This is an Accepted Manuscript of an article published by Elsevier which is available at: Mirzaei A, Sangree RH, Velchev K, Comeau G, Balh N, Rogers CA, Schafer BW (2015), "Seismic capacity-based design of narrow strap braced cold-formed steel walls", Journal of Constructional Steel Research 115: 81-91. DOI: https://doi.org/10.1016/j.jcsr.2015.08.023

Seismic Capacity-Based Design of Narrow Strap-Braced Cold-Formed Steel Walls

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Abstract

In North America, the seismic design of strap-braced cold-formed steel shear walls is carried out using the AISI S213 Standard, which is soon to be replaced by a new seismic specific standard AISI S400. Both Standards require the use of a capacity-based design procedure in which the tension-only diagonal braces are assumed to act as the inelastic fuse elements in the pin connected seismic force resisting system, while all other elements remain essentially undamaged under loading. Experimental work has shown this assumption to be valid for walls with low aspect ratios; however, the testing of high aspect ratio walls has revealed that large moments develop in the frame members, which can result in their failure prior to yielding of the braces. This paper describes a simple method with which these frame moments can be determined and accounted for in the capacity-based design procedure.

Keywords: cold-formed steel, strap brace, shear wall, capacity-based design, seismic, wall aspect ratio.

Highlights

- Testing and analysis of high aspect ratio cold-formed steel strap-braced shear walls
- High aspect ratio walls do not behave as pin connected truss systems
- Frame action must be accounted for in the capacity design procedure
- A simple analysis and design method is proposed for inclusion in relevant standards

1. Introduction

In North America, the seismic design of cold-formed steel lateral framing systems is carried out following the provisions found in the American Iron and Steel Institute (AISI) S213 Standard for "Cold-Formed Steel Framing – Lateral Design" [1], which is soon to be replaced by a new seismic specific design standard, AISI S400 "North American Standard for Seismic Design of Cold-Formed Steel Structural Systems" [2]. The seismic capacity-based design procedure for diagonal strap-braced cold-formed steel (CFS) walls (Figure 1a) in both of these standards was formulated, in part, considering the performance of full-scale wall test specimens with 1:1 to 2:1 aspect ratios [3-14]. The intent of the capacity-based design procedure is to ensure that the strap braces act as fuse elements, dissipating seismic energy while limiting the wall resistance with a controlled, ductile yielding of the cross-section along the brace length. All other elements in the seismic force resisting system (SFRS), i.e. brace connections, gusset plates, chord studs, track, anchor rods, hold-downs and shear anchors, must be designed to have a resistance higher than the forces that are associated with the expected yield strength in tension of the strap braces along with any gravity loads that are applied in combination with the earthquake loads.

At present, the Eurocode standards for seismic and cold-formed steel design [15, 16] do not specifically address the lateral system design of a cold-formed steel framed wall. Nonetheless, researchers in Europe have proposed relevant design methods. Dubina [13], for example, has summarized methods to analyse and design a variety of cold-formed steel framed wall systems, including strap-braced walls. Fiorino et al. [17] did develop a design method for oriented strand board (OSB) sheathed shear walls, which was implemented in Italy; this method however, is not applicable for strap-braced walls. Macillo et al. [18] have established links with the existing Eurocode 1998-1-1 [15] for hot-rolled steel cross-braced frames and that of the cold-formed steel strap-braced walls tested by Iuorio et al. [19] and others. This Eurocode 1998-1-1 standard contains provisions for traditional hot-rolled steel concentrically braced frames with X diagonals, including tension-only configurations. However, these provisions were not originally intended for use with the specifics of a CFS framed structure. Furthermore, this work addressed low aspect ratio walls within the range 1:1 to 2:1. Note, Macillo et al. made mention that non-energy dissipating members of the wall such as beams and columns are "…evaluated by considering the interaction with the bending moment (M_{Ed}), that is generally null for the examined systems;…" [17]. Given the aspect ratio of the walls that were being studied this statement does have an element of truth, however, the development of moments in the framing occurs as the aspect ratio increases, as will be demonstrated herein. Further to this, Tian et al. [8] carried out tests on 2:1 aspect ratio walls (2.45 m in height × 1.25 m in length) for which chord stud failures were observed; the design of these studs was not done following a capacity based approach that accounted for both probable axial compression force and probable bending moment.

Research on strap-braced walls is of course not limited to North America and Europe. As an example, Moghimi and Ronagh [20] also presented a design approach of strap-braced walls in which the risk of connection and stud failure is minimized; however, this again was carried out for 1:1 aspect ratio walls (2.4 m x 2.4 m).

In the calculation of the forces that transfer through the SFRS following the North American AISI Standards, as the strap braces yield the triangulated configuration of the wall leads to the notion of pin connected truss-like behavior, which results in only axial forces being applied to the chord studs. Experimental evidence has shown that for walls with 1:1 aspect ratios the joint fixity is for the most part inconsequential [12]; however, tests on narrow 4:1 aspect ratio walls, and even 2:1 aspect ratio walls, by Comeau & Rogers [21] and Velchev & Rogers [22] have revealed that as the aspect ratio increases the joint fixity at the wall corners leads to frame-like behavior, subjecting the chord studs to combined bending moments and axial forces. The hold-down devices used to connect the chord studs to the underlying foundation (Figure 1b), or to the braced wall located in the story above or below, and the gusset plates commonly used to attach the chord studs and track to the brace (Figure 1b) result in flexurally stiff connections and the subsequent development of moments in the frame members. The combination of high axial compression force and bending moments may lead to the failure of the chord studs prior to strap yielding if unaccounted for in capacity design. This will result in a decrease of the ductility of the SFRS and a loss of the post-earthquake gravity load-carrying ability of the structural walls.

The objective of this paper is to describe an investigation of the response to lateral loading of high aspect ratio strap-braced walls in which both experimental and analytical evidence is provided to illustrate the development of frame bending moments. Ultimately, a proposal is made for a simple capacity-based design procedure that accounts for both the axial and flexural forces applied to the frame members of cold-formed steel strap-braced walls.

2. Current Design Approach

The North American approach for the design of CFS strap braced walls subjected to in-plane lateral seismic loading can be found in the AISI S213 [1] and S400 [2] Standards. The assumption of truss behavior allows for the use of simple trigonometric relations to determine axial force demands in the chord studs, for example, associated with the expected tensile force of the strap braces and any companion gravity loads. As long as the factored axial resistance of the selected chord stud is greater than this demand, the belief is that the braces will yield in tension before any damage is done to the studs. The "undamaged" chord studs would then be available to maintain their gravity load-carrying role post-earthquake.

2.1 SFRS member demand and resistance

The brace members in a cold-formed steel framed structure are initially selected in consideration of the lateral force and drift requirements imposed by the relevant building code for both wind and seismic loading. The factored tension resistance of the braces is determined using the standard approach found in AISI S100, the "North American Specification for the Design of Cold-Formed Steel Structural Members" [23], accounting for the net cross-section fracture and gross cross-section yielding failure modes. Strap braces are considered to have no resistance in compression due to their high slenderness; hence, a tension-only lateral structural system exists. The brace selection may also be contingent upon the stability requirements of the structure under gravity loading and drift limits.

