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Performance of Knee-Braced Cold-Formed Steel Shear Walls Subjected to Lateral Cyclic Loading

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Abstract

Light weight Steel Framed structures currently in use in Australia, are normally braced using face mounted thin straps, cross braces that are of the same shape as studs, or compressed cement boards screwed to the face of the walls. While these are found adequate in low seismic regions of Australia, an investigation into the earthquake resistance properties of LSF have led authors to investigate alternative bracing types that may present a more favourable ductility. Knee braces that are specially designed for this purpose are introduced in the paper and studied in a specially designed testing rig. The tests are on four full scale walls of 2.4 m × 2.4 m and are of a cyclic nature. Of particular interest are the specimens maximum lateral load capacity and the load-deformation behaviour. The study also looks at the failure modes of the system and investigates the main factors contributing to the ductile response of the LSF walls in order to suggest improvements so that the shear steel walls respond plastically with a significant drift and without any risk of brittle failure such as connection failure or stud buckling. The walls tested have different length of Knee-elements with or without brackets which have same length of Knee-elements. The study shows that although the performance of this kind of LSF lateral resistant system under cyclic loads is satisfactory, its shear strength is significantly lower than those LSF lateral resistant systems which are currently in use in Australia. In regions with medium to high seismic activity, the use of these braces would not be sufficient purely as to the lateral resistance.

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Introduction

Light Weight Steel Frames are widely used in housing industry especially in low rise residential buildings. They are cost-effective, light and easy to work with. Compared to common hot rolled steel structures, the structural behaviour of LSF structures is more complicated as they are very thin-walled members and suffer from intersection plate instability. Steel Framed structures currently in use in Australia, are normally braced using face mounted thin straps, cross braces that are of the same shape as studs, or compressed cement boards screwed to the face of the walls. While these are found adequate in low seismic regions of Australia, an investigation into the earthquake resistance properties of LSF have led authors to investigate alternative bracing types that may present a more favourable ductile response. Knee braces that are specially designed for this purpose are introduced in the paper and studied in a specially designed testing rig.

Of particular interest in this study are the effects of Knee-element length and the use of brackets on the lateral performance. Knee elements maintain a considerable reserve of post-local buckling strength prior to yielding. So, it is expected that their presence would facilitate a more ductile response. The brackets also add to the redundancy of the system and as such increase the ductility of the system in a similar manner.

The walls which are studied here are unlined and the positive effect of gypsum board on the lateral performance of the frame under cyclic loading is ignored; that is because post-earthquake observations of the timber frame structures in the Northridge earthquake have also shown that many gypsum board shear walls failed under imposed dynamic load (Serrette and Ogunfunmi, 1996). Also, some design codes (US Army Corps of Engineers, 1998) have recommended neglecting the gypsum board contribution and relying only on the bare steel frames. Scrutinizing the obtained results and comparing the results to other experiments which performed by the authors and other researchers, show that although this failure is ductile, the strength is not high enough, and as such the use of this kind of LSF structure is not preferable particularly in medium to high seismic regions.

Test Setup

The general configuration of the testing rig is shown in Figure 1. Each specimen was installed on the rig in between the fixed support beam at the bottom and a rigid loading beam at the top using four M16 high strength bolts in the vicinity of chords and middle of the tracks either side. The bolts were tightened by a torque wrench to a torque of about 190 Nm that was corresponding to about 53 KN tension in the bolt. A strong combination of washers and nuts were used to ensure that there was no slip possibility between the tracks and the beams. Also, as shown in the figure, four hold-down angles were used at the four corners of the wall in order to lower the possibility of overturning and providing a proper load path from the braces to the wall chords and studs. An accurate Horizontal Drift (DH) transducer was used to evaluate the horizontal displacement of the top track. In order to evaluate the amount of uplift, four transducers were placed at the four corners of the walls in between the frame and the tracks. Also, one load-cell was used to measure the racking resistance. All data from the transducers and load-cell were analysed and transferred to the computer using Lab View Signal Express software (LabVIEW, 2007); and then the lateral performance of each frame was plotted.

