Mahendran, Mahen and Subaaharan, Subhash (2002) Shear Strength of Sandwich Panel Systems.

Australian Journal of Structural Engineering 3(3):pp. 115-126.

Shear Strength of Sandwich Panel Systems

By M. Mahendran and S. Subaaharan
Physical Infrastructure Centre,
School of Civil Engineering,
Queensland University of Technology
Brisbane Qld 4000.

Any correspondence in relation to this paper should be addressed to

Associate Professor Mahen Mahendran
Director, Physical Infrastructure Centre
School of Civil Engineering,
Queensland University of Technology
Brisbane Qld 4000, Australia
Ph: 61 7 3864 2543 Fax: 61 7 3864 1515 Email: m.mahendran@qut.edu.au

ABSTRACT

The use of sandwich panels as roof and wall claddings has increased considerably in recent times. For the designers to take advantage of the diaphragm action of stronger sandwich panel systems under in-plane shear forces due to wind loading, appropriate data on shear strength and stiffness of these systems is required. An experimental investigation was therefore conducted to investigate the shear behaviour of commonly used crest-fixed sandwich panel systems. An improved fastening system was developed which resulted in approximately 2.5 times greater shear strength, and improved ductility. Shear strength and stiffness data were developed for sandwich panel systems with the improved fastening system for varying aspect ratio. Analytical formulae were also developed to predict the shear strength of sandwich panel systems using the basic tearing loads obtained from simple connection tests. This paper presents the details of both experimental and analytical studies on the shear behaviour of sandwich panel systems and the results.

1. INTRODUCTION

During high wind events such as storms and cyclones, low-rise steel buildings are subjected to uplift pressures on roof claddings and lateral pressures on wall claddings. The lateral pressures can be resisted by the diaphragm action (stressed skin) of steel cladding systems. However, in Australia, where roof claddings are crest-fixed, designers ignore the diaphragm action, and instead provide separate bracing systems to carry lateral/racking forces. However, the true behaviour of buildings under wind loads is influenced by the diaphragm action of roof and wall panels whether designers acknowledge it or not. Full scale tests on houses (Reardon and Mahendran, 1988) and steel portal frame buildings with profiled steel claddings (Mahendran and Moor, 1999) have shown that both roof and wall claddings acted as shear diaphragms in transferring the lateral forces to the stiffer end walls. Therefore it is important that building design is based on the true behaviour of the building instead of its assumed behaviour. This means that the shear strength and stiffness of steel claddings should be included in the building design (Davies and Bryan, 1982, Nash, 1985), and claddings should be designed to withstand both wind uplift and in-plane racking (shear) forces caused by high wind events.

Profiled steel sheeting made of thin, high strength steels (0.42 mm base metal thickness and G550 steel with a minimum yield stress of 550 MPa) are commonly used as roof and wall claddings in the Australian Building industry. The behaviour of these crest-fixed claddings under in-plane shear forces was first investigated at Queensland University of Technology (QUT) and appropriate shear strength and stiffness data have been developed (Mahendran and Subaaharan, 1995). In recent years, structural sandwich panels are increasingly used as roof and wall claddings in the building industry. These panels are made of two steel faces with a polystyrene foam core, and are considerably stronger than the conventional profiled steel claddings (Figure 1). The inclusion of shear strength and stiffness data of sandwich panels in the building design is expected to provide greater improvements. Therefore the QUT research was continued to include shear behaviour of crest-fixed sandwich panels using large-scale shear/racking experiments of panels and small-scale connection tests. This paper presents the details of the shear / racking test rigs, test methods and the results, and compares them with some simple analytical predictions. It includes recommendations on the use of suitable design formulae for the strength and stiffness of sandwich panels in shear.

2. EXPERIMENTAL METHOD

2.1 General

Based on the experimental methods used by Nash and Boughton (1981), Davies and Bryan (1982) and Bryan and Davies (1981), Mahendran and Subaaharan (1995) have already investigated the behaviour of crest-fixed profiled steel roof claddings under shear loads. A similar approach was used in this investigation on large-scale sandwich panel systems.

The main objective of this investigation was not only to assess the shear capacity of crest-fixed sandwich panel systems with commonly used fastening systems, but also to increase their shear capacity using improved fastening systems. This would also eliminate premature brittle failure of fasteners. At present, the fastening system used for crest-fixed sandwich panel systems is very similar to that of conventional profiled crest-fixed steel cladding systems. It is obvious that the current fastening system will not make use of the higher strength of composite sandwich panels. For a greater racking/shear capacity, more fasteners have to be added along the purlin and the lap joint. Therefore this investigation attempted to improve the fastening system in order to increase the shear capacity of sandwich panel cladding system.

