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Better connection details for strap-braced CFS stud walls in seismic regions

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ABSTRACT

Buildings composed of cold-formed steel (CFS) framing systems have become popular in many parts of the world. Light-gauge CFS frames can be built up from C-shaped sections and laterally braced with tension-only thin strap members. Recent research has raised many concerns about the lateral performance of these light wall systems in regions of high seismic activity. This paper focuses on the failure modes of different systems and on the main factors contributing to the ductile response of the CFS walls. The aim is to ensure that the diagonal straps yield and respond plastically with a significant drift, thereby preventing any risk of brittle failure such as connection failure or column buckling. Several superior connection details are then proposed, which provide a reliable lateral performance even in large lateral deformations by using perforated straps and/or bracket members in four corners of the wall. Performance of the proposed systems is evaluated by experimental tests on full-scale 2.4 m × 2.4 m and 1.2 m × 2.4 m specimens under a particular cyclic load regime.

Method B of ASTM E2126-05 standard was selected as lateral loading regime, which introduces a progressively increasing cyclic loading with two stabilized cycles at each displacement amplitude. In order to maximize the precision of the result, each specimen was connected to the testing rig by high strength bolt. The tests showed that all of the new proposed systems can provide reliable response with good ductility and maintain their lateral and vertical load-bearing abilities up to a relative lateral displacements of 3.3%, which was the limit of the testing rig.

1. Introduction

Cold-formed steel (CFS) structures offer a light-weight and inexpensive alternative to traditional construction techniques especially for low rise, lowest or two-story residential buildings. Although CFS walls are not new and have been used as non-structural partitions for many decades [2], their application as main structural load-bearing components of frames is fairly new. As a result, guidelines that address the seismic design of CFS structures are not yet fully developed and the design of these systems is not yet covered in detail in standards.

Among existing codes and standards, NEHRP recommendation (FEMA 450) [1] specifies that the connection for diagonal bracing members and boundary members (chord members) shall have a design strength greater than or equal to the nominal tensile strength of the strap-bracing members. This recommendation also states that the pull-out resistance of screws shall not be used to resist seismic load because it does not allow the straps to develop their full tensile capacity (which is vital for the system's ductile performance in high seismic events). In addition, according to NEHRP, diagonal braces and studs or chords supporting the brace force should be anchored such that bottom and top tracks are not required to resist uplift forces by bending of the track or track web. As well, both flanges of studs should be braced to prevent lateral–torsional buckling. The code also limits the low-rise story drift ratio to 2.5%, 2.0% and 1.5% for seismic use groups I, II and III, respectively.

The US Army Corps of Engineers has published TI 809-07 [2], which provides similar but more stringent regulations for the design of strap-braced CFS stud walls, in which straps are connected only to the frame's exterior corners and where chords are constructed from box sections. Other codes, such as ASCE 7 [3] and IBC [4], refer to AISI standards [5–9] for lateral design which are essentially similar to FEMA 450 [1], with no special guidelines regarding the design of new systems nor any distinction between different types of strap-braced systems.

Ideally, strap braces should be able to maintain their yield capacity over extended lateral inter-story inelastic drift without any brittle failures, such as connection failure (strap-to-frame or track-to-chord), lateral–torsional buckling of studs or hold-down failure. However, most past research found poor lateral performance for this system under progressive cyclic loading. Mostly, one of the aforementioned undesirable failure modes happened before the system could accommodate the load of fully yielded

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strap. These undesirable modes of failure prevent the strap from reaching or maintaining the yielding stress and therefore considerably reduce the ductility and energy dissipation capacity of the system.

One aim of the current research was to introduce new strap-bracing systems and detailing those systems that comply with codes provisions and satisfy ductility criteria. Another objective of this research was to investigate the effect of the chord members' cross section on lateral seismic performance of the wall. Scrutinizing the obtained results, the main reasons for the past poor performances have been found. Interestingly, all codes and standards currently introduce only one type of strap bracing: that is connecting the straps to the four exterior corners. Having more research done on the proposed systems, new systems along with appropriate design guidelines can be introduced into the design codes and standards. In order to reach these goals, several new systems for strap bracing have been introduced and their performances were examined by means of full-scale experimental testing. These performances, at the end, are compared together and against the past studies results.

The walls studied here are unlined despite the positive effects of gypsum board on the lateral performance under cyclic loading. Post-earthquake observations of the wood-frame structures in the Northridge earthquake have shown that many gypsum board shear walls failed under imposed dynamic load [12]. As a result, some design codes [2] have recommended neglecting the gypsum board contribution and relying only on the bare steel structure.

2. Literature review

Previous experimental studies on the lateral performance of strap-braced CFS stud walls are reviewed below.

Adham et al. [11] provided five cyclic loading tests of a $2.44\,\mathrm{m} \times 2.44\,\mathrm{m}$ CFS shear panel with back-to-back double studs at the ends which were sheathed with diagonal straps and gypsum board. Two hold-downs, one at each end, were bolted to the testing rig at the base to prevent the specimens from horizontal slide or uplift at the toes. Straps were connected to the CFS frame by over-designed gusset plates to ensure that no failure in the connection would occur. The diagonal straps were being identified as the most important components, and the focus of study was mostly on the effect of different strap sizes. Most walls were constructed from 16 mm gypsum board on both sides along with one-side X-strap. Each specimen was simultaneously subjected to a constant vertical load and a cyclic lateral load. Most walls reached yield at about 0.6% of lateral inter-story drift. In specimens with strong strap bracing, local buckling combined with crushing of the top chords and the tracks that were attached to them were observed at lateral drift ratio larger than 0.8%; however, no failure was observed at the strap-to-frame connections (gusset plates). Tests revealed that by increasing the strap cross-sectional area, load capacity of the panel increases and its deflection reduces. Also it was seen that upon buckling occurring in the top corner of the stud, the load resistance capacity reduced significantly. However, by preventing buckling and connection failure at the design stage, the system could be effective in dissipating energy especially in the first cycle of each displacement amplitude. The results showed that walls exhibit higher resistance in pull direction (in contrast with opposite direction or push direction) and in the first cycle displacement amplitude (in contrast with stabilized cycles). The maximum lateral inter-story drifts measured in this study was less than 1.2% (except for one test with a 1.4% drift), which is much less than the permissible design lateral drift allowed by design codes.

Serrette and Ogunfunmi [12] investigated the performance of $2.44\,\mathrm{m} \times 2.44\,\mathrm{m}$ strap-braced CFS wall studs under a quasi-monotonic lateral load control regime. Each frame was constructed with three studs and two double section back-to-back chord members. Among eight specimens, three were tested by one-side strap bracing, four by one-side strap bracing and two-side gypsum board, and one by two-side strap bracing and gypsum boards. Over-design gusset plates were used to connect the strap to the exterior corners of the frame. Each frame was connected to the testing rigs by means of two hold-downs at the bottom of the chord studs and bolts at the end studs and chords. The walls with one-side strap bracing failed at lateral displacement of 2% while the walls with strap bracing and two-side gypsum board failed at lateral displacement less than 1.7% because the failure mode was governed by the failure of the gypsum wallboard. They concluded that the contribution of studs in shear load resistance would be relatively negligible in comparison to sheathing or strap bracing. Also in the wall with strap bracing and gypsum board, the effect of strap bracing on lateral stiffness is small; however, the presence of the strap resulted in a reduction in ultimate deflection (or reduction in ductility) and an increase in the range of 20% in the shear load capacity of the system. The study proved that the effect of gypsum board on lateral performance of the system is very significant. For the specimen with two-side strap bracing and gypsum board, the same response was observed, i.e. higher shear resistance and lower ultimate lateral deformation (or ductility).

Gad et al. [13,14] investigated the seismic performance of strap-braced CFS wall studs with or without gypsum board cladding by means of experimentation (cyclic and shake table testing) and numerical studies. The experimental study consisted of cyclic tests on strap-braced walls with and without gypsum board cladding in addition to shake table tests on a three-dimensional single story CFS box-shaped structure. All tests were performed on the wall with single section chord members as opposed to most of the other studies which took advantage of double section back-to-back or front-to-front chord members. The results of tests on the unlined single wall under cyclic loading showed failure (under shear) of one screw, followed by pull-out of the rest of the screws connecting the straps to the frame corner prior to the strap reaching yield. For this wall under earthquake loading, yielding of the straps and significant yielding at brace fasteners were observed. The same test was repeated for the wall with welded stud connections but was stopped by the tearing of the strap. They concluded that the type of connection (rivet or weld) does not have any influence on the lateral response of the strap-braced wall. Several different tests were performed for the one-story box-shaped structure under cyclic and earthquake loading (shaker table test). In all tests a brittle failure such as net section failure (tearing) of the strap at the tension unit location or strap-to-frame connection, or tearing of the plaster board around the screws was observed. They concluded that non-structural components make a significant contribution to the lateral bracing of the frame. It seems, however, that if the system had been designed to withstand the yielding force of straps, a different conclusion might have been reached. Gad et al. reported that for strap-braced lined frames, the overall stiffness and strength of the system are equal to the sum of the individual contributions from plaster board and strap bracing. Also they concluded that for unlined frames, the failure mechanism is governed by tearing of the strap at the strap-to-frame connection or at the tension unit location.

Tian et al. [15] conducted experimental and analytical studies on the racking resistance and stiffness of CFS walls. Three different strap-braced stud walls of $2.45\,\mathrm{m} \times 1.25\,\mathrm{m}$ dimension with single section chord members were tested under a monotonic loading regime. Both one- and two-side strap-bracing schemes connected to the frame corners by rivets were tested. For two-side strap bracing, they reported the failure of studs under overall and heavy local buckling, and failure of the rivets connecting straps to the frame corners. For the one-side strap bracing, failure of the rivets at the strap-to-frame connection was observed, along with distortional buckling in studs at an earlier stage of the test than for two-side strapping. Based on the results, they concluded that the cross-sectional area of a strap significantly affects the deflection and stiffness of a frame, but has little influence on its racking load capacity. Similar to the majority of previous studies, this conclusion is based on the racking resistance results of inadequately designed walls that do not accommodate the strap yield, but rather fail prematurely due to incompetent connection detailing.

