap X-bracing per AISI S400-15, demand parameter: story drift ratio (%).

Steel Grade	Damage State	Median	Log SD	# Samples	Lilliefors Test *
All	DS1	0.78	0.29	49	Passes
	DS2	3.07	0.69	49	Passes
	DS3	3.07	0.69	23	Fails**
33 ksi	DS1	0.71	0.26	35	Passes
	DS2	2.06	0.45	35	Passes
	DS3	2.24	0.45	16	Passes
50 ksi	DS1	0.97	0.26	14	Passes
	DS2	5.61	0.19	14	Passes
	DS3	9.37	0.19	7	Passes

* at 5% significance level.
** passes the test at 10% significance level.

show that the variability in repair cost is mainly due to frequent modestly sized deviations. The cost ranges between 0 and $24,000 with an average of $10,000. On the other hand, the distributions of the repair

Fig. 27. CFS-NEES test building on shake tables at University of Buffalo SEES Lab.

Table 10
Earthquake scenarios and overall building response for the CFS-NEES case study.

Scenario	Ground Motion	Hazard Level	Scale Factor	Spectral Acceleration (g)*
1	CNP	50% in 50 years	0.436	0.45
2	CNP	10% in 50 years (DBE)	1	1.04
3	RRS	2% in 50 years (MCE)	1	1.4

* Spectral acceleration of the input motion at the building's fundamental period in the direction of shaking.

cost estimated using the updated fragility functions are highly right-skewed, which implies that the variability in the repair cost is mainly due to infrequent extreme deviations. The updated fragility functions for all shear walls with WSP sheathing estimate the cost to range from 0 to $19,000 with an average of $3000, while the functions for OSB-only walls estimate relatively higher costs, up to $23,000, with an average of $5000.

In scenario 3, the maximum considered earthquake scenario, the building experienced a maximum inter-story drift ratio of 0.72%. The distributions of the repair cost per current and updated fragility functions are only slightly skewed with broader peaks than other scenarios. Although all three sets of fragility functions estimate the cost variable to be less than $35,000, the variability in cost per current functions is left-skewed with an average of $20,000, while for the updated functions, the variabilities are right-skewed with an average of $11,000 and 15,000 based on all walls with WSP sheathing and OSB-only sheathing, respectively.

In general, the average repair cost over all three scenarios according to updated fragility functions for all WSP sheathed walls is 60% lower than the estimate based on current fragility functions. For updated fragility functions for OSB-only sheathing, the average cost is 40% lower.

7.2. Case study II: CFS-HUD

CFS-HUD is a completed project funded by the U.S. Department of Housing and Urban Development, Office of Policy Development and Research, to study the earthquake and post-earthquake fire behavior of mid-rise CFS framed buildings. The project is a collaboration between University of California, San Diego (UCSD) and Worcester Polytechnic Institute, Department of Housing and Urban Development, California Seismic Safety Commission, and more than fifteen industry partners [34].

The test building was a full-scale CFS framed building with CFS framed shear walls braced with steel sheet sheathing. The building used a proprietary steel sheet sheathing product, but for the purposes of

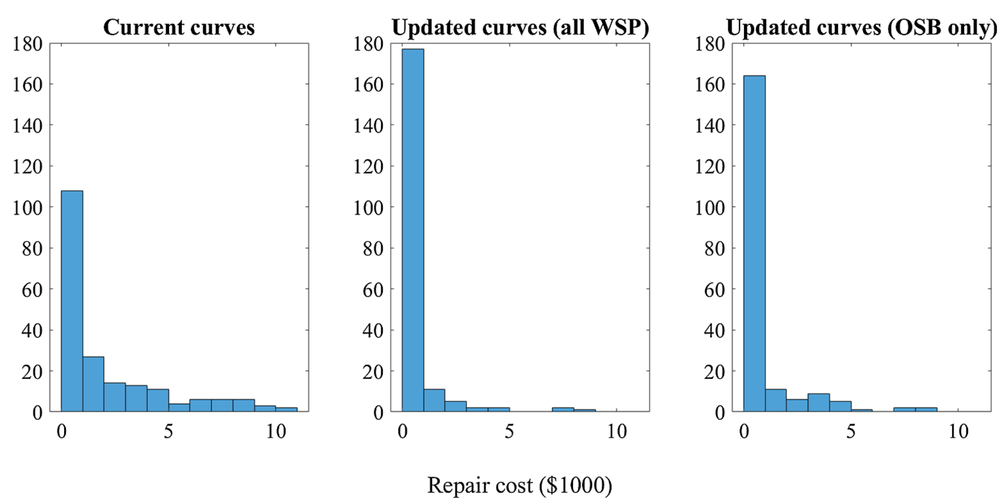

Fig. 28. Histograms of repair cost for scenario 1 (CFS-NEES).