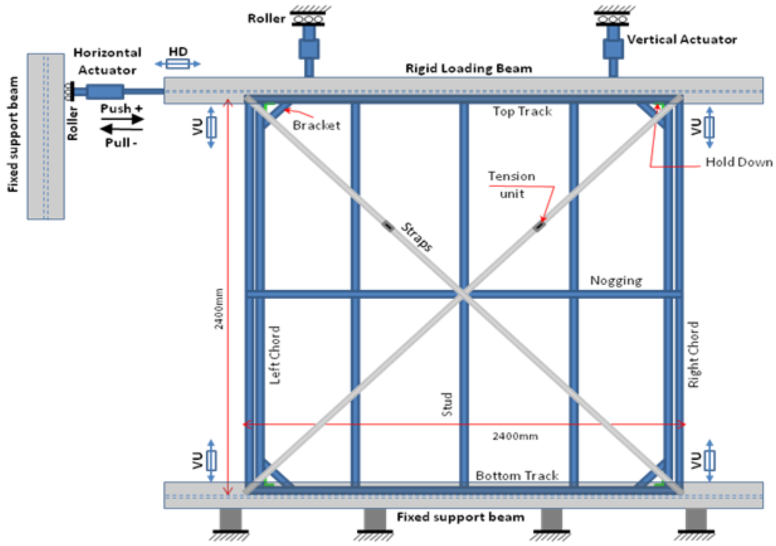


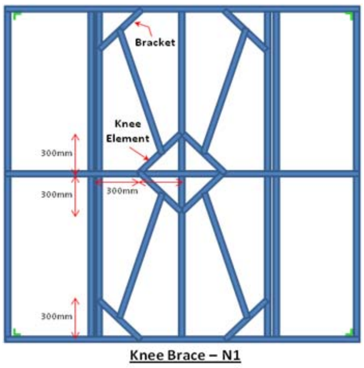
Figure 1- Testing Rig Diagram and notation convention

The cyclic loading regime that has been used in this research study is based on Method B of ASTM Standard (E2126-07, 2007), which was originally developed for ISO (International Organization for Standardization) standard 16670. This loading methodology consists of one full cycle at 0.5, 1, 2, 3, 4 mm and three full cycles at 8, 16, 24, 32, 40, 48, 56, 64, and 72 mm, unless failure or a significant decrease in the load resistance occurs earlier. The mentioned lateral amplitudes are corresponding to 1.25%, 2.5%, 5%, 7.5%, 10%, 20%, 40%, 60%, 80%, 100%, 120%, 140%, 160%, and 180% of the ultimate lateral displacement of the walls. It is worth noting that Method B of ASTM E2126-07 stipulates that the amplitude of cyclic displacements has to be selected based on fractions of monotonic ultimate displacement. If it was to be used here, since each specimen has its own ultimate displacement, the loading regime would vary for different specimen types. However, as set out earlier, one of the current research objectives is the comparison of different types of Knee-braced configurations of the shear walls. This would necessitate using identical cyclic amplitudes for different walls, as represented earlier. Hence, Method B is therefore used in this study with lateral amplitude independent of monotonic testing. Moreover, although 75 mm, or 3.125%, inter-story drift ratio was the maximum amplitude of our actuator, it was considered adequate as the maximum allowable story drift ratio specified by Standard FEMA450 is 2.5% (BSSC, 2003). The average loading velocity was about 2mm/s which is compatible with the ASTM E2126-07 which recommends the loading velocity must be in the range of 1–63mm/s.

Experimental Program

The program consisted of four 2.4 to 2.4 m full scale frames to investigate the hysteretic lateral performance of different configuration of Knee-braced walls as shown in Figures 2 to 5. Specimens N1 and N3 included concurrent Knee-braced system and brackets in the four interior corners of the wall. This was to investigate the effects of brackets on the frames performances. In order to reduce the number of geometric variants, the length of knee elements and brackets were considered equal. The Knee-elements length was $300\sqrt{2}$mm which is equal to thirteen times the half wave-length (HWL) of local buckling of the stud section in specimen N1, and $200\sqrt{2}$mm (eight times the local buckling HWL) in specimen N3. The diagonal elements were connected to the middle of elements exactly as shown in Figures 2 and 4.
These walls were tested in the Structural Laboratory of the School of Civil Engineering, the University of Queensland using a specially made testing rig illustrated previously. All of the frame elements, such as: top and bottom tracks, noggins, studs and Knee-elements were made by an identical C section of dimensions 90x36x0.55. The section structural properties are shown in Table 1; and its detailed section geometry is shown in Figure 6.

All components were connected together at each flange using just one rivet with the shear strength capacity and tensile strength capacity of 3.3 KN and 3.8KN respectively.
The effects of different components such as: the use of bracket, length of bracket, length of Knee element, are monitored and investigated in this research by changing them from one specimen to another specimen.