Subsequent to the initial selection of the braces further seismic specific design provisions, material property requirements and detailing as per the AISI S213 [1] and AISI S400 [2] Standards must be met. Although a factored resistance of the braces has been determined in the initial selection of the members, in the event of a rare design level earthquake it is expected that the force in the braces will exceed this resistance level and will enter into the inelastic range of behavior. Energy arising from the ground motion will be dissipated due to the ability of the steel braces to carry load after yielding of the cross-section. However, to ensure this response the remaining elements in the SFRS must be prevented from being damaged, to avoid a reduction in the wall's ability to carry lateral loads, through capacity protection. In this process, it is necessary to determine the expected force level at which brace yielding will take place. For the purposes of this paper it is assumed that the expected brace capacity falls below the upper limits for the capacity forces, i.e. the amplified seismic load (US) and the maximum anticipated seismic loads calculated using the seismic force modification coefficients $R_dR_o = 1$ (Canada). AISI S213 [1] and AISI S400 [2] define the expected brace capacity, F_B, as:

$$F_B = R_y A_g F_y \qquad \text{Eq. 1}$$

where A_g is the gross cross-sectional area of the brace, and F_y is the nominal yield strength of the steel. The ratio of expected yield strength to nominal yield strength, R_y, ranges between 1.5 for 230 MPa steels to 1.1 for 340 MPa steels [1,2]. Once F_B is known, simple trigonometric relations provide the solution to the lateral force (V) associated with the expected brace yield strength and chord stud demand (F_C), assuming the wall exhibits truss-like behavior (Figure 2):

$$V = F_B \cos\theta \qquad \text{Eq. 2}$$

$$F_C = F_B \sin\theta \qquad \text{Eq. 3}$$

where $\theta = \tan^{-1}\left(\frac{h}{w}\right)$

The expected brace capacity is then used along with the companion gravity loads to determine the forces arising in all other members and connections of the SFRS.

As part of the capacity design calculations, AISI S213 [1] and AISI S400 [2] require that the expected tensile strength of the brace exceeds its expected yield strength such that net section fracture does not occur prior to the inelastic seismic drift limit being reached:

$$R_t A_n F_u \geq R_y A_g F_y \qquad \text{Eq. 4}$$

where A_n is the net cross-sectional area of the brace, F_u is the nominal tensile strength of the steel and where R_t extends from 1.2 for 230 MPa steels to 1.1 for 340 MPa steels [1,2].

Assuming only axial demands, as is the case in the AISI S213 [1] and AISI S400 [2] capacity-based design approach, the factored concentric compression resistance of the chord stud, determined as per AISI S100 [23] must not be less than the vertical component of the expected yield strength of the diagonal strap bracing member plus the corresponding gravity component of force due to the load combination being considered. Similarly, the brace connections, hold-downs, shear anchors, gusset plates and tracks are designed based on the force level associated with the expected yield strength of the braces.

3. Experimental Program

Tests of 44 strap-braced walls were performed in the Jamieson Structures Laboratory at McGill University on a test frame built specifically for the lateral loading of CFS framed walls (Figure 3) [12]. The specimens were designed using the capacity-based approach provisions in AISI S213 [1] for walls with diagonal strap-braces, where the elements in the SFRS were chosen based on the expected yield strength of the braces and assuming truss action in the distribution of member forces; note, gravity loads were not applied to these test walls, nor were they considered in design. The scope of testing included walls ranging from 610 mm in length to 2440 mm, all of which were 2440 mm tall. The 2440 x 2440 mm walls were designed for testing using three levels of factored lateral strength: light (20 kN capacity), medium (40 kN capacity) and heavy (75 kN capacity). Test walls 1220 x 2440 mm and 610 x 2440 mm in size were also detailed using the same size braces as found for the larger 1:1 aspect ratio walls. The expected vertical and horizontal forces in the SFRS, as well as the selection of other elements in the lateral load path, were determined considering the modified angle of the braces in these higher aspect ratio walls. Details of all initial 44 walls including the measured material properties and the lateral loading response can be found in the work of Comeau & Rogers [21] and Velchev & Rogers [22]. A summary of the test program is found in the paper by Velchev et al. [12].

An additional high aspect ratio strap-braced wall (49A-M) was later included in the test program; it contained thicker chord studs (1.74 mm vs 1.37 mm) than the comparable 4:1 walls with 69.9 mm wide (medium) braces (19B-M1, 19B-M2 and 20B-C) (Table 1). A schematic drawing of walls 19B-M1, 19B-M2 and 20B-C is found in Figure 4, while wall 49A-M is illustrated in Figure 5. In addition to the thicker chord studs, for ease of construction wall 49A-M was built using screw connections instead of welds, and slightly larger gusset plates were installed which resulted in higher connection stiffness between the chords and track. The intent was to construct a narrow wall that could carry the expected axial and moment forces applied to the chord studs under lateral loading.

The test program comprised displacement-controlled monotonic and reversed cyclic protocols applied using a 250 kN capacity actuator with a stroke length of ±125mm. Loading continued until a significant drop in resistance was observed or until the useable travel of the actuator was reached. The Consortium of Universities for Research in Earthquake Engineering (CUREE) ordinary ground motions reversed cyclic loading protocol [24, 25] was used for the cyclic tests. Displacement data was collected from the actuator's LVDT and a cable-extension transducer connected to the top of the wall. A load cell placed in line with the actuator provided a measurement for lateral resistance of the wall. Strain gages were placed on each diagonal strap to identify the onset of yielding during the test. Additional LVDTs and load cells were used to measure slip and uplift of the wall with respect to the frame near the bottom corners of the walls. The measurement instruments were connected to Vishay Model 5100B scanners that were used to record data using the Vishay System 5000 StrainSmart software [21,22].

3.1 *Results of narrow strap-braced wall tests*

The tests of the high aspect ratio (4:1) walls demonstrated that the current capacity-based design procedure for strap-braced walls found in AISI S213 [1] and AISI S400 [2] is inadequate. Due to the slenderness of these braced walls, and the use of gusset plates and hold-downs, the chord studs develop consequential bending moment under lateral loading (Figure 6a). The chord studs are in effect beam-columns subjected to double curvature bending as well as axial compression; they are not simple axial load carrying truss members.

The tests of specimens 19B-M1, 19B-M2, 20B-C, 23C-M1, 23C-M2 and 24C-C [12,21,22] illustrated that these high aspect ratio walls are not able to reach and maintain a lateral load carrying resistance corresponding to that defined by yielding of the braces; Figure 7 shows the results of the monotonic tests. Failure of the chord studs under bending (Figure 6b) and axial loads reduced both the lateral resistance and ductility of the walls.

To address this combined axial compression - bending failure mode it was proposed that the capacity design of the chord studs be carried out accounting for both the expected axial and bending forces, as described in Section 4. Specimen 49A-M (Figure 5; Table 1) was redesigned using the same braces as walls 19B-M1, 19B-M2 and 20B-C, constructed in the laboratory, and then tested under a monotonic loading protocol to a drift of over 8% (Figures 7a, 8). The wall was able to reach and maintain the lateral resistance associated with yielding of the braces; strain gauge measurements confirmed that the braces yielded [22]. The chord studs did not fail, although at large drifts they did show signs of elastic distortional buckling (Figure 9b). Note: once the lateral load was removed from the wall the chord studs rebounded elastically, while the tension braces were plastically elongated. The wall was less stiff than tests 19B-M1 and 19B-M2 (Figure 7a) because the brace angle was steeper due to the change in placement of the brace end screw connections on the gusset plates versus that used for the weld connected braces (Figures 4 & 5).