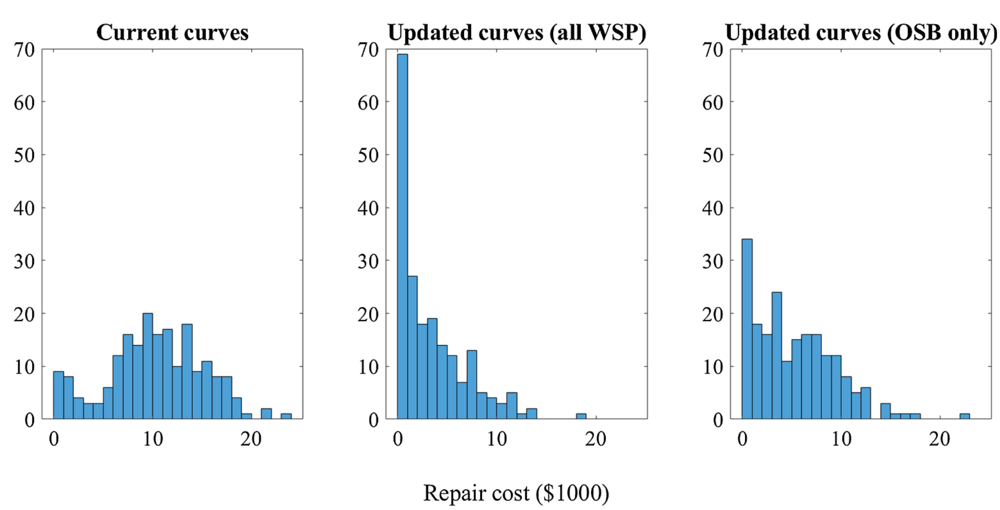

Fig. 29. Histograms of repair cost for scenario 2 (CFS-NEES).

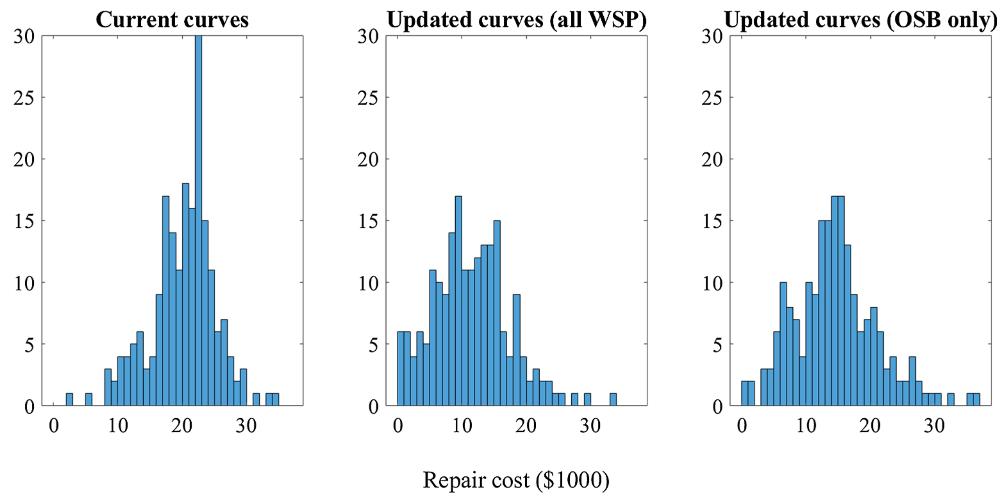

Fig. 30. Histograms of repair cost for scenario 3 (CFS-NEES).

Table 11
Average repair cost of shear walls for the CFS-NEES case study.