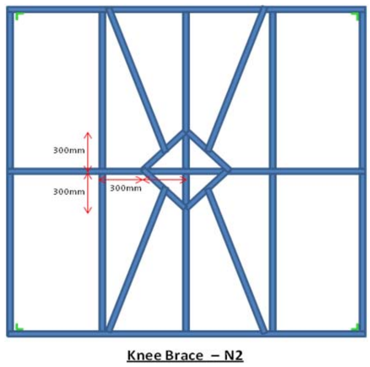
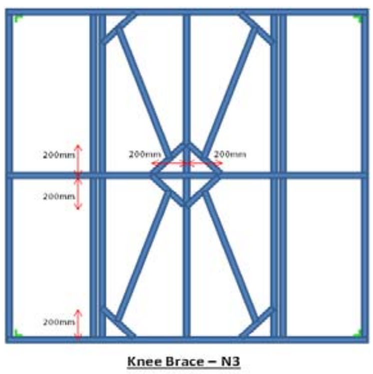


Figure 2 - Specimen N1 Figure 3 - Specimen N2



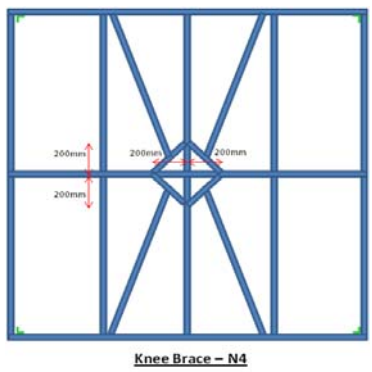


Figure 4 - Specimen N3 Figure 5 - Specimen N4

Nominal Grade	550 MPa	Yield Strain	0.45 %
Nominal Thickness	0.55 mm	Ultimate Stress, Fu	617.25 MPa
Elastic Modulus	168.93 GPa	Ultimate Strain	2.86 %
Yield Stress, Fy	592.26 MPa	Fu/Fy	1.04

Table 1 - Mechanical properties of the C Section Stud

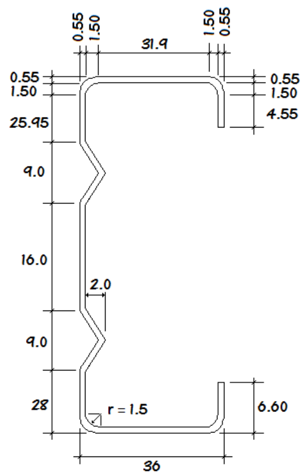


Figure 6 - C90x36x0.55

Experimental Results

The first specimen, N1, as depicted in Figure 2 was consisted of a wall panel with four brackets in the interior corners. To prevent buckling in the side chords, double studs sections were used. Interestingly, the panel performance was perfect and no failure mode was observed up to the end of the test that was corresponding to maximum drift cycle of 74 mm, though some plastic local buckling were occurred in the Knee-elements connections at the central part of the frame which was followed by plastic bending in the middle of the brackets. The hysteretic envelope curves and Load-Deflection Hysteretic Cycles for all

Specimens are shown in Figure 7 to 11. The envelope curves are derived from the load-deflection hysteretic cycles which are obtained from racking tests using accurate transducers and Lab View software (LabVIEW, 2007). The outputs of the software are in the EXCEL format, and can be used for the required post-experimental analyses such as the described envelope curves.

For specimen N2 (presented in Figure 3), after the application of the lateral loads, early plastic local buckling occurred in the Knee-elements connections; however the frame lost its capacity only after the rivet pull-out at the end of diagonal braces. This was considered as the main failure mode of the frame and was corresponding to the third cycle of 56 mm drift in the upward cyclic loading. Next specimen was N3 (shown in Figure 4). It was similar to specimen N1 with a smaller length for Knee-elements and brackets.

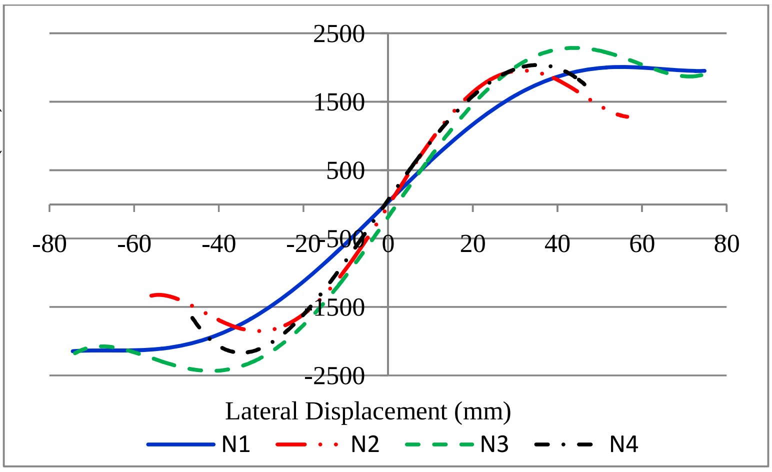


Figure 7 - Hysteretic Envelope Curve for all Specimens

Again for specimen N3 no specific failure mode was observed up to the end of the test. The only phenomenon was plastic local buckling in the Knee-elements connections followed by plastic bending in the brackets. Figure 5 shows the final shear wall, N4, which was tested. The major failure mode for this wall was a plastic global buckling in the longer Knee elements followed by the rivet pull-out corresponding to the second cycle of 48 mm drift.