Fulop and Dubina [16] tested three double-sided X-strap braced wall panels with an aspect ratio of 1.5 (3.6 m length $\times$ 2.44 m height) under monotonic and cyclic loading. The chord member was made of a double stud, and care was exercised to avoid strap-to-frame connection failure in order to facilitate yielding of the strap. Although corners were further restrained using a U profile in the track to provide more capacity and rigidity, damage was concentrated entirely in the lower corners of the panels. Some sign of connection elongation and redistribution of load to the second and third studs was also observed. Despite large elongations in the straps, the results of this research may not be a true representation of lateral displacement of strap-braced walls because failure at the corners increased the lateral drift of the wall. The hysteresis curves show that the maximum lateral load resistance capacity is at a lateral drift ratio of about 1%, after which a continuous but stable reduction occurs in the total shear resistance, reflecting the local failures at bottom corners.

Pastor and Rodríguez-Ferran [17] presented a hysteresis model for strap-braced CFS wall studs that are properly designed for seismic loading, i.e. frame detailing to allow sufficient yielding of straps prior to any failure at connections or any buckling. This model captures the most important characteristics of these wall types, including stiffness degradation and pinching of the load–displacement hysteresis loops.

Kim et al. [18] performed a shaker table test on a full-scale two-story one-bay CFS shear panel structure. Each story consisted of two identical shear walls of 2.8 m length and 3.0 m height separated from each other by 3.9 m centre to centre. The two chords were constructed from a box section, welded to steel anchors and bolted to the slab through top and bottom tracks. A heavy square RC slab, along with additional mass, was placed at the top of each floor level to simulate actual mass of the system. As the second story frame was identical to the first story, the damage mostly occurred in the first story, as expected. No pre-tensioning was applied to the tension-only straps in spite of the explicit recommendation in the codes [1,2]. The structure then was loaded to a normalized accelerogram, which possessed spectral response acceleration equal to the design response spectrum around the fundamental period of the test specimen. The test caused significant yielding in the form of severe non-linear behavior in the first floor straps along their entire length, and yielding of studs near the anchors. The studs did not develop full flexural strength due to local buckling and anchor deformation which impaired the potential contribution of the frame to the story shear resistance. The results showed that during the large amplitude tests, the X-strap bracing showed very ductile but highly pinched hysteretic behavior.

Al-Kharat and Rogers [19] investigated the inelastic performance of 16 strap-braced 2.44 m $\times$ 2.44 m CFS wall studs with double section stitch welded front-to-front chord members under cyclic loading regime. Three different strap configurations were tested: light, medium and heavy strap-braced walls. The walls were braced with diagonal flat straps on both sides. Four L-shaped hold-downs were welded to four corners of the light wall frames at the interior face of the chord studs and then bolted to the test frame, while for the medium and heavy wall types, flat plate (instead of L shape) hold-downs were placed within the upper and lower tracks at the four corner locations. Unfortunately the chosen detailing for the hold-downs of medium and heavy frames were inconsistent with the expected function of hold-downs which should transfer the chord stud forces to the support. For the light walls, strain gauges showed yielding of the strap but it was combined with the progressive compression failure of the track and/or failure of the chord-to-track connection similar to the failure reported by Fulop and Dubina [16]. Tearing of the strap and fracture of the anchor bolts were also reported. For the medium wall size, yielding of the straps occurred in some tests. However, the frames were not able to maintain strap yielding force due to extensive damage in the track and gusset plates adjacent to hold-downs, triggered by punching shear failure around the hold-down location. Accordingly, heavy wall types were not able to demonstrate yielding of the straps because of extensive damage to the frame and gusset plate area adjacent to the hold-down plates followed by punching shear failure of the track and pull-out of the screws connecting the interior studs to the bottom track. In all walls, the system reached maximum shear resistance at an inter-story displacement ratio of 2%. The shear resistance then reduced as lateral drifts increased further, showing local failure at hold-down location. The main failure mode in heavily braced walls was hold-down failure, while other types of failure (such as strap tearing, track buckling and track-to-chord connection failure) occurred in the lightly braced walls. These failure modes are all undesirable as they do not allow straps to reach yield and can be avoided by proper detailing.

As can be seen, in all of the past studies, just one system of strap bracing has been tested: connecting the strap to the four exterior corners of the wall. Further, most past research has reported poor ductility and in some cases contradictory results for this system. The current research is aimed at introducing more efficient and innovative detailing for strap bracing. Moreover, comparing the current results with the results of past researchers, the reasons of poor past results are explained.

3. Test setup

3.1. Testing rig and instrumentation

The configuration of the testing rig is shown in Fig. 1. Each specimen was fixed to the base beam by means of five M16 high strength bolts in the vicinity of the middle stud and chords, which were tightened by a torque wrench to a torque of about 190 N m that is corresponding to about 53 kN tension in the bolt. Between a bolt head and the base beam and a nut surface and the track, two glossed 50 mm $\times$ 50 mm washers were placed to increase the contact surface and friction and to reduce the slip possibility between the bottom track and the base beam. A similar arrangement was implemented to connect the top track to the loading beam, but with three M16 bolts connected in the vicinity of the chords and middle stud. Moreover, to reduce the possibility of overturning and to provide a proper load path from the strap to the wall supports, four hold-down angles were placed near the interior surfaces of studs adjacent to the top and bottom tracks as shown in Fig. 1.

Displacement transducers were used to measure the horizontal displacement of the top track (DH1 and DH2) and to measure the amount of imposed displacement and slip between the top

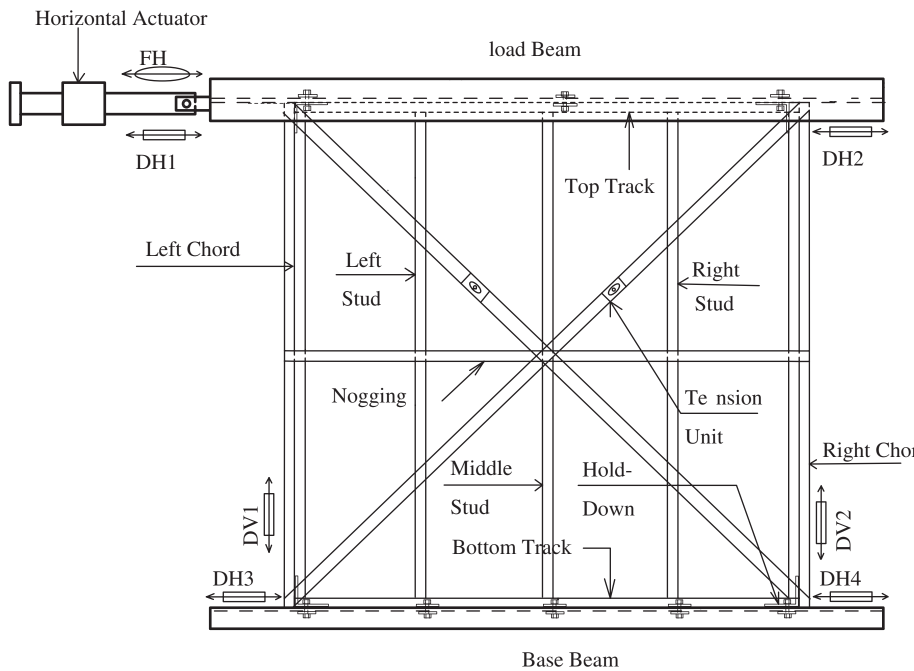


Fig. 1. A diagram of the testing rig.

track and the load beam. Two transducers were also installed at the bottom track (DH3 and DH4) to measure the amount of slip between the bottom track and the base beam. To evaluate the amount of uplift, two transducers were placed at the bottom of the chords. Also a load cell (FH) was used to measure the racking resistance.

3.2. Loading protocol

Cyclic loading methodology followed method B of ASTM E2126-05 standard [10], which was originally developed for ISO (International Organization for Standardization) Standard 16670. In the current study, the loading regime consisted of three full cycles of 1.5, 3, 4.5, 6, 9, 12, 18, 24, 36, 42, 48, 54, 60, 66 and 72 mm, unless failure or a significant decrease in the load resistance occurred earlier. The mentioned lateral amplitudes correspond to 0.0625%, 0.125%, 0.1875%, 0.25%, 0.375%, 0.5%, 0.75%, 1.00%, 1.50%, 1.72%, 2.00%, 2.25%, 2.25%, 2.50%, 2.75% and 3.00% of inter-story drift. At least two stabilizing cycles are needed for the lateral displacement amplitudes greater than 1%, as proposed by ISO. This was adapted in the current study, because many different types of failure such as failure in connection, tearing of straps and pull-out of screws were observed in the stabilizing cycles. Although the 75 mm or 3.125% inter-story drift ratio was the maximum amplitude of the actuator, it was considered adequate since the maximum allowable story drift ratio specified by FEMA 450 is 2.5% [1]. The loading velocity was 3 min/cycle or about 0.8 mm/s. ASTM E2126-05 recommends a loading velocity within the range of 1–63 mm/s (0.8 mm/s here is close to the lower end of the range).

4. Experimental program

4.1. Member size

The program consisted of nine full-scale specimens to evaluate the performance of four different strap-braced walls as shown in Figs. 2–5. These walls, which were donated by QuickFrame Technologies, were tested in the Structural Engineering Laboratories of the University of Queensland in a specially made testing rig.

All of the frame components, i.e. top and bottom tracks, noggings and studs, were identical C channels of $90 \times 36 \times 0.55$, connected together by one rivet at each flange. Back-to-back double sections were constructed by connecting the web of the two sections together by screws with spacing of 150 mm.

Bracing was implemented by means of $30 \times 0.84\,\text{mm}^2$ straps connected to one or both sides of the frame. To prevent premature tearing of the strap at the strap-to-frame connection or at the location of the tension unit, a perforated strap was used. Straps were fixed to the wall panels by #10 10-16, self-drilling tapping screws. Sufficient screws were used to avoid failure at the strap-to-wall connection (tearing of strap or pull-out/pull-over of the screws) and to allow yielding of the strap.