Although the results of test 49A-M show that high aspect ratio walls can reach a lateral load level associated with the expected yield strength of the braces, this required lateral drifts approaching 4%. As such, the design of these walls under factored lateral forces (wind or seismic) would be controlled by drift limitations. The predicted stiffness for drift calculations must not be based solely on the axial stiffness of the braces. As was described by Velchev et al. [12] other components in the lateral load carrying path such as anchor rods, hold-downs, brace connections and possible brace flexure (Figure 9a) need to be accounted for such that the predicted stiffness approximates that which was measured in the laboratory. Note: the predicted lateral stiffness, K_p, values shown in Figure 7 were determined using the axial stiffness of the braces alone.

4. Proposed Capacity-Based Design Approach

Failure of the chord studs as was observed during the strap-braced wall experiments indicates that the current AISI S213 [1] and S400 [2] capacity design approach, with its assumed truss analysis model, does not adequately ensure brace yielding as the primary inelastic response. The chord studs of the tested 4:1 aspect ratio walls, except 49A-M, failed in combined axial compression and bending, indicating that high-aspect ratio walls behave more like frames than trusses, generating bending moment demand in the chord studs. This behavior must be considered in the capacity-based design approach for narrow braced walls; a proposed method is introduced in the following sections.

4.1 Moment demand in chord studs and the effect of aspect ratio

To determine the moment demand in the chord studs, the elastic analytical model shown in Figure 10 was employed. The model assumes fixed connections except at the ends of the braces. The compression strap is removed, since its high slenderness would not allow any compressive force to develop. A shear force is placed parallel to the top track; the bottom track is fixed to the support (foundation) at each chord stud location. The model is simplified to identify key components of the wall; a more detailed model may include interior studs and other wall elements, if present.

The model was consequently decomposed into a truss and frame action component (Figure 10) to identify the participation of each system in carrying lateral loads. The two components are connected in series by an axially rigid pinned link. A finite element study was performed in which a unit lateral load was applied to both systems shown in Figure 10. This study demonstrated that the lateral deflection and internal forces of the decomposed model match the results obtained from the initial wall model (Figure 11). This operation was performed to simplify the derivation of equations that were used to determine the moment in the chord studs, and to study the effect of the wall aspect ratio on the generated moment magnitude due to the frame action.

The decomposed model was then replicated for strap-braced walls having various height-to-width ratios (h/b); these models were employed to quantify the shear participation of the truss and frame action. As can be seen from Figure 12, where V_F is the total base shear due to the frame action and V_T is the base shear generated by the truss action component, for walls with an aspect ratio of 2:1 (h/b = 2) and above the frame action is substantially increased with the simultaneous reduction of the truss action in terms of carrying lateral loads. The behavior of the strap-braced walls with aspect ratios of 1:1 and below is governed by the truss action (over 93% participation), while the frame action component does not have a major contribution. It follows that strap-braced walls with aspect ratios of 1:1 and below can be designed using the current truss action capacity-based design approach. However, for higher aspect ratios the increased shear participation of the frame action leads to moments being developed in the chord studs (Figure 13), which highlights the importance of considering this generated moment in the design of these frame members. Note, this includes the 2:1 aspect ratio walls. Although it was reported by Velchev et al. [12] that the 1220 × 2440 mm walls were able to attain and maintain a lateral load corresponding to the yield strength of the braces, significant bending of the chord studs resulting in damage did occur during testing, as shown in Figure 14. As such, the frame moments should be included in the design process even for walls with aspect ratios less than 4:1.

4.2 ***Moment demand equation derivation***

The moment demand in the chord stud can be determined by using the finite element model methodology explained in Section 4.1. However, a more practical approach for incorporation into design standards is to use the equations presented in this section. The lateral deflection and stiffness of a single-story moment frame with a fixed base (representing the frame action) can be determined by using Eqs. 5 and 6. The lateral deflection and stiffness of a single-story pinned braced frame with a pinned base (representing the truss action) can be determined by using Eqs. 7 and 8:

$$\delta_F = V_F\left[\frac{\frac{6\,I_b}{I_c}+\frac{4b}{h}}{\frac{6\,I_b}{I_c}+\frac{b}{h}}\times\frac{h^3}{24EI_c}\right] \qquad \text{Eq. 5}$$

$$k_F = \left[\frac{\frac{6\,I_b}{I_c}+\frac{4b}{h}}{\frac{6\,I_b}{I_c}+\frac{b}{h}}\times\frac{h^3}{24EI_c}\right]^{-1} \qquad \text{Eq. 6}$$

$$\delta_T = V_T \left[\frac{h^3}{b^2 E A_c} + \frac{(h^2 + b^2)^{1.5}}{b^2 E A_s} \right] \qquad \text{Eq. 7}$$

$$k_T = \left[\frac{h^3}{b^2 E A_c} + \frac{(h^2 + b^2)^{1.5}}{b^2 E A_s} \right]^{-1} \qquad \text{Eq. 8}$$

where:

δ_F: Lateral deflection of the frame system

δ_T: Lateral deflection of the truss system

k_F: Lateral stiffness of the frame system

k_T: Lateral stiffness of the truss system

V_F: Total base shear of the frame system

V_T: Total base shear of the truss system

E: Modulus of elasticity

I_b: Moment of inertia of the track

I_c: Moment of inertia of the chord stud

b: Width of the wall

h: Height of the wall

A_c: Cross sectional area of the chord stud

A_s: Cross sectional area of the strap

Due to the link between the truss and frame systems, the total lateral deflection δ is equal to $\delta_F + \delta_{T.}$ The total lateral force $= V_F + V_T$, therefore the total deflection δ can be calculated by using Equation 9:

$$\delta = \frac{V}{k_F + k_T} \qquad \text{Eq. 9}$$

Shear participation of each system can be then calculated by utilizing Equations 10 and 11:

Frame action shear participation: $V_F = k_F \cdot \delta$ Eq. 10

Truss action shear participation: $V_T = k_T \cdot \delta$ Eq. 11

The gusset plate, base track, and hold-down strengthen the chord stud at its base and increase its rigidity. Experimental results showed that the failure of the chord stud in narrow strap-braced walls occurred directly above the hold-down (Figure 6b). The moment at the base (M_b) and above the hold-down due to frame action (M_h) as shown in Figure 15 can be calculated by using Eqs. 12 and 13.

$$M_b = \frac{V_F\, h}{2}\left[\frac{\frac{3\, I_b\, h}{I_c\, b} + 1}{\frac{6\, I_b\, h}{I_c\, b} + 1}\right] \qquad \text{Eq. 12}$$

$$M_h = M_b\left[\frac{\frac{M_b}{0.5\, V_F} - h_0}{\frac{M_b}{0.5\, V_F}}\right] \qquad \text{Eq. 13}$$