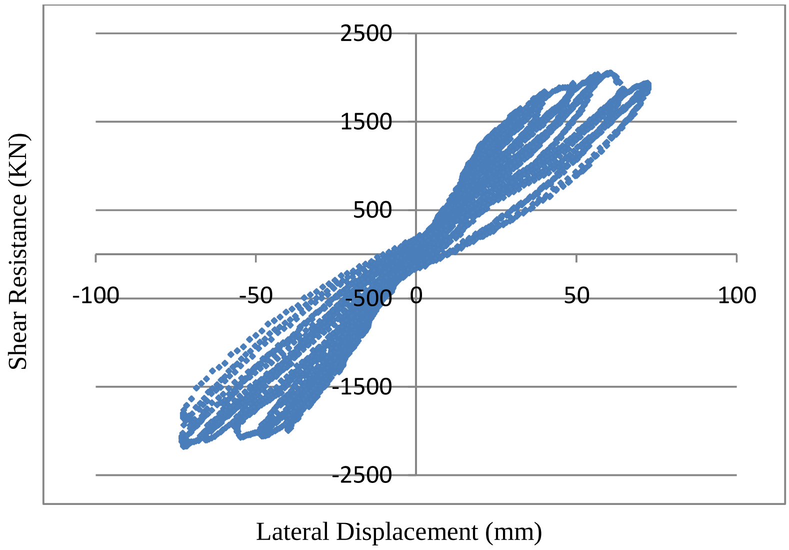


Figure 8 - Load-deflection hysteretic cycles for Specimen N1

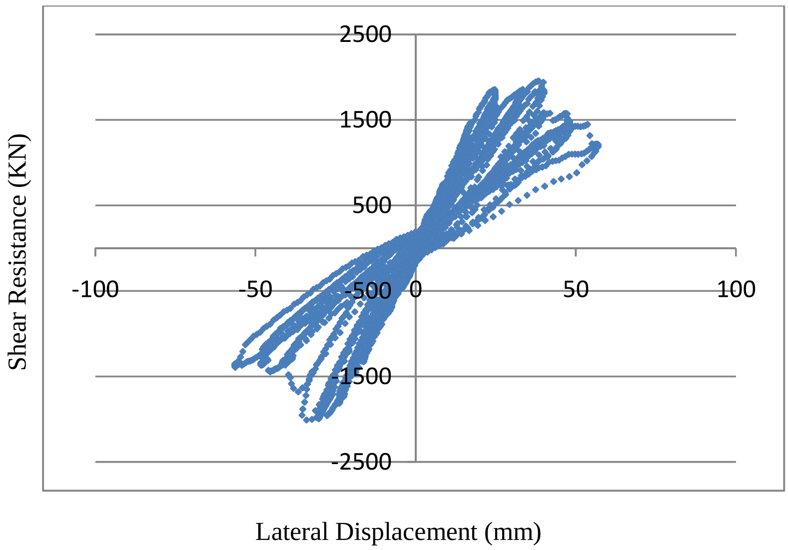
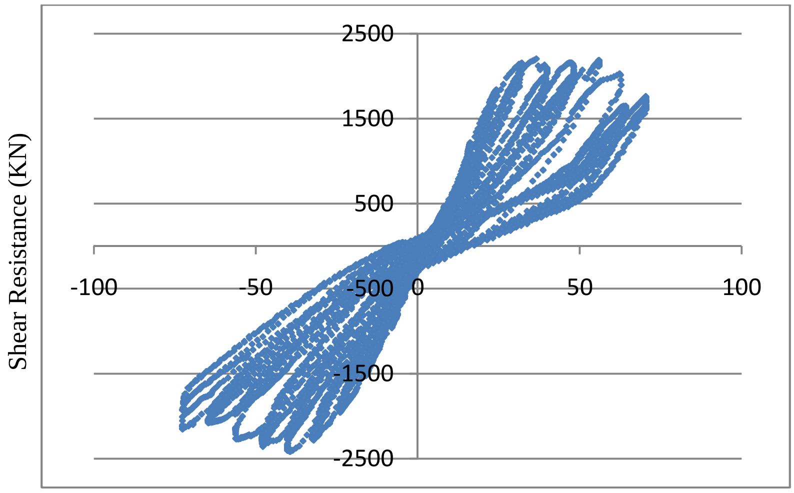
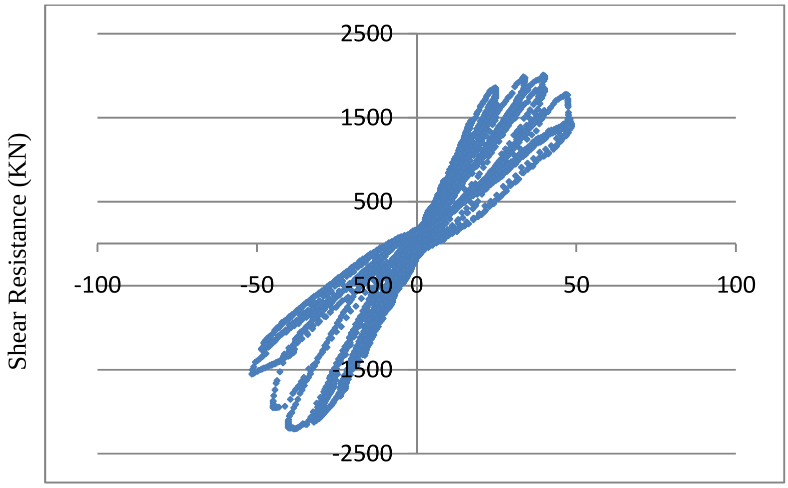