4.2. Material properties

Mechanical properties of the materials were investigated by tests performed at UQ labs and are provided in Table 1. For the G550 steel, the stress–strain curve did not exhibit a clear yield plateau, so the yield point was found from 0.2% proof stress. The mechanical properties given for the perforated strap represent the properties as a whole including the effect of the holes.

4.3. Specimen types

The first strapping scheme takes advantage of four brackets placed at the four corners of the wall as shown in Fig. 2. Strength, stiffness and ductility of this system depend mostly on the bracket's shape and size, which were examined by specimen CC2 (Fig. 2(a)), and to a lower extent on the chords, which were examined by specimen CD1, as depicted in Fig. 2(b). As it can be seen in the figure, specimen CC2 has a single section as a chord member, while chord members of CD1 are constructed from back-to-back double section.

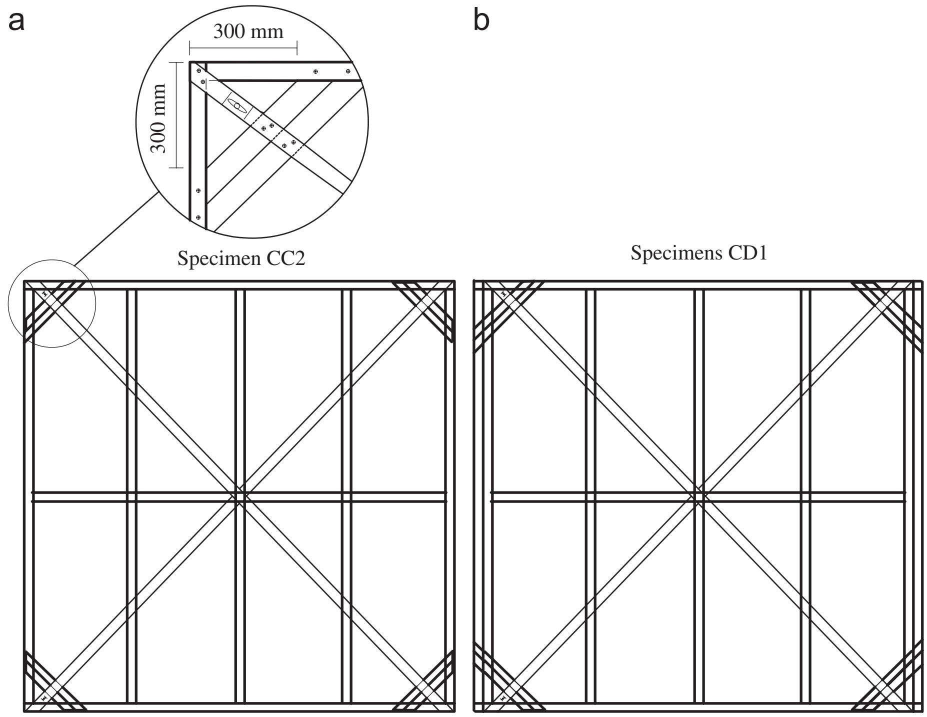


Fig. 2. Specimens CC2 and CD1.

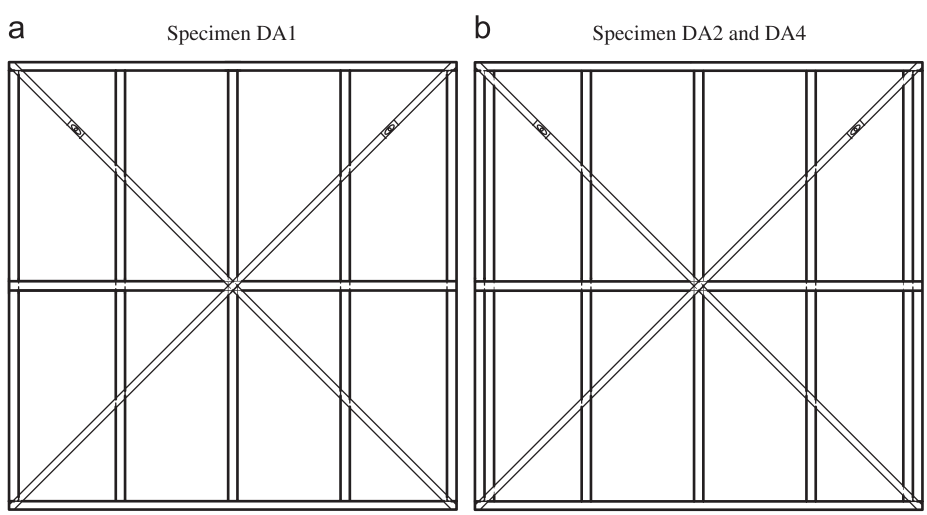


Fig. 3. Specimens DA1, DA2 and DA4.

The second scheme (wall DA1, Fig. 3(a)) investigates direct screw connection of straps to the four outermost corners of the wall panel. The effects of chords and double-side strap bracing on the lateral performance of this type of strapping were investigated by specimens DA2 and DA4, respectively, which are shown in Fig. 3(b). A similar study was conducted for the connection of straps to the interior frame joints, as shown in Figs. 4(a) (wall DB1) and (b) (wall DB4). Moreover, to investigate the effect of continuity of tracks on the lateral performance of strap-braced stud walls, specimen DC3 depicted in Fig. 4(c) was tested.

In all of the above bracing schemes, after the installation of perforated strap bracing, one tension unit per strap was placed and tightened to ensure that the straps were loaded immediately after the racking load was applied. However, this pre-tension is important only in earlier stages of the test (to an inter-story drift ratio of about 0.5%) and after that it has no effect on the lateral performance of the system.

To compare the relative performance of perforated and solid straps, the lateral performance of a strap-braced wall panel with solid strap connected to gusset plates at four corners, as shown in Fig. 5, is investigated. The solid strap was selected so that its gross section is equal to the net section of perforated strap. It is believed that this system is not practical as it may cause unevenness in the wall surface after cladding with gypsum board, due to the thickness of the gusset plate and the screw head. Gusset plates are advantageous as they provide enough room to connect the solid straps to the frame corners eliminating the possibility of tension failure at the net section prior to yielding of the strap. No tension unit was used in this specimen, in order to eliminate the

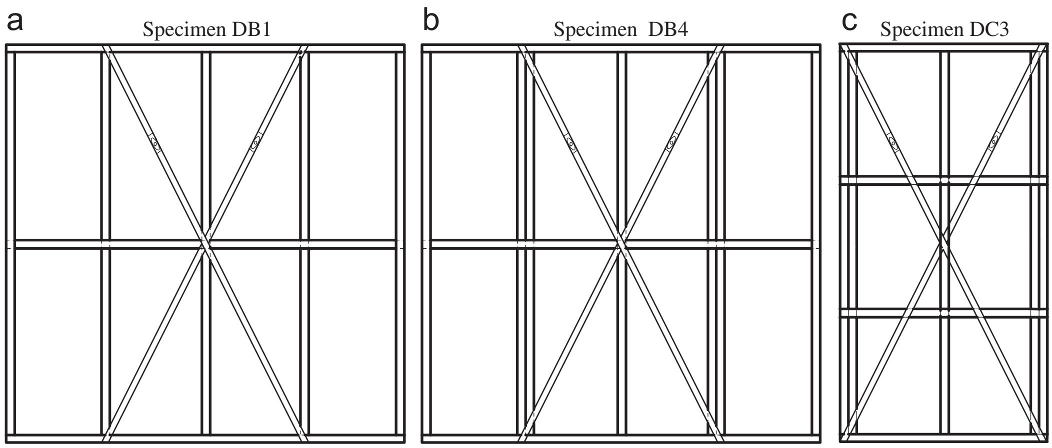


Fig. 4. Specimens DB1, DB4 and DC3.

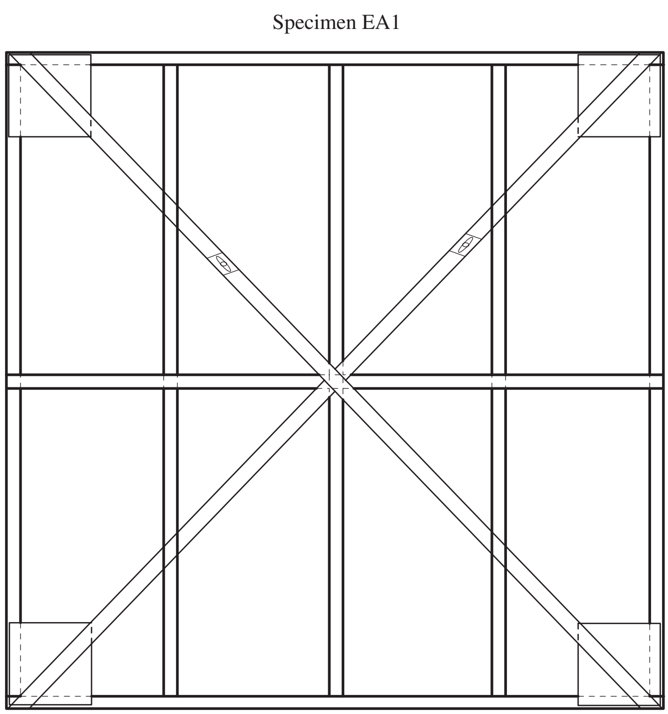


Fig. 5. Specimen EA1.

possibility of net section failure at that location. All the specimens were designed based on FEMA 450 [1] regulations.

5. Experimental results

5.1. Specimens CC2 and CD1

The first specimen, CC2, as depicted in Fig. 2(a), consisted of a wall panel with four double back-to-back stud sections at four corners of the wall. One-side strap bracing was used and ends of the straps were connected to the brackets and the corner of the panel with adequate screws, to allow the full yield capacity of the strap to be developed without any pull-out failure of the screws. The envelope of the first cycle hysteretic loops of the wall is shown in Fig. 7. Perfect performance with no failure was observed up to the end of the test, in addition to a high level of yielding in the strap material. The system showed high stiffness especially at racking displacements less than ± 30 mm (1.25%).

Specimen CD1 (Fig. 2(b)) was prepared to investigate the effect of having double chord members (back-to-back) and the effect of two-side strap bracing. Load–displacement hysteretic loops, a graph of load versus number of cycles and the associated envelope curve for the first cycle are shown in Figs. 6(a), (b) and 7, respectively. The results show that the performance of the wall is very good with no failure anywhere in the panel to the end of the test's available displacement range.