4.3 *Proposed capacity-based design approach*

The proposed capacity-based design approach for strap braced walls is similar to that prescribed in the current AISI S213 [1] and AISI S400 [2] Standards in that the maximum lateral force applied to the wall is defined by means of the truss action system as the horizontal component (Fig. 2) of the expected brace capacity ($F_B \cos\theta$). The compression force in the chord stud is approximated as the vertical component of the expected brace capacity ($F_B \sin\theta$) plus the gravity loads included in the load combination being evaluated ($\overline{P_r}$). The additional bending moment demand on the chord stud is determined by using Eq. 13, or it can be obtained from a structural analysis of the wall system that accounts for realistic member end fixity ($\overline{M_{ry}}$). Once the compression force demand on the chord stud is determined, it may initially be selected based on the current AISI capacity design approach for axial resistance. However, a second step is required given that the cross-section of the chord stud is now known; the applied moments due to the frame action are calculated followed by the use of the beam-column interaction equations for cross-section strength (Eq. 14) and member stability (Eq. 15) from AISI S100 [23]. By controlling the interaction of expected compression force and bending moment in the chord stud, the likelihood of stud failure is minimized. Without this approach, failure of the chord studs will result in a lower than predicted resistance to lateral forces as well as reduced ductility, as observed in the experimental work of Comeau & Rogers [21]; and further, will place the gravity load carrying system of the building at risk of collapse. Specifically, the following interaction check is required:

$$\frac{\bar{P}_r}{P_{no}} + \frac{\overline{M_{ry}}}{M_{ny}} \leq 1.0 \qquad \text{Eq. 14}$$

$$\frac{\bar{P}_r}{P_n} + \frac{C_{my}\overline{M_{ry}}}{M_{ny}\alpha_y} \leq 1.0 \qquad \text{Eq. 15}$$

where:

$\bar{P}_r$: Required axial compression demand.

$\overline{M_{ry}}$: Required flexural demand about the y-axis.

$\frac{C_{my}}{\alpha_y}$: Moment amplification factor of the y-axis for the consideration of second-order moments.

P_n, P_{n0} : Nominal capacity terms for axial compression.

M_{ny}: Nominal bending capacity about the y-axis.

Note: The capacity terms P_n, P_{n0} and M_{ny} are determined following AISI S100 [12].

4.4 ***Verification of the proposed design approach with experimental results***

The proposed capacity-based design approach was examined to evaluate its effectiveness in predicting failure of the chord studs, as was observed for the narrow strap-based wall tests conducted by Comeau & Rogers [21]. The results of the investigation are presented in Table 2.

For each wall the measured lateral yield force is listed as V_y, and the predicted (expected) lateral yield capacity (Eq. 2), determined using the nominal brace properties as per the current AISI S213 and S400 Standards, is given as V_{yp}. Furthermore, the proposed value of V_n is listed; it is defined as the lesser of V_{yp} and the minimum predicted lateral force, V_{yPM} that normalizes the axial force-moment interaction equations (Eq.14 & Eq.15). The moment used in these interaction equations was that determined to occur above the hold-down position (Eq. 13). The test-to-predicted ratios of V_y / V_{yp} and V_y / V_n are included, along with the resulting value obtained from the interaction equations for cross-section strength and member stability, Eq. 14 and Eq. 15, respectively, where the measured lateral force V_y was used.

The combination of axial compression force and moment on the chord studs of high aspect ratio walls is addressed by including the interaction check in the design procedure. It was found that the addition of this new design check improved the strength prediction of the high aspect ratio strap-braced shear walls, and did not inadvertently affect the strength prediction for those walls with lower aspect ratios. In cases where the interaction check controlled the lateral load applied to a wall the mean test-to-predicted ratio was 0.96 if the interaction check were not made, and 1.06 if the interaction check was considered in design (Table 2b). The standard deviation also decreased from 0.13 to 0.11 with the use of the interaction check in these cases. Furthermore, the failure mode of the walls for which the interaction equation controlled the lateral load that could be applied was witnessed to occur in the chord studs [21]; thus, the predicted limit state was consistent with the laboratory observations. The test-to-predicted comparison for all walls in which inelastic damage was limited to yielding of the strap braces was not affected by the inclusion of the new interaction check on the chord studs (Table 2b). When all walls were considered the overall test-to-predicted ratio experienced a slight increase, however, the standard deviation of the results was reduced (Table 2b). Note, in tests 15B-M and 16B-C (2:1 aspect ratio) the predicted lateral load based on strap yielding and that obtained from the chord stud failure interaction equations were essentially the same. In these two walls both failure limit states were observed. In walls 19B-M, 20B-C, 23C-M and 24C-C (4:1 aspect ratio) the combined axial force and bending failure mode in the chord studs dominated the behavior; this response was predicted with the proposed interaction equations. In addition, for the redesigned test wall 49A-M, in which the chord studs were increased in thickness to account for the anticipated bending moment, it was observed that failure of these boundary members under lateral loading of the wall could be avoided.

5. Conclusions

Testing of high aspect ratio strap-braced walls has revealed that the existing truss analysis capacity-based design procedure found in the current AISI S213 lateral design standard for cold-formed steel structures and the new seismic specific standard AISI S400 is inadequate because it does not account for the flexural demand on the chord studs that arises from member end fixity. A proposed simple calculation method with which these frame moments can be determined and accounted for in the capacity-based design procedure was developed. A comparison of the test-to-predicted results demonstrated the ability of the new procedure to identify walls in which chord stud failure occurred, i.e. 4:1 aspect ratio walls, and walls in which obvious damage to the chord studs was observed due to bending action, i.e. 2:1 aspect ratio walls. It is recommended that the proposed capacity-based design method be considered for inclusion in the new AISI S400 North American standard for the seismic design of cold-formed steel structural systems to improve upon the inelastic response of strap-braced shear walls subjected to earthquake ground motions.

Acknowledgements

The authors would like to acknowledge the support provided by the American Iron and Steel Institute and their Standards Council Small Project Fellowship Program, the Canadian Sheet Steel Building Institute, the Canada Foundation for Innovation and the Natural Sciences and Engineering Research Council of Canada. Materials for the test specimens were supplied by Bailey Metal Products Ltd., Simpson Strong-Tie Co. Inc., ITW Buildex and Grabber Construction Products.

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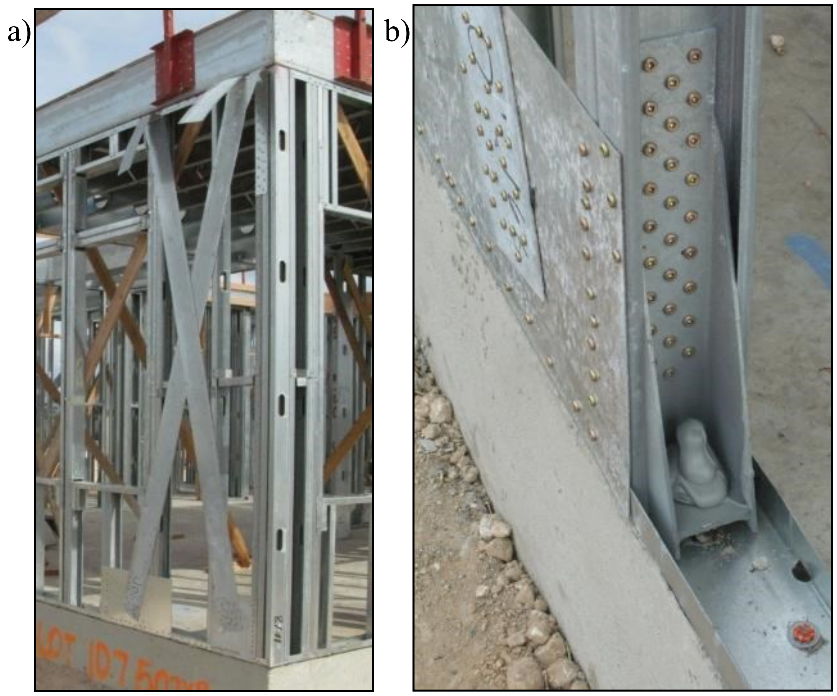