Scenario	Average repair cost ($)		
	Current curves	Updated curves (all walls with WSP)	Updated curves (walls with OSB)
1	2000	0	1000
2	10,000	3000	5000
3	20,000	11,000	15,000

comparison, here, we will assume standard fastened steel sheet sheathing. The building dimensions were 34 ft. by 24 ft. in plan and 64 ft. in height designed as a functioning six-story residential building with the hypothetical location at Los Angeles, California. The nonstructural components of the building consisted of interior partition walls, doors, and household appliances. The test building was constructed on the world's largest outdoor shake table (LHPOST) at UCSD (see Fig. 31) and was subjected to seven earthquake tests of increasing motion intensity. After initial seismic tests, the building underwent a set of six live fire tests at two selected floors. Lastly, the test building was subjected to two post-fire earthquake tests.

The repair costs of shear walls are calculated for three seismic scenarios based on the 1994 Northridge earthquake using ground motion data from the Canoga Park records (CNP) and Rinaldi Receiving Station (RRS). Table 12 lists the details of these scenarios.

Fig. 31. CFS-HUD test building on the world's largest outdoor shake table (LHPOST) at UCSD (photo credit: [34]).

Table 12
Earthquake scenarios for the CFS-HUD case study.

Scenario	Ground Motion	Hazard Level	Test phase	Spectral Acceleration (g)*
1	CNP	10% in 50 years (DBE)	Pre-fire	1.37
2	CNP	2% in 50 years (MCE)	Pre-fire	2.01
3	RRS	2% in 50 years (MCE)	Post-fire	2.54

* Spectral acceleration of the input motion at the building's fundamental period in the direction of shaking.

Figs. 32–34 show the histogram of repair costs based on 200 realizations. The average repair costs, rounded to nearest 1000, calculated using the current and updated fragility functions are listed in Table 13.

The results show that the updated fragility functions estimate more damage than what the current functions provide for low, medium, and high intensity earthquake scenarios. Recall that current estimates are based on a very small sample and the updated estimates on a much larger sample size. Furthermore, the current fragility functions for walls with steel sheet sheathing consist of two damage states corresponding to peak load and 80% post-peak load while the updated functions introduce the damage state corresponding to the end of the elastic behavior spectrum. However, as the repair cost corresponding to the added damage state is not available, DS1 was not included in the repair cost estimation. Had we included DS1 of the new fragility functions, the estimated repair cost would increase.

In scenario 1, which is the least intense earthquake scenario, according to the test report, the building experienced a maximum inter-story drift ratio of 0.88%. The distributions of the cost variable are highly right-skewed and varies up to $50,000 with an average of $6000 according to current fragility functions and $11,000 per the updated functions, which shows an 80% increase in the cost estimation.

For scenario 2, the maximum inter-story drift ratio in the building was 1.71%. The distribution of the cost variable obtained using current fragility functions show that the variability in repair cost is mainly due to frequent modestly sized deviations and has a broader peak than a normal distribution. The cost ranges between 0 and $80,000 with an average of $37,000. The distributions of the repair cost estimated using the updated fragility functions follows a normal distribution. The updated fragility functions estimate the cost to range from 0 to $85,000 with an average of $37,000, which is about 20% higher than the estimation based on current fragility functions.

In scenario 3, the building experienced a maximum inter-story drift

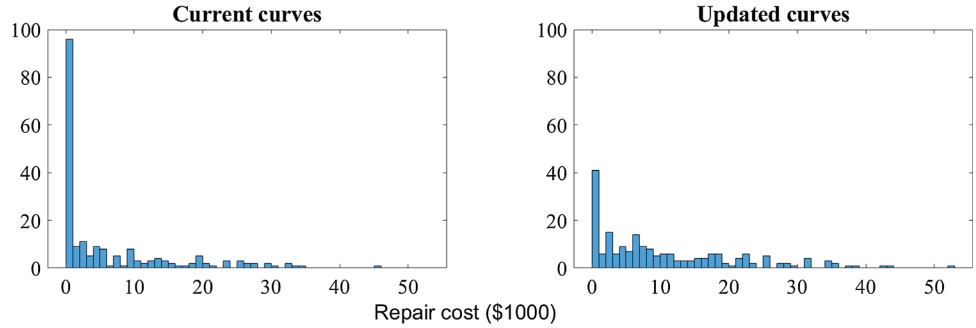

Fig. 32. Histograms of repair cost for scenario 1 (CFS-HUD).