5.2. Specimens DA1, DA2 and DA4

In this scheme, straps were connected to the four exterior corners of the frame. In the first wall panel, DA1, as shown in Fig. 3(a), one-side strapping was used along with a single section chord member. The load beam was connected to the top track only via one bolt near the middle stud. The envelope of load–displacement curve for this wall is shown in Fig. 7. The specimen lost its lateral load resistance capacity completely in the first cycle of +30 mm (+1.25%) and -38 mm (-1.58%) lateral displacement due to distortional buckling of the left chord's upper portion and severe distortional buckling of the top track's right portion adjacent to the right chord. Lateral loads corresponding to the above failures were +3.7 and -4.2 kN. Because straps are at a 45° incline, the induced forces in the left chord and in the top track, which caused them to buckle, were equal to the lateral load resistance at the time of failure, yet both were lower than the buckling capacity of a single stud extracted from experimental tests (7.4 kN) due to a lack of continuity of the tracks at the four corners.

Another wall panel, denoted as DA2 (Fig. 3(b)), with two back-to-back studs as chord members was tested. Straps were connected to both the back-to-back studs. The lateral load resistance behavior (envelope of the load–displacement curve for the first cycle) of this wall panel is shown in Fig. 7. The performance of the wall was good up to the maximum lateral displacement of ± 72 mm (3.0%), and the straps reached yield. This test was repeated and similar results were obtained.

The final test of this series was on DA4 (Fig. 3(b)) which was similar to DA2 but with two-side strap bracing. Load–displacement hysteretic loops, a graph of load versus number of cycles and its associated envelope curve are shown in Figs. 6(c), (d) and 7, respectively. The performance of this wall panel is considered very good, being capable of resisting almost twice the

Table 1
Mechanical properties of the frame member and strap (mean value)

Member	Nominal grade (MPa)	Nominal thickness (mm)	Base metal thickness (mm)	Elastic modulus (GPa)	Yield stress, F_y (MPa)	Yield strain (%)	Ultimate stress, F_u (MPa)	Ultimate strain (%)	F_u/F_y
Solid strap	300	0.85	0.844	248.86	320.64	0.7	391.04	21.9	1.22
Perforated strap	250	0.85	0.844	121.74	244.15	0.5	272.53	3.26	1.12
Cold-formed member	550	0.55	0.55	168.93	592.26	0.45	617.25	2.86	1.04

lateral load of the wall panels with single brace and being able to accommodate strap yield, although admittedly it showed a lower stiffness especially in low- to mid-level lateral deformations.

Although this scheme showed a very good response, earlier tests had shown that higher forces and displacement demand apply to chord-to-track connection in comparison to previous schemes especially after inter-story displacement ratio of 1.5%. Proper accommodation of hold-downs in four corners of the frame is vital for an acceptable response to be obtained in this scheme. Moreover, placing of four C-section cut-offs in the track at the four corners where the straps are connected can help improve the performance of the system. These cut-offs, even in the small lengths equal to 400 mm, when connected to both track's flanges with one screw every 100 mm, can enhance the response of the system significantly.

5.3. Specimens DB1, DB4 and DC3

This scheme has braces similar to the previous type but they are at a steeper angle, as shown in Fig. 4. In this frame type, the left and right chords that are the interior studs to which straps are connected are different from the end studs. For the first test, DB1 depicted in Fig. 4(a), the left and right chords were single studs. Also, similar to specimen DA1, the top track was connected to the load beam by only one bolt adjacent to the middle stud. The envelope of the first cycles is shown in Fig. 8. Although the wall presented a stable performance, a major reduction in the lateral load resistance occurred in the first cycle to +42 mm (+1.75%) and −60 mm (−2.5%) displacements because of the local-distortional buckling in the upper portion of the left and right chords; however, the straps were mostly at yield as evidenced by strain gauges. The wall showed a ductile response with a lower lateral load resistance in comparison to walls DA1 and DA2, attributable to the higher slope of straps. Although the testing configuration was similar to DA1, no lateral–torsional buckling occurred, due to the effect of continuity of the track at the location of the joint where the straps are connected.

To investigate the effect of double section in the left and right chords along with two-side strap bracing, wall panel DB4 of Fig. 4(b) was tested. Results including load–deflection hysteresis loops, graphs of load versus number of cycles, and envelope of the first cycles are shown in Figs. 6(e), (f) and 8, respectively. The figure shows that the wall can provide approximately twice the lateral resistance of one-side strap brace but only at large inter-story drifts. Moreover in wall DB4, no hold-down was provided at load beam location, in order to investigate the effect of track continuity on the hold-down demand at the strap-to-chord connection. Unlike DA1, due to continuity of track, no minor failure was observed in the chord-to-track connection. Hence, the incorporation of hold-downs is absolutely critical when straps are connected to the outer corners of a wall.

To investigate the effect of track continuity on the lateral performance of wall studs, specimen DC3 as depicted in Fig. 4(c) was tested. This wall consisted of half frame (1.2 m $\times$ 2.4 m) with back-to-back double studs as chords and two noggings spaced 800 mm from each other and from the track members. The envelope of the response is shown in Fig. 8. The wall showed very good performance with no visible failure to the end of the test. Comparison of the performance of this wall with that of the previous one shows that back-to-back double stud chords, in which both sections are properly connected to the track, provide full continuity for the chord member. Accordingly, the difference between interior and exterior corners in terms of rotational support for the buckling of the chords is negligible. However, the demand on hold-down in the latter is higher and more critical.

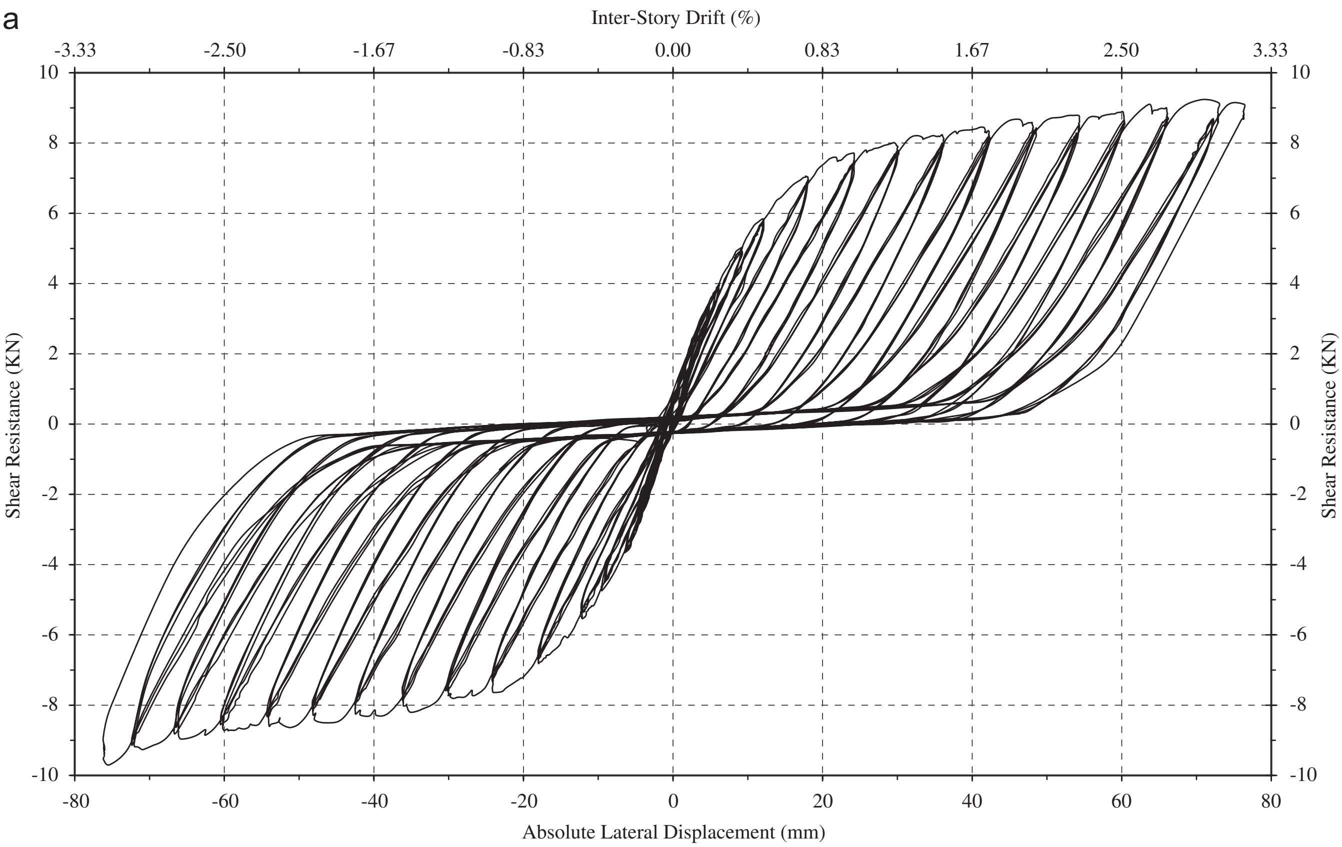
5.4. Specimen EA1

The last strap-bracing scheme examined was wall panel EA1 (Fig. 5). In this system, a gusset plate is used to connect the strap end to the panel. While this type of connection seems the most rational, it is not practical due to the unevenness produced in the gypsum board cladding by the gusset plate and screw head thickness. Because ample room existed for the screws this time, it became possible to load a solid (not perforated) strap to yield, rather than a perforated strap that yields at lower loads. A tension device was not used in order to avoid creating a weak spot along the length.