Figure 1: a) Typical narrow diagonal strap-braced cold-formed steel wall, b) hold-down device and gusset plate

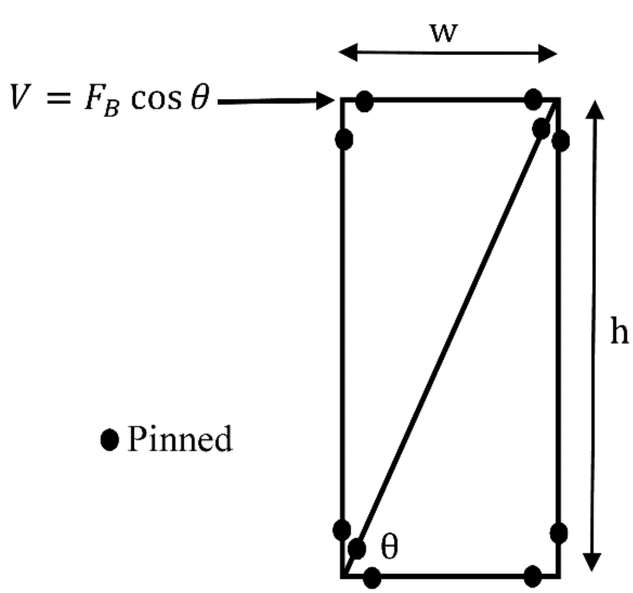

Figure 2: Truss model of a strap-braced wall

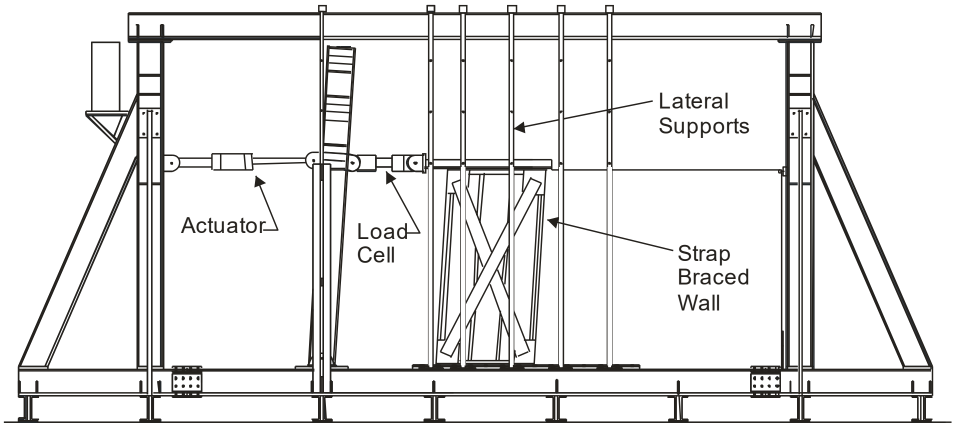

Figure 3: CFS strap-braced wall test frame

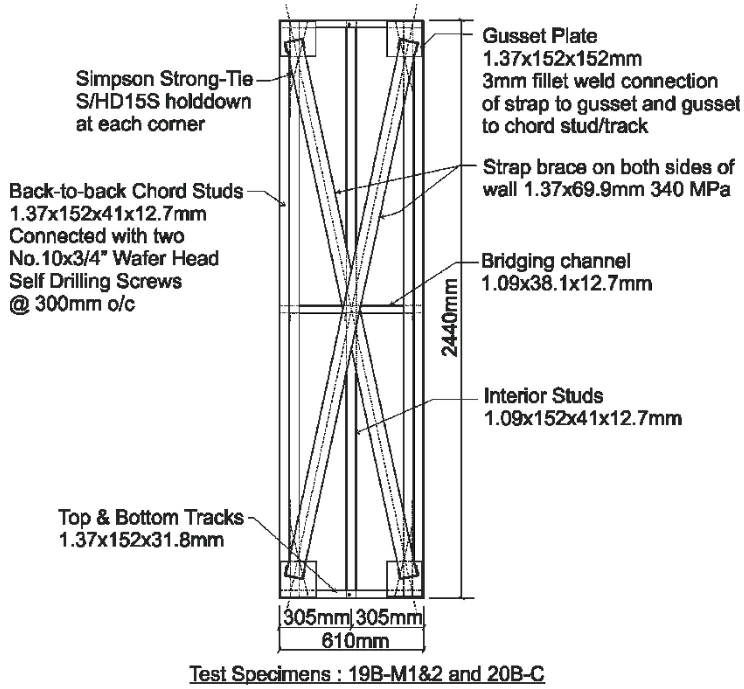

Figure 4: Narrow strap-braced test wall designed assuming truss action (aspect ratio = 4:1)

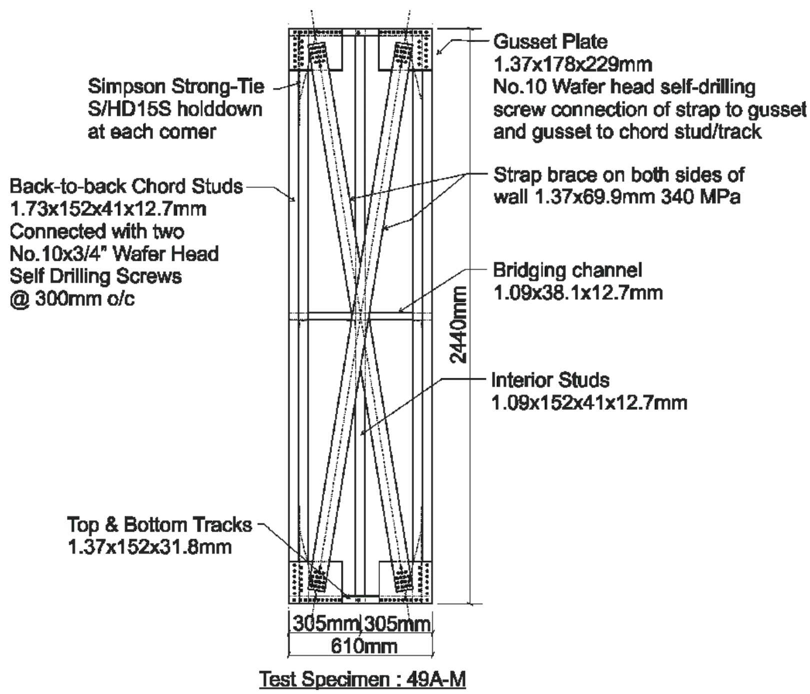

Figure 5: Narrow strap braced test wall constructed with thicker chord studs to account for flexural frame action (aspect ratio = 4:1)

a) b)