2.2 Sandwich Panel Cladding Systems

A series of laboratory experiments simulating shear/racking forces due to wind loading was carried out on crest-fixed sandwich panel roofing systems commonly used in Australia (see Figure 1). A profiled sandwich panel manufactured by National Panel Pty Ltd was used in this investigation. This panel had a high tensile steel face at the top (0.42 mm base metal thickness and minimum yield strength of 550 MPa) and a forming grade/mild steel face at the bottom (0.60 mm base metal thickness and minimum yield strength of 300 MPa). Polystyrene was used as the foam core, which was glued to the top and bottom steel faces using a special adhesive. The panels used in this investigation had a width of 1020 mm and foam thickness of 50 and 75 mm.

The steel RHS members of 100 mm x 100 mm x 6 mm were used as purlins that are commonly used in the sandwich panel cladding systems. The main screw fasteners were No.14-10 x 135 mm and the seam/lap fasteners were No.12-11 x 25 mm. They were all self-drilling HiTeks screws. Maximum size of panel system tested was 6.0 m x 3.0 m.

2.3 Shear Test Set-up

The shear/racking test rig was designed and constructed to be able to test crest-fixed sandwich panel systems up to a maximum size of 6.0 m x 6.0 m as shown in Figure 2. The same principles used by Davies and Bryan (1982) for valley-fixed steel claddings were used. A sandwich panel system was used between two rafters as they are commonly used in the industrial and commercial buildings. As seen in Figures 2 (a) and (b), the test rig had two rafters, and one of them was free to move under the racking load. The steel purlins (100 mm x 100 mm x 6 mm RHS) were then fixed to these rafters at 3 m spacing. The purlins were fixed to the rafters via specially made joints that allowed free rotation of purlins under the racking load on one of the rafters (Figure 2 (c)). In the absence of sandwich panels, the entire test rig was free to move even under a small racking load. Once the panels were screw-fastened to the purlins, the shear resistance was only provided by the sandwich panel system. This arrangement was similar to that used with tests on profiled steel claddings by Mahendran and Subaaharan (1995). Figure 2 (b) shows the application of racking load to one of the rafters using a hydraulic jack.

2.4 Shear Test Program and Results

The first test panel system was 3.0 m x 3.0 m with the fastening system as used in current practice (Figure 2(b)). A series of tests was then carried out on similar size panels as the first test, but with continuous improvements to the fastening system until reaching the optimum fastening arrangement. Tests were also carried out on panels with varying dimensions in order to investigate the effect of aspect ratio (width/length) on the stiffness of the panels. For this purpose, both length and width were changed in order to give an aspect ratio range of 0.5 to 2. Table 1 shows the details of each test. During each test, the shear/racking load was increased in steps and the corresponding load cell and dial gauge readings were recorded until panel failure. In addition to the maximum load, the load at which the panel commenced tearing or fasteners fractured were also noted. Deflections were also measured at the loading point. Details of each test and purpose are given next.

Test 1: A 3 m x 3 m panel system was built with sandwich panels of 50 mm thick and 1020 mm width and was fastened as used in practice (National Panel, 1995). Main fasteners of No.14-10 x 135 mm were used at every crest (250 mm spacing) whereas seam fasteners of No.12-11 x 25 mm were used along the lap at 1000 mm spacing to connect longitudinal edges of adjacent top panel widths as shown in Figure 3 (a). In contrast to the conventional steel claddings, the sandwich panel systems had seam fasteners along the lap joints. The shear failure (tearing) occurred at the lap joint at a shear load of 10 kN and was followed by main fastener fracture as shown in Figure 3 (b). This is because of the insufficient strength of lap joint to transfer the load from one panel to the other. Also the shear capacity of the main fasteners was incompatible with the higher shear capacity of the panel. This test showed that the strength of the lap joint had to be improved to obtain a greater capacity from the sandwich panel system and to avoid brittle failure of main fasteners. Despite the use of stronger sandwich panels the racking/shear capacity of 10 kN was the same as that obtained for conventional profiled steel claddings (Mahendran and Subaaharan, 1995). Figure 3 (b) also shows the overall shear failure of the sandwich panel system.