The hysteretic load–displacement curves, graph of load versus number of cycles and envelope curve are shown in Figs. 6(g), (h) and 8, respectively. These figures show a delay in the up-take of the lateral loads by the strap due to the lack of tensioning unit. The overall lateral performance of the wall panel is very good, especially from the point of view of stiffness and ductility, and this system showed the highest stiffness before yielding. However, for lateral displacements larger than 24 mm (1% inter-story drift), a local-distortional buckling occurred at the upper and lower portions of the tensile chord members near the gusset plate connections, possibly because the gusset plates imposed a rigid connection to the chord flanges forcing compatibility of deformations. A similar mode of failure was reported by Kim et al. [18] and Al-Kharat and Rogers [19]. Although this buckling had no effect on the lateral performance of the wall panel, it did not allow the straps to elongate into the strain hardening range as was evidenced by the strain gauge records.

6. Comparing results of the current study to the past studies

Adham et al. [11] reported differences in the response of his wall panels in push and pull directions. His specimens were all loaded first in push and then in pull direction and he suggested that this method of loading would disturb the virgin state of the panel in the push direction and degrade it to some extent before it is loaded in the pull direction. This slight loss of integrity for the pull direction results in a somewhat lower load-carrying capacity of the panel in this direction. In the current research, although the specimens were loaded in push direction and then in pull direction, no significant difference was observed between the capacities. The reason can be attributed to the fact that in Adham's



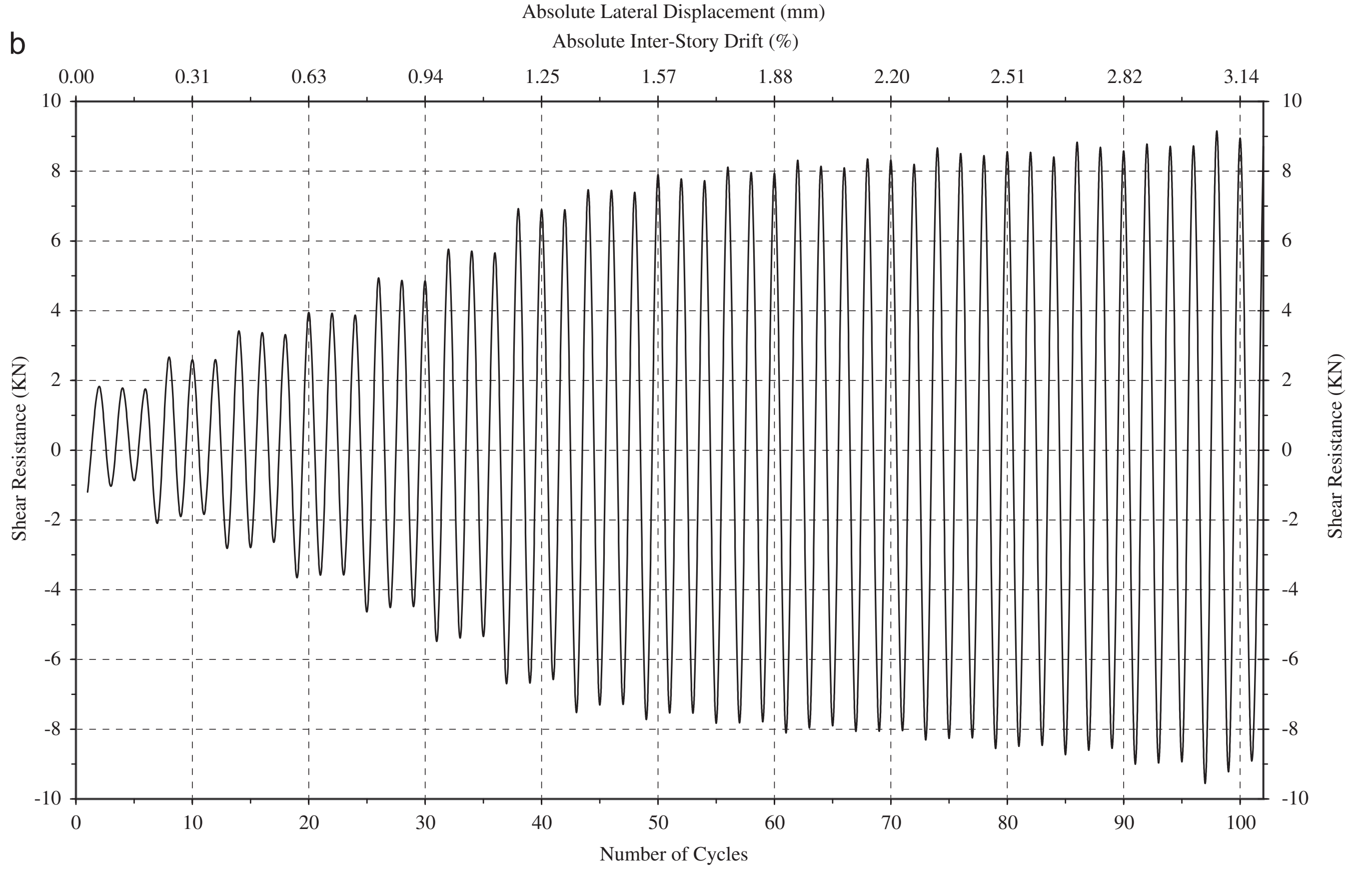
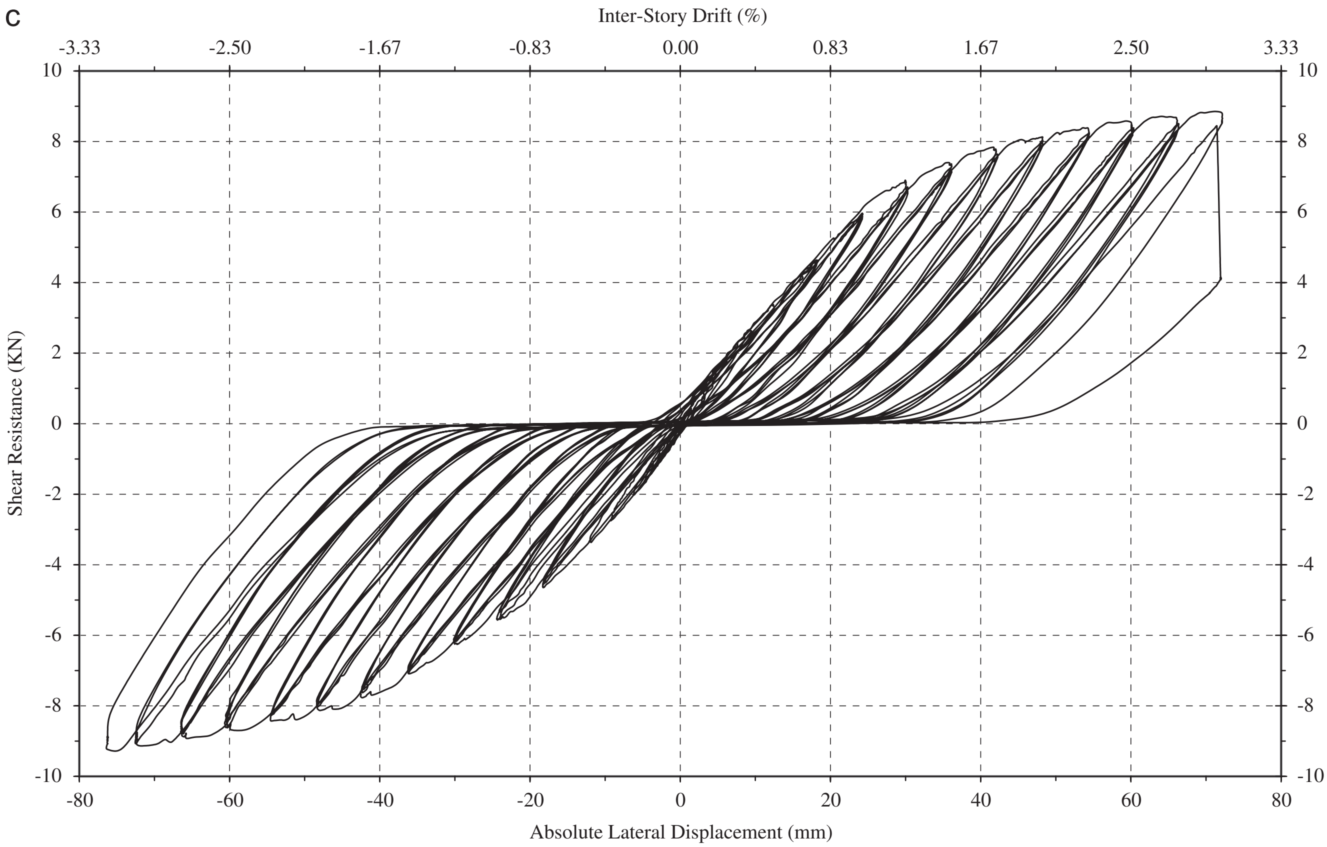


Fig. 6. Load-deflection hysteretic cycles for: (a) wall CD1, (c) wall DA4, (e) wall DB4, (g) wall EA1 and load-cycles diagram for: (b) wall CD1, (d) wall DA4, (f) wall DB4, (h) wall EA1.