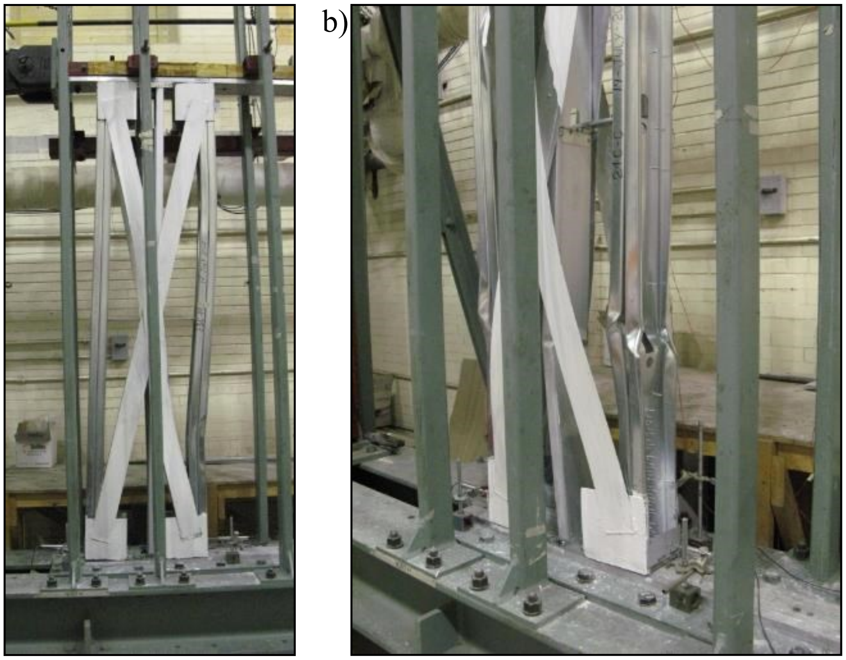

Figure 6: a) Double curvature bending of 610 x 2440 mm strap braced wall 23C-M2, b) Chord stud failure of wall 24C-C

a) b)

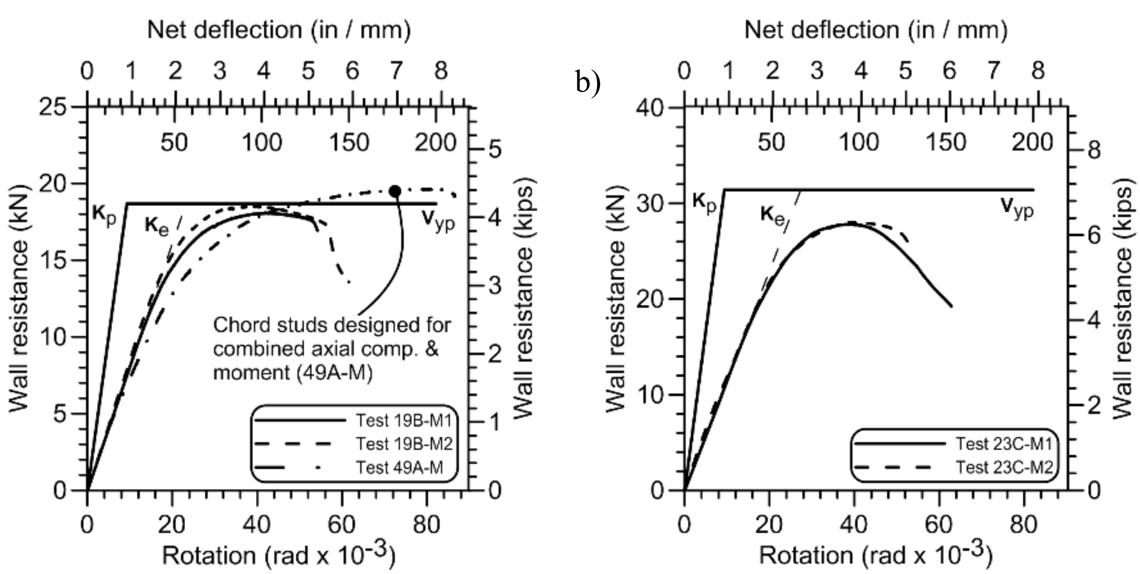

Figure 7: Resistance vs. deflection behavior of 4:1 aspect ratio walls; a) Medium walls, b) Heavy walls

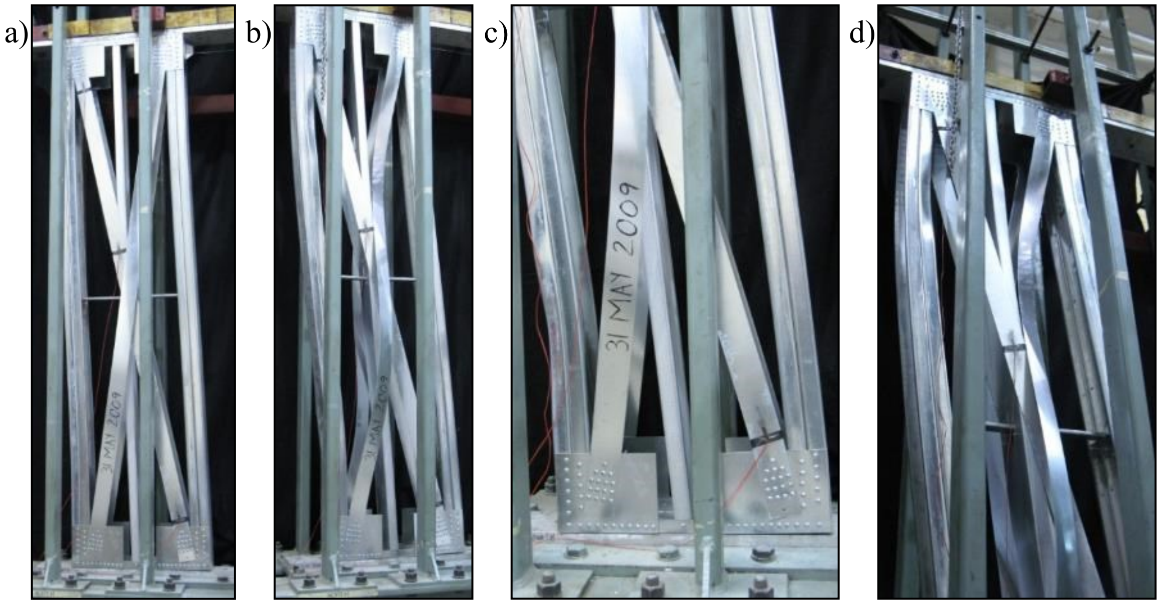

Figure 8: Test 49A-M ; a) 4% drift, b) 7% drift, c) 8% drift, d) 8% drift

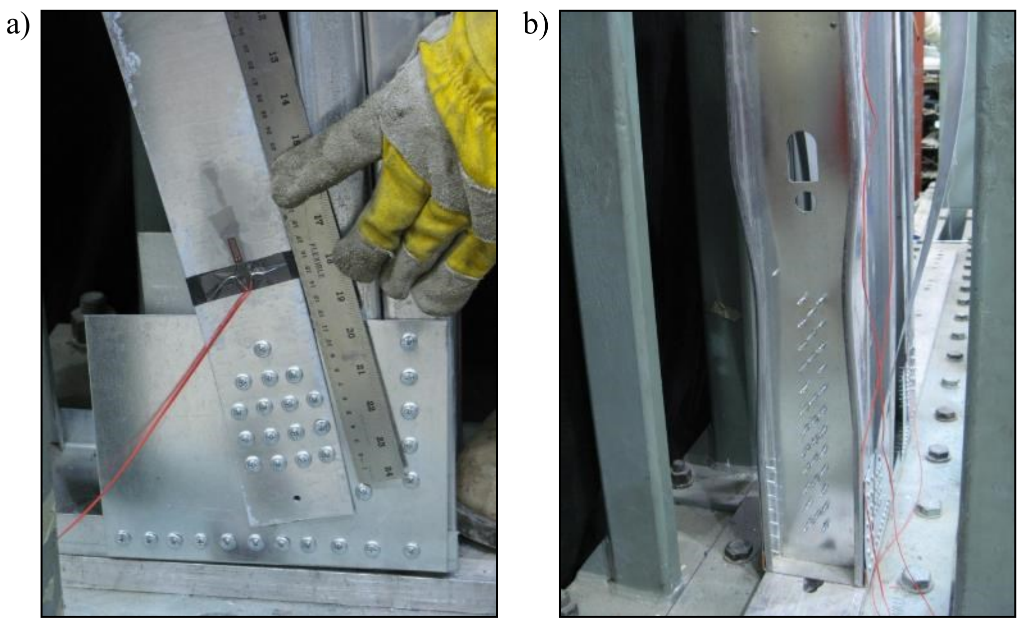

Figure 9: Test 49A-M at 8% drift; a) Flexure of braces, b) Elastic distortional buckling of chord studs