Tests 2, 3, 4 & 5: The test panel system in Tests 2 to 5 was similar to Test 1 panel, but had more seam/lap fasteners as shown in Figure 3 (c). For Test 2, seam fasteners of No.12-11 x 25 were used at 500 mm spacing, ie. twice the number of seam fasteners than Test 1, but the same number of main fasteners as in Test 1. The shear failure commenced at the seam fasteners along the lap joint and the panel failure and main fastener fracture occurred at a racking load of 10.5 kN. This is because the lap joint strength was still insufficient. Therefore, in Test 3, seam fasteners of No.12-11 x 25 were used at 250 mm spacing. The failure was identical to Tests 1 and 2 and occurred at a shear load of only 11.5 kN. Therefore, in Test 4, more seam fasteners No.12-11 x 25 were used at 100 mm spacing. Number of main fasteners remained the same as in Tests 1 to 3.

In Test 4, the failure commenced at the edge of panel system by the fracture of main fasteners at a shear load of approximately 16.5 kN and further loading caused the other main fasteners to fail (brittle failure) as observed in previous tests. But the lap joint shear failure was delayed as the strength of lap joint was sufficient to transfer the load from one panel to the other. The number of seam fasteners was therefore sufficient for this system, but the shear capacity of the main fasteners was insufficient. This test showed that the shear capacity could be further increased and brittle failure avoided if the capacity of the main screw fasteners was increased. It was therefore decided to improve the strength of main screw fasteners and to keep the seam fasteners as in Test 4. As larger size screws were not available, it was decided to increase the number of main fasteners at each crest of the panels. In Test 5, the same 3 m x 3 m panel system as in Tests 1 to 4 was fastened with more main fasteners at each crest as shown in Figure 3 (c). Two main fasteners of No.14-10 x 135 were used at every crest of the panels and seam fasteners of No.12-11 x 25 were fastened at 100 mm spacing along the lap. A tearing failure commenced at the main fasteners along the lap joint and further loading caused seam fasteners and also other main fasteners to fail by tearing. The main fasteners did not fracture in a brittle manner. Ultimately, both main and seam fasteners failed in shear at a load of 27 kN, ie, both fasteners' failures were ductile. This showed that panels reached the maximum shear capacity and any further improvements of fastening system would not improve the shear capacity as both main and seam fasteners failed in shear.

Test 6: Based on the above five tests, the behaviour of the panel system was analysed and it was found that the same shear capacity of the system obtained in Test 5 could be reached by just adding one more main fastener (three fasteners) at the crest along the lap joint and reducing the number of seam fasteners to a spacing of 500 mm instead of 100 mm as shown in Figure 3 (c). This was verified by this test on the same 3 m x 3m panel system made up of sandwich panel of 50 mm thick and 1020 mm width. It was fastened with the three main screws of No.14-10 x 135 at the lap crest of panel system, two main fasteners at other crest locations of the panel system, and seam fasteners of No.12-11 x 25 at 500 mm spacing along the lap joint. A tearing failure commenced at the main fasteners along the lap joints and further loading caused all seam and main fasteners to fail by tearing (see Figure 3 (d)). Ultimately, both main and seam fasteners failed in shear at a load of 24 kN. This failure is similar to that observed in Test 5. This rearrangement of main and seam fasteners was considered to be a better and efficient arrangement of the fastening system than that used in Test 5 even though both arrangements produced approximately the same results.

Tests 7, 8 and 9: In these tests the improved fastening system developed from Test 6 was used with various aspect ratios of the panel system to develop the relationship between the strength and stiffness/flexibility of the panel systems and aspect ratio. Panel systems of 3 m x

6 m, 4.7 m x 3 m and 6 m x 3 m were built of sandwich panels of 50 mm thick and 1020 mm width. In these tests, the failure was similar to that of Test 6. These tests validated the development of the most efficient fastening system in Test 6.

Test 10: This test was carried out with 75 mm thickness panel to study the shear behaviour of panels with different thicknesses. A panel system of 3m x 3m was built of sandwich panels of 75 mm thick and 1020 mm width and was fastened as in Test 6. The shear failure was similar to that observed in Test 6 at a shear load of 23.5 kN. This also showed that panels reached the maximum shear capacity and any further improvements of fastening system would not improve the shear capacity of the panel system. The shear strength of the 75 mm thick panel system was almost equal to that of the 50 mm panel system. Also both main and seam fasteners failed in shear at the ultimate load. Therefore, this test proved that the shear capacity of the panel system was independent of the thickness of the sandwich panel, but was dependent mainly on the fastening system. Therefore, it was decided not to proceed with further tests on 75 mm thick panel system. Results obtained for 50 mm thick crest-fixed sandwich panels systems could be used for 75 mm thick crest-fixed sandwich panel systems.