c



d

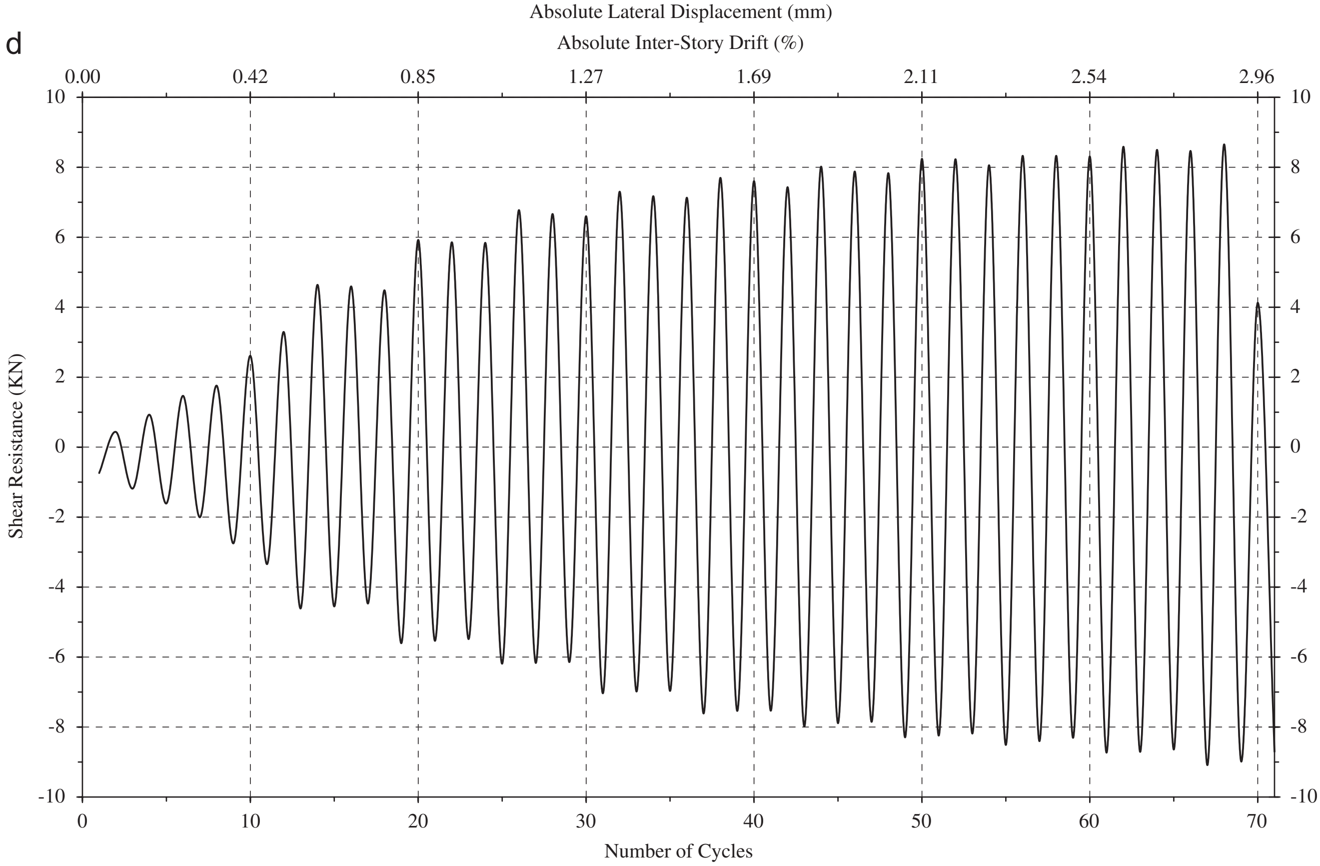
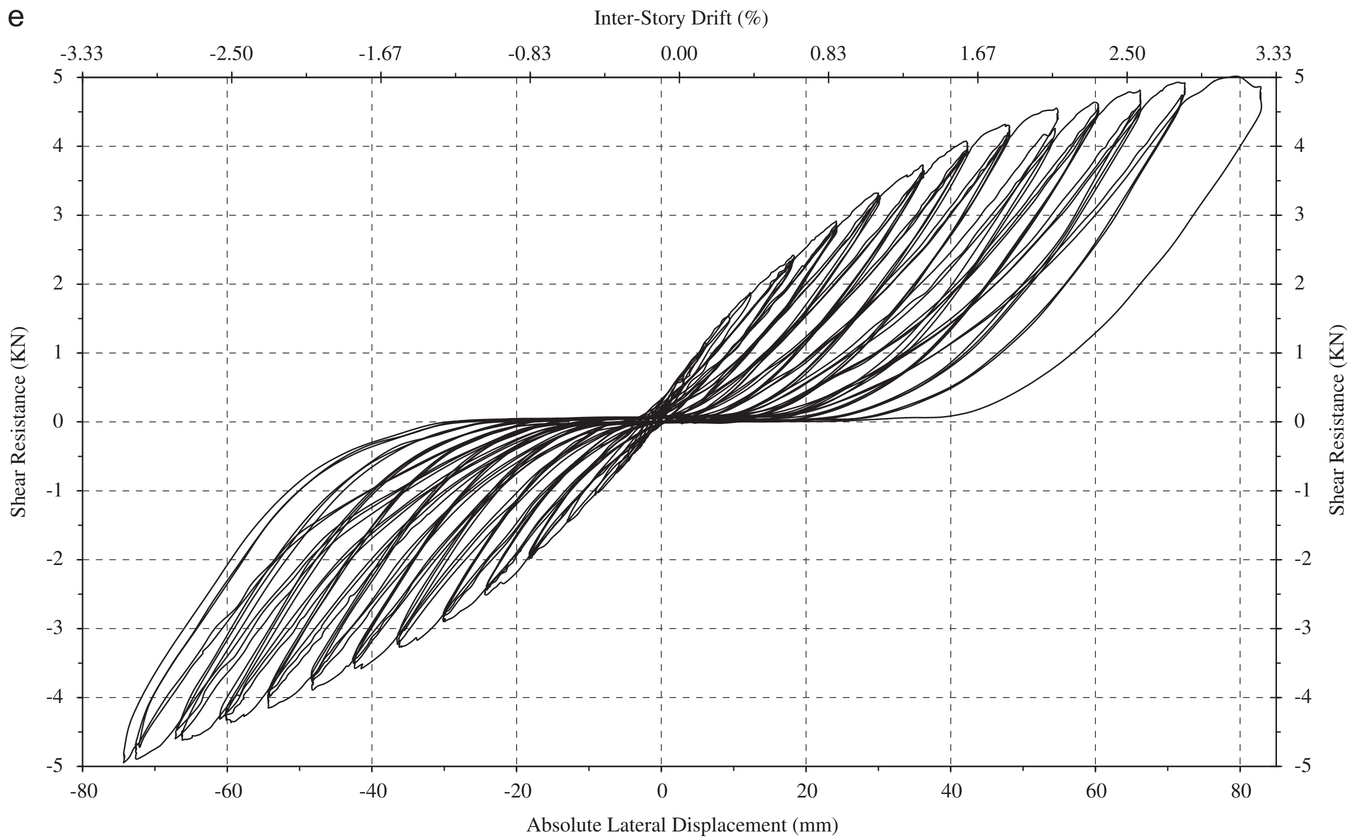


Fig. 6. (*Continued*)



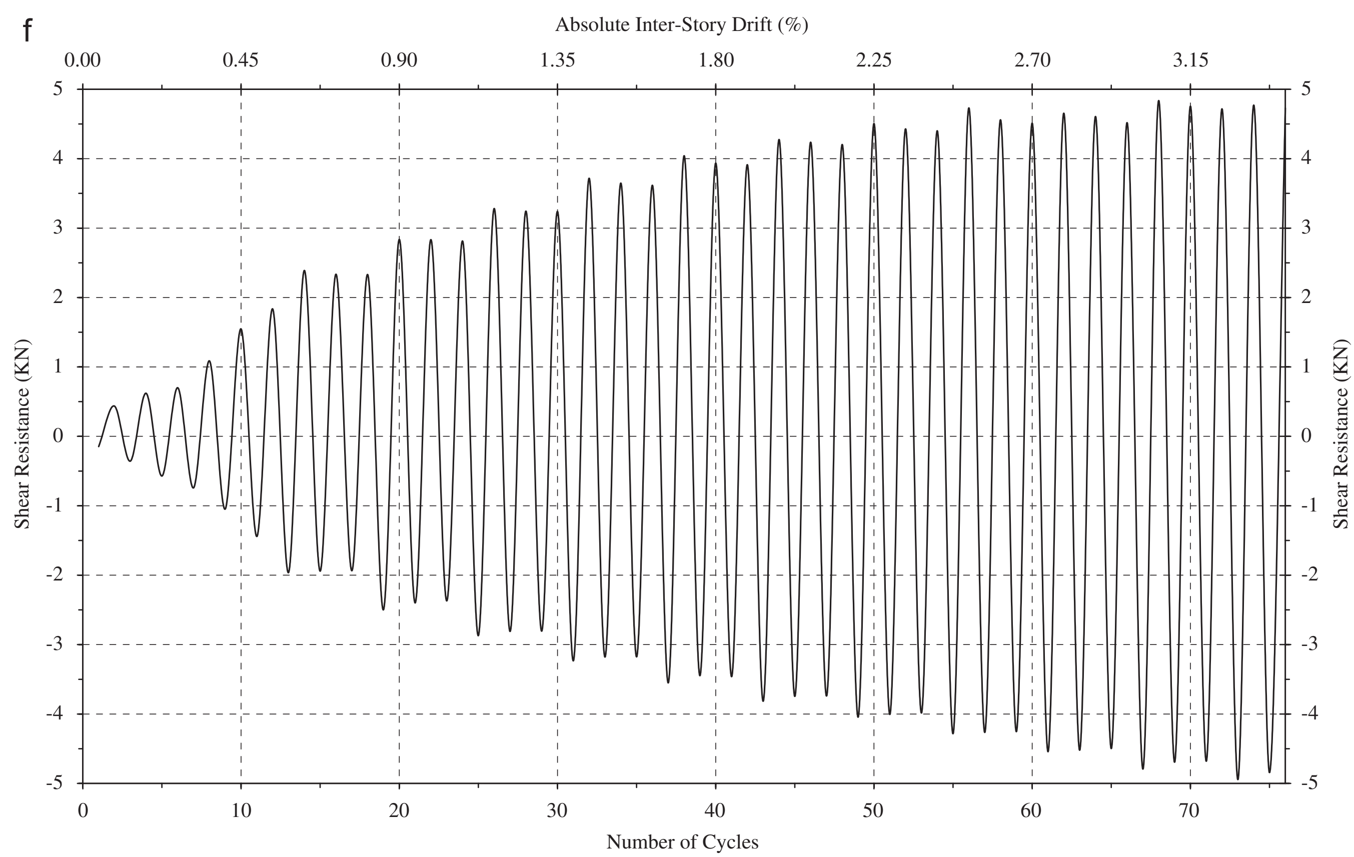
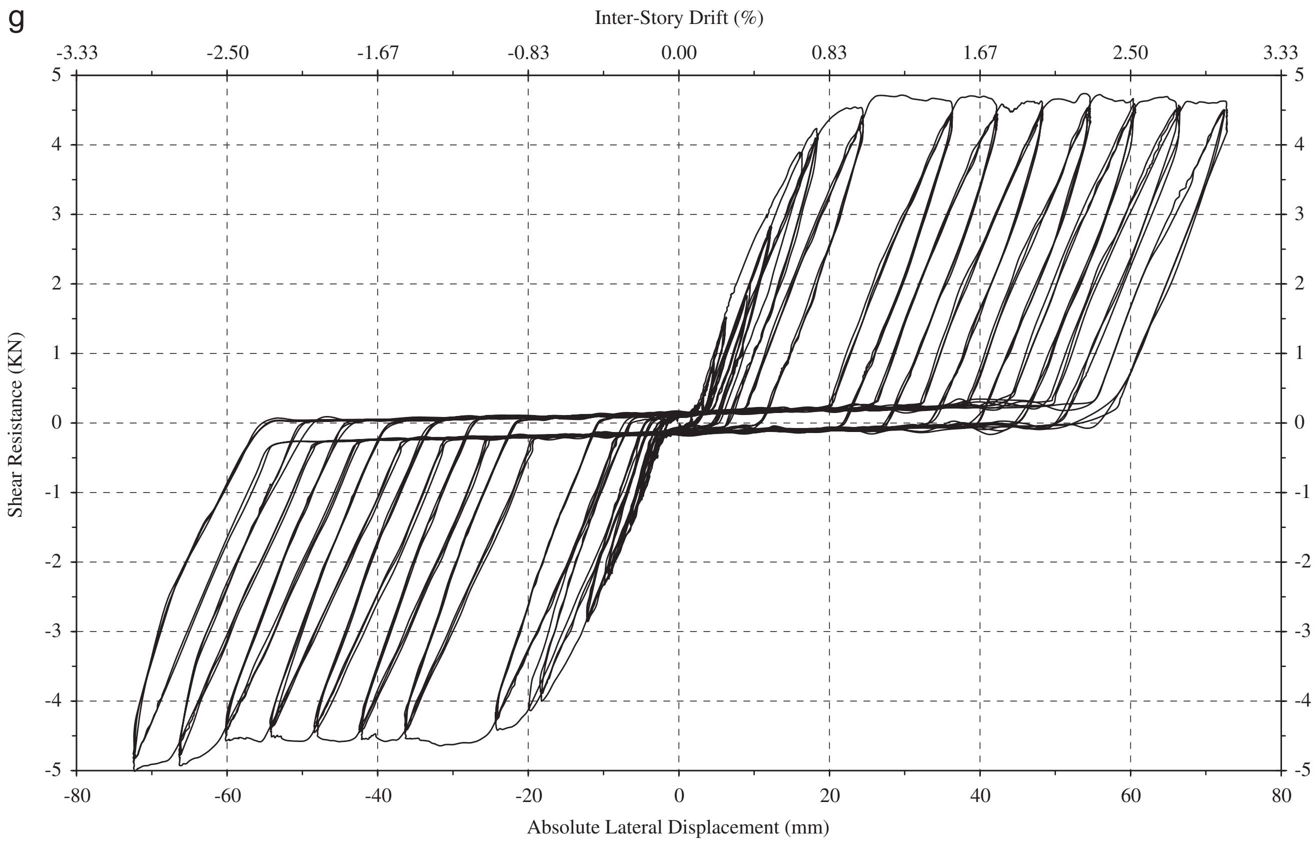