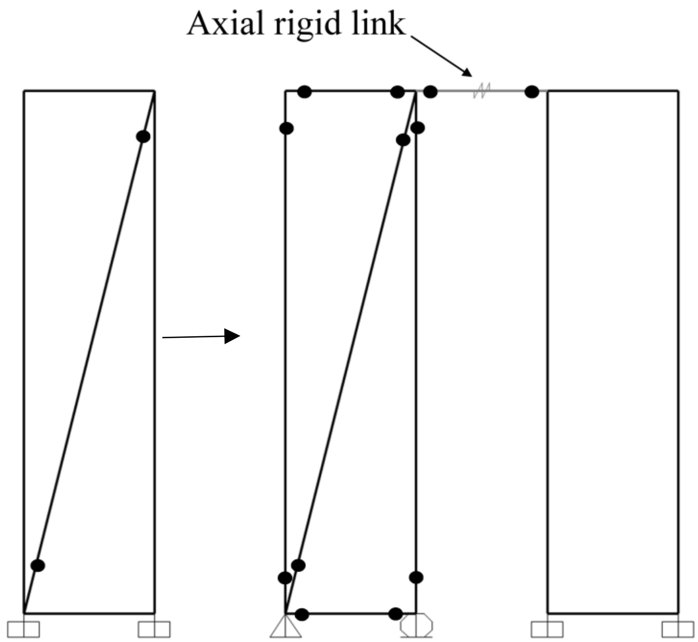

Figure 10: Truss and frame action decomposition of a narrow strap-braced wall

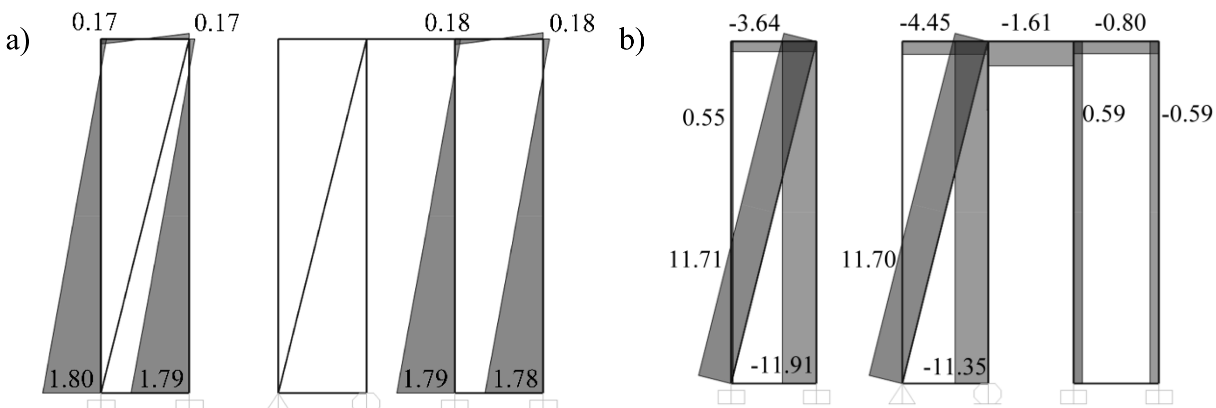

Figure 11: a) Internal moments of the 4:1 aspect ratio system compared to the linked system (kN-m), b) Axial force diagram of the 4:1 aspect ratio and linked system (kN)

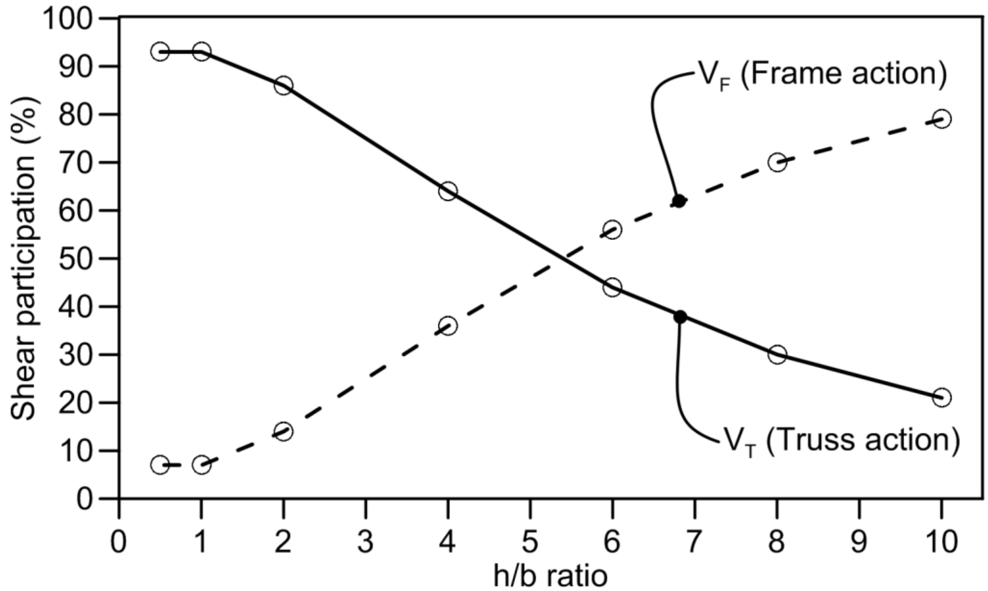

Figure 12: Effect of the wall aspect ratio on the shear participation of the decomposed truss and frame action

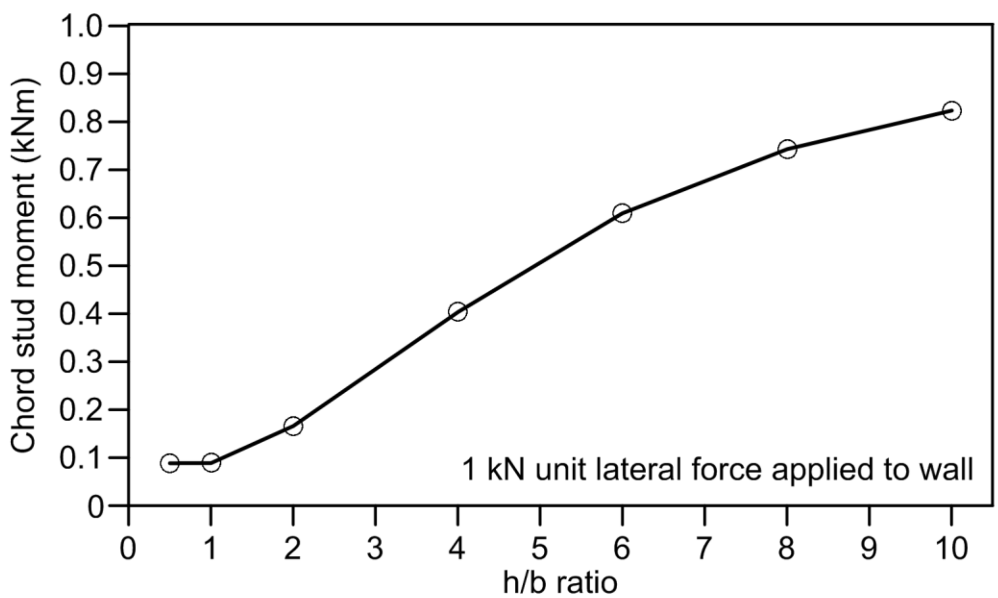

Figure 13: Effect of the wall aspect ratio on the maximum moment developed in chord stud

a)  b) 