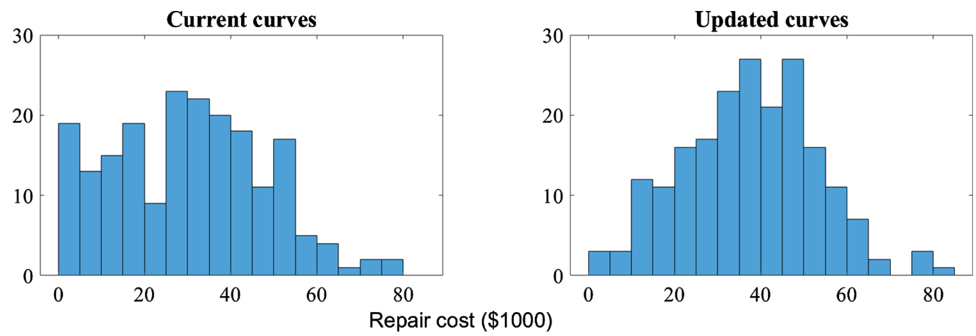

Fig. 33. Histograms of repair cost for scenario 2 (CFS-HUD).

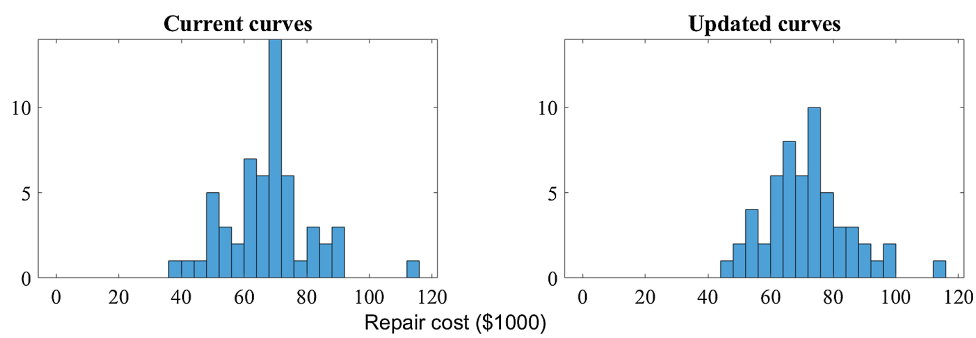

Fig. 34. Histograms of repair cost for scenario 3, excluding collapse cases (CFS-HUD).

Table 13
Average repair cost of shear walls for the CFS-HUD case study.

Scenario	Average repair cost ($)	
	Current curves	Updated curves
1	6000	11,000
2	30,000	37,000
3	739,000	740,000

ratio of 3.00%. 144 from 200 realizations resulted in the replacement of the whole structure based on both the current and updated fragility functions. The replacement cost of the building is estimated to be $1 million. The distributions of the repair cost per current and updated fragility functions are only slightly skewed with broader peaks than other scenarios. Both the current and updated fragility functions estimate the repair cost to go over $100,000 with an average of about $740,000 (i.e., essentially total loss in the post-earthquake followed by post-fire scenario).

8. Conclusions

Performance-based earthquake engineering (PBEE) aims to help engineers better understand and manage earthquake risks through resilient design of new buildings and better quantification of seismic risks for existing buildings. In this process, it is essential to understand how damage is connected to seismic hazard. In this regard, a set of 60 fragility functions are developed for Cold-Formed Steel (CFS) framed shear walls with wood structural panel sheathing, steel sheet sheathing, and flat strap X-bracing. These functions are developed based on a large database of CFS framed shear wall tests and in accordance with the reporting requirements for the U.S. FEMA P-58 methodology. These updated fragility functions are recommended to be used for probabilistic damage assessment for buildings with CFS framed shear walls. To further enable PBEE, more research needs to be conducted on the development of such fragility functions for other types of CFS framed walls and beyond shear walls, and more extensively, for other structural systems.

CRediT authorship contribution statement

Fardad Haghpanah: Conceptualization, Data curation, Formal analysis, Investigation, Methodology, Software, Validation, Visualization, Writing - original draft, Writing - review & editing. **Benjamin W. Schafer:** Conceptualization, Funding acquisition, Investigation, Methodology, Project administration, Resources, Supervision, Validation, Writing - original draft, Writing - review & editing.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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