Fig. 6. (*Continued*)

g



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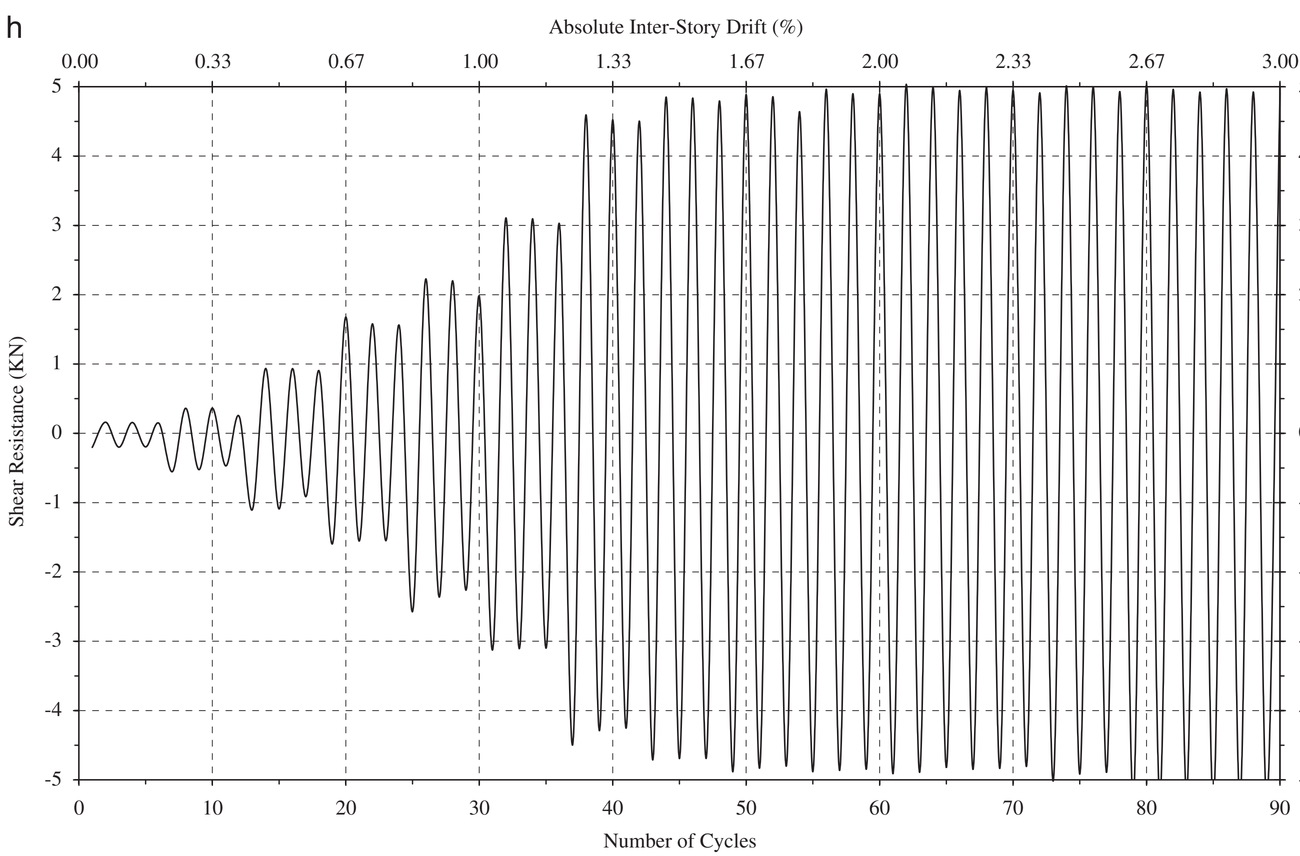

Fig. 6. (*Continued*)

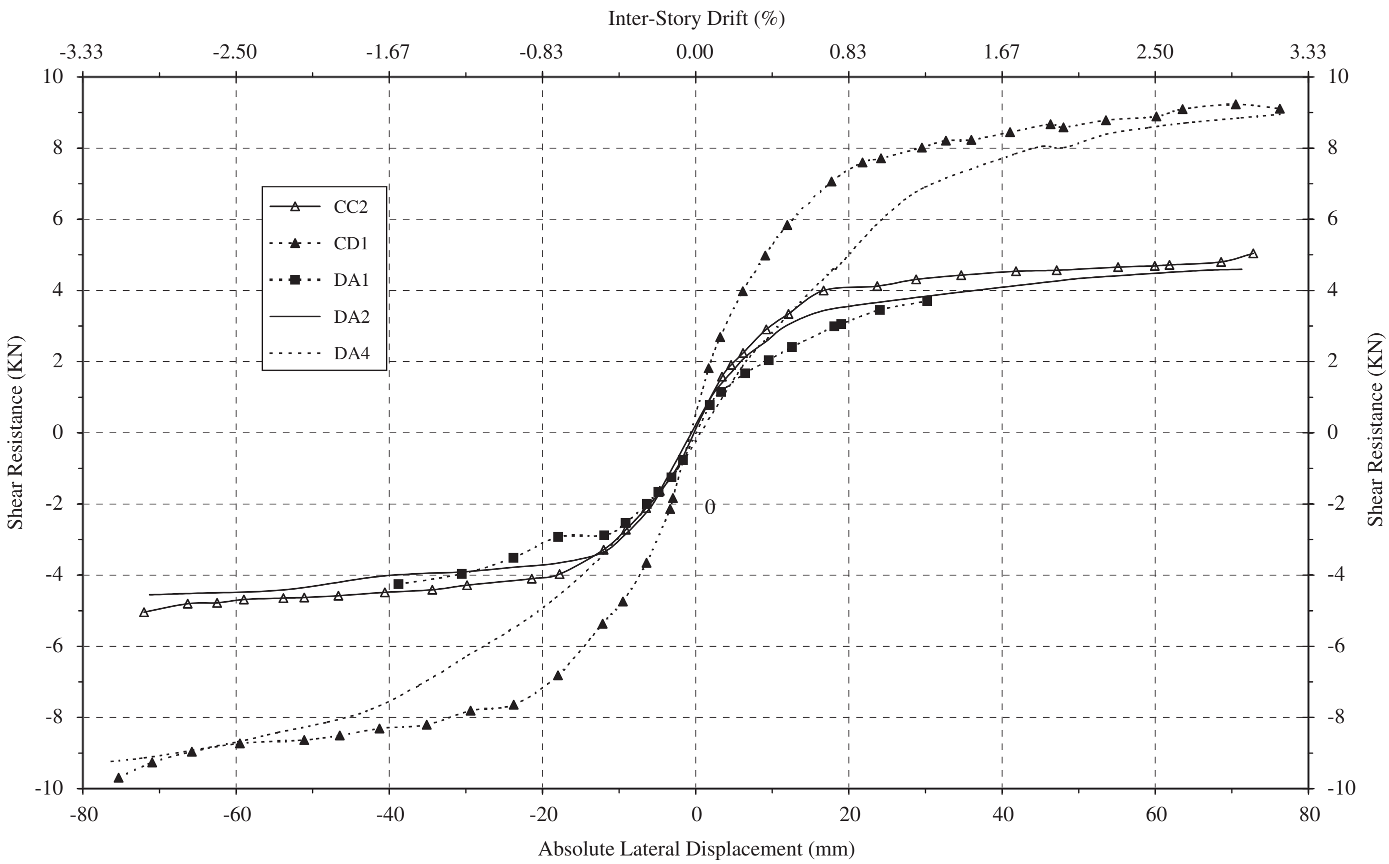

Fig. 7. First cycle load–displacement envelope plots for specimens CC2, CD1, DA1, DA2 and DA4.