Figure 9 - Load-deflection hysteretic cycles for Specimen N2



Lateral Displacement (mm)
Figure 10 - Load-deflection hysteretic cycles for Specimen N3



Lateral Displacement (mm)
Figure 11 - Load-deflection hysteretic cycles for Specimen N4

Conclusions and Recommendations

According to the current research results, comparing the associated envelope curves and load-deflection hysteretic cycles in Figures 7 to 11, following conclusions can be made:

1- Using brackets at four interior corners of a Knee braced wall panel improves the lateral performance of the panel considerably, including both shear wall strength and the panel ductility. Besides supporting the chords and the tracks against buckling by reducing the buckling length of the members, one great advantage of using brackets is to use the plastic bending capacity of the brackets as an additional plastic energy dissipating mechanism in the frame. It is necessary to mention that using double stud sections for the chord members is essential to improve the lateral performance of the walls when brackets are incorporated as it increases the chord buckling capacity.

2- The performance of the Knee Brace lateral resistant system would be improved by decreasing the length of Knee-elements from $300\sqrt{2}$mm (thirteen times the half wave-length of local buckling of the stud section) to $200\sqrt{2}$mm (eight times the local buckling half wave-length). In another word, although the lateral performances of both specimens N1 and N3 which include the brackets were acceptable and no specific failure modes were observed during the tests and the ultimate drifts were approximately similar, the maximum absolute shear load for specimen N3 which had shorter Knee-elements was higher than that of N1. As is evident in Figure 7, the area which is enclosed by the Equivalent Energy Elastic Plastic (EEEP) curve and the capacity of energy dissipation for specimen N3 is higher than other specimens.

3- Comparing the envelop curves of specimens N2 and N4, it is seen that a shorter Knee-element leads to a greater shear strength for the wall but at the expense of a lower ductility. That is because larger Knee-elements provide more post local buckling reserve which allows the walls to deform further under the lateral loads.

4- Investigating the test results and the final failure modes for different specimens, a suggestion would arise with regard to preventing the brittle failure of the walls (with no bracket) associated with rivet pull-out; and this is to use appropriate washers under the rivets or use a rivet with wider head. This suggestion has been implemented in the current study and as seen confirmed by the results.

5- It is noted that the frame performance depend on the accuracy of the manufacturing of LSF elements. Existing gap (The lack of continuity of the web element) in the Knee-elements to stud elements connections causes the early plastic local buckling in the connections that finally leads to undesirable failure modes such as tearing in the connections; and as such the real capacity of frame cannot be utilized. Also, as the bending capacity of studs is low, it is essential to connect different Knee-elements at the same point or as close to each other as it possibly can be to prevent any lever arm and bending moment development in the studs.

6- Considering the aforementioned results and comparing those to the results of strap bracing performed by (Moghimi and Ronagh, 2009), it is concluded that although the performance of Knee-braced cold-formed steel lateral resistant system under cyclic loads with respect to ductility is satisfactory, the shear strength of this kind of lateral resistant system is much lower than what a typical LSF house needs especially in medium to high seismic regions. Hence; it seems that Knee-brace system is not a preferable choice as an efficient lateral resistant system.

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