Figure 14: Chord stud damage observed during testing of 2:1 aspect ratio strap-braced wall [10]; a) General wall displacement, b) Chord stud flexure and elastic distortional buckling

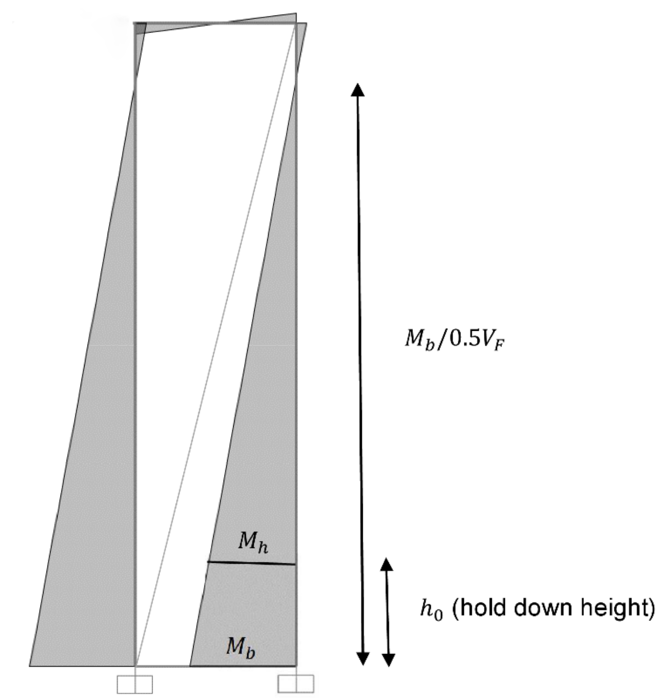

Figure 15: Moment at base and directly above hold-down

Table 1- Matrix of 610x2440 mm (4:1 aspect ratio) strap-braced wall test specimens

Specimen properties	**Medium**	**Heavy**	**Medium**
	Veld connected braces	***Weld connected braces***	***Screw connected braces***
	19B-M1, 19B-M2 20B-C	23C-M1, 23C-M2 24C-C	49A-M
Strap bracing[a] *(cross brace on both sides of wall)*			
Thickness (mm)	1.37	1.73	1.37
Width (mm)	69.9	101.6	69.9
Grade (MPa)	340	340	340
Chord studs[a] *(double studs screwed together back-to-back)*			
Thickness (mm)	1.37	1.73	1.73
Dimensions (mm)	152x41x12.7	152x41x12.7	152x41x12.7
Grade (MPa)	340	340	340
Interior[a] *studs*			
Thickness (mm)	1.09	1.09	1.09
Dimensions (mm)	152x41x12.7	152x41x12.7	152x41x12.7
Grade (MPa)	230	230	230
Tracks[a]			
Thickness (mm)	1.37	1.73	1.37
Dimensions (mm)	152x31.8	152x31.8	152x31.8
Grade (MPa)	340	340	340
Gusset plates[a]			
Thickness (mm)	1.37	1.73	1.37
Dimensions (mm)	152x152	203x203	178x229
Grade (MPa)	340	340	340
Nominal axial compression resistance of chord studs using AISI S100 [12]			
Full composite action & web holes not considered (kN)	121.0	163.3	163.3
Expected force in SFRS due to brace yielding AISI S213 [1]			
$R_yA_gF_y$ Single Brace (kN)	35.8	65.7	35.8
Total Horizontal Force[b] (kN)	17.4	31.9	17.4
Total Vertical Force[b] (kN)	69.5	127.5	69.5

[a]Nominal dimensions and material properties,

[b]Total force based on expected capacity of two tension braces

Table 2: Proposed method interaction ratios for experimental work of Comeau and Rogers [10]

(a) details

Test ID	h/b	V_y (kN)	V_{yp} (kN)	$\frac{V_y}{V_{yp}}$	Eq. 14 check at V_y	Eq. 15 check at V_y	V_{yPM} (kN)	V_n (kN)	$\frac{V_y}{V_n}$
13A-M (1:1)	1	32.98	33.77	0.98	0.57	0.76	44.65	33.77	0.98
14A-C (1:1)	1	36.59	33.77	1.08	0.57	0.76	44.65	33.77	1.08
15A-M (1:1)	1	31.05	33.77	0.92	0.57	0.76	44.38	33.77	0.92
16A-C (1:1)	1	36.29	33.77	1.07	0.57	0.76	44.38	33.77	1.07
15B-M (2:1)	2	20.22	21.36	0.95	0.74	1.00	21.32	21.32	0.95
16B-C (2:1)	2	22.11	21.36	1.04	0.74	1.00	21.32	21.32	1.04
17A-M (1:1)	1	55.66	50.65	1.10	0.48	0.60	84.23	50.65	1.10
18A-C (1:1)	1	62.04	50.65	1.22	0.48	0.60	84.23	50.65	1.22
19B-M (4:1)	4	18.11	17.37	1.04	0.73	1.09	15.95	15.95	1.14
20B-C (4:1)	4	19.46	17.37	1.12	0.73	1.09	15.95	15.95	1.22
19A-M (1:1)	1	56.66	50.65	1.12	0.48	0.61	83.68	50.65	1.12
20A-C (1:1)	1	64.27	50.65	1.27	0.48	0.61	83.68	50.65	1.27
21A-M (1:1)	1	92.68	92.97	1.00	0.63	0.69	134.39	92.97	1.00
22A-C (1:1)	1	104.12	92.97	1.12	0.63	0.69	134.39	92.97	1.12
23B-M (2:1)	2	55.71	58.80	0.95	0.81	0.91	64.70	58.8	0.95
24B-C (2:1)	2	60.57	58.80	1.03	0.81	0.91	64.70	58.8	1.03
23C-M (4:1)	4	27.83	31.89	0.87	0.96	1.24	25.82	25.82	1.08
24C-C (4:1)	4	23.76	31.89	0.75	0.96	1.24	25.82	25.82	0.92
23A-M (1:1)	1	93.07	92.97	1.00	0.63	0.70	133.25	92.97	1.00
24A-C (1:1)	1	103.38	92.97	1.11	0.63	0.70	133.25	92.97	1.11
49A-M (4:1)	4	19.57	17.37	1.13	0.55	0.85	19.98	17.37	1.13

(b) summary

		$\frac{V_y}{V_{yp}}$	$\frac{V_y}{V_n}$
All walls	mean	1.04	1.07
	st. dev.	0.12	0.10
Walls where strap yielding controlled the predicted loads	mean	1.07	1.07
	st. dev.	0.10	0.10
Walls where the proposed interaction equations controlled the predicted loads	mean	0.96	1.06
	st. dev.	0.13	0.11