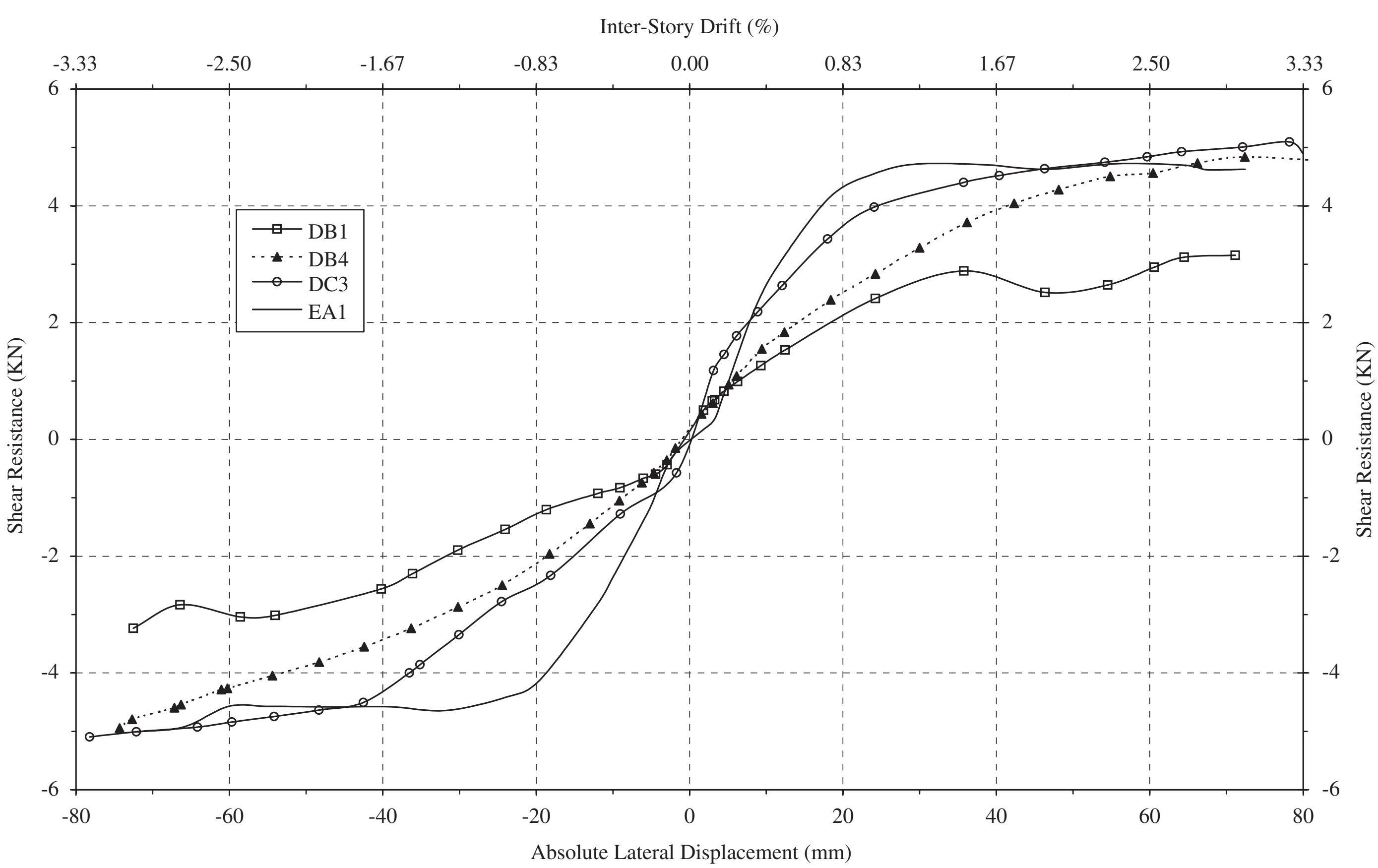

Fig. 8. First cycle load–displacement envelope plots for specimens DB1, DB4, DC3 and EA1.

study all specimens were clad on both sides with gypsum boards. The crack propagation in the boards in the first cycle of push direction may have caused a reduction in the load bearing of the gypsum boards in the first cycle of pull direction.

Moreover, Adham et al. [11] reported low ductility with considerable difference between the shear load capacity of the system in the first cycle and the stabilized cycles. In contrast, the current study showed fairly high ductility with a constant shear resistance in all displacement amplitudes, as is seen in Figs. 6(b), (d), (f) and (h). Again, the difference is in the presence of gypsum board in Adham's study.

In contrast with Gad et al. [13,14] who concluded that the failure mechanism of unlined frames is governed by tearing of the strap at strap-to-frame connection or at the tension unit location, the result of the current study showed that by using a perforated strap (or a solid strap with sufficient ratio of ultimate to yielding stress) and using brackets or gusset plates, this brittle failure mode can be completely eliminated. A similar conclusion was made by Tian et al. [15] who reported that the cross-sectional area of a strap significantly affects the deflection and stiffness of a frame, while having little influence on its racking load capacity (racking failure load). This misleading conclusion is due to incompetent detailing of the walls (weak strap-to-CFS frame connection and weak single section chord member), which do not allow straps to reach yield. The system fails instead at the strap connections or by buckling of the chord members.

In contrast with Fulop and Dubina [16] and Al-Kharat and Rogers [19], who reported a continuous descent in the total shear resistance of the system after a peak at about 1–2% inter-story drift, all walls in this study showed an ascending load-bearing capacity after yield in 0.5–0.6% drift ratio. This can be attributed to the detailing of the corners in these studies, which resulted in local failure at the corners rather than the desired yielding of the strap. Al-Kharat and Rogers [19], in particular, used flat plates in the four top and bottom corners of the tracks similar to base plates. These connections were inconsistent with the expected function of hold-downs which are supposed to transfer the chord stud forces to the support.

Overall, these studies [12–17,19] conducted monotonic and/or cyclic tests on gravity designed CFS stud walls and observed failure mechanisms triggered by incompetent detailing and connections which did not allow the full yield capacity of the straps to be developed. Also in most of the previous studies [11,12,15–19], no pre-tensioning was applied to the tension-only straps, in spite of explicit recommendations of most of the design codes [1–9].

7. Conclusion and recommendations

Based on current research results, following conclusions can be drawn:

(a) New systems for strap-braced system:

1. Adding brackets at four corners of a wall panel improves the lateral performance of the panel considerably, even when only a single stud is used as a chord member. Besides supporting the chords and the tracks against buckling (by reducing the buckling length of the members), one great advantage of this system is the removal of the tension unit from the main load transfer path thereby eliminating the possibility of strap tension failure at this location.
2. By choosing appropriate perforated straps, the tearing of the strap at the tension unit location or at the strap-to-frame connection would not occur. In addition, yielding of the strap would occur alongside the distributed holes. For a strap with close tensile and yield strength, perforating may be the only option to eliminate the brittle tension failure.

(b) Appropriate details to improve seismic performance of strap-braced system:

1. When straps are connected to the exterior chord–track joints, as neither of the adjoining members is continuous, the overall buckling load capacities of these members are low, especially when hold-downs are not provided at the top track. In order not to allow the undesirable buckling failure modes to govern, double back-to-back studs can be used as chord members. This restores the strap yielding failure mode as long as the chords are properly connected to tracks.
2. The performance of the X-strap system can be improved by placing four C-section cut-offs in the track at the four corners of the frame where the straps are connected. These cut-offs, even in small lengths equal to about eight times the half wave-length of local buckling of the track member, when connected to both track's flanges with single screws spaced around the local buckling half wave-length, can reduce the amount of bearing stress in the connection considerably, as well as provide a stiffer semi-rigid stud-to-track connection which improves the lateral load capacity of the frame.
3. When straps were connected to interior joints, a very good performance was observed, even with single chords and a top track without hold-downs. Two-side strap-braced wall showed a total lateral resistance close to twice the lateral resistance of one-side strap-braced wall especially for inter-story drift ratios greater than 2%. However, due to the higher slope of the strap, the stiffness and maximum shear resistance were lower than that of other types of strap bracing, and due to the high flexibility of the wall, full capacity of the strap was only reached in large lateral displacements.
4. The initial slackness in the strap should be as small as possible; otherwise, premature failure in the strap at the tension device location would occur.
5. When hold-downs are located inside the frame, such as that shown in Fig. 1, better performance and strength are achieved, in contrast to hold-downs that are connected to the outer face of the chord members, due to higher punching shear capacity.

(c) Lateral performance of strap-braced system:

1. The frame itself provides limited shear strength and stiffness, even in the frame with brackets. This finding is similarly reported in [18], where even by using box members for the studs, fixed to the top and bottom floors, the frame provided limited strength, stiffness and energy dissipation due to the local buckling of studs.
2. Local and distortional buckling in frame members are very stable modes of buckling and the braced wall can support a considerable amount of shear after the first signs of these bucklings occur. Even more, used frames, with initial distortional buckling but with no global (lateral–torsional) buckling due to previous loading history, behave similar to intact frames after the straps are replaced with new ones. However, the local and distortional bucklings in the studs and chords can cause reduction in the lateral load capacity of the frame itself, as a moment-resisting frame. Similar results were reported in [18] for the built-up box chord members.
3. The load–displacement and load versus number of cycles curves, depicted in Fig. 6, show that although this system represents highly pinched hysteretic behavior, it is still very

much ductile and can reach a shear resistance equal to that of the first cycle in every subsequent stabilized cycle.

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