2.5 Shear Test Results

Table 1 presents the results of the ten large-scale shear tests on crest-fixed sandwich panel systems described in the last section. Results include the onset of tearing load and the ultimate load in kN, and shear flexibility in mm/kN.

In summary, Tests 1 to 4 showed the improvements when the lap fastener spacing was increased. The shear strength was improved steadily from 10 kN to 16.5 kN, but Test panels 1, 2 and 3 failed due to tearing along the lap joints and fracture of main fasteners (brittle failure) rather than tearing. This indicated that the main fasteners did not have adequate capacity to match the higher shear capacity of the panel system. In Test 4, the main fasteners fractured before the tearing failure along the lap joint. In Test 5, the shear strength of the panel system was improved from 16.5 kN to 27 kN by adding one more main fastener at all the crest locations. Similar shear strength was obtained by increasing one more fastener at the crest along the lap joint and reducing the number of seam fasteners (Test 6). This was considered a simpler arrangement of fasteners. If larger screw fasteners with a higher shear capacity were available, a single main fastener would be adequate, leading to a simpler fastening arrangement.

A typical load-deflection curve for the 3 m x 3 m sandwich panel system is presented in Figure 4. Other curves are presented in Subaaharan and Mahendran (1997). Elastic shear flexibility of each sandwich panel system was obtained from the **e**lastic loading part of the load-deflection curve and was included in Table 1. These elastic shear flexibility values of sandwich panel systems can be used in the three-dimensional analysis of buildings attempting to include the stiffening effect of claddings. The shear flexibility values are preferred in these analyses as used by Davies and Bryan (1982). From Tests 6 to 9 the effect of aspect ratio on the shear flexibility of sandwich panel system was investigated, and their results are shown in Figure 5.

It was found that the shear flexibility of sandwich panel system did not depend on the size of the panel system used as long as purlin spacing remained the same. It was mainly dependent on its aspect ratio. This is a useful result as it means that a smaller test panel can be used in the shear tests to determine shear flexibility/stiffness values.

3. ANALYTICAL METHOD

3.1 General

It was found that the failure of large scale cladding systems commenced at main fasteners in the case of conventional profiled steel cladding system and at both main fasteners and seam fasteners along the lap joints in the case of sandwich panel cladding systems. This load is referred to as the onset of tearing load. Further loading caused tearing at all fasteners, which is referred to as the ultimate load. These failure loads of steel cladding systems can be predicted using a simple analytical method provided the basic tearing load of fastener connections (main fasteners or both main and seam fasteners) along the lap joint (referred to as lap tearing load herein) and tearing load of main fastener connections at locations other than the lap (referred to as edge tearing load herein) are known.

Nash and Boughton (1981) developed a simple theory that predicts the onset of tearing and ultimate failure loads of crest-fixed steel cladding panels with and without lap joints. Despite the approximations of their theory, it produced reasonable predictions for their experiments on corrugated roofing (Nash and Boughton, 1981) and for the recent experiments on profiled steel cladding systems (Mahendran, 1994, and Mahendran and Subaaharan, 1995). In this investigation the same theory was further enhanced to predict the shear strength of crest-fixed sandwich panel systems. These analytical predictions were validated using full-scale experimental results presented in Section 2.

3.2 Shear Strength

This theory is based on a number of assumptions, which are described in this section. Figure 6 shows the onset of tearing and ultimate failure modes of cladding panels and the associated force distribution at each purlin on the cladding panel system used in the analysis. Assuming that the total applied racking load P is shared equally by the purlins (n), the load on each purlin is P/n. The shear forces at the fasteners on each purlin are F_1, F_2,---, F_m where m is the number of crests in the panel fastened at each purlin. Note that the maximum shear force is on the edge fasteners (F_1 or F_m) (see Figure 6 (b)). Despite the complexity of the structure, it is assumed that for elastic conditions, the force at a fastener is proportional to the distance of the fastener location from the neutral axis, that is, the centre of the panel system. Therefore, the force at the i^{th} crest fastener $F_i = F_m . X_i / 0.5h = 2 F_m . X_i / h$, where X_i is the distance of the crest from the neutral axis and h is the panel width as shown in Figures 6 (a) and (b).

The shear force carried by the panel can be found by summing the loads carried by the individual fasteners along one purlin. If there is a lap connection near the centre of the panel as shown in Figure 6 (a), the shear in the panel must be transferred from one panel to the other through the fasteners at the lap connection. The roof panel will yield when the shear in the panel at the lap connections exceeds the tearing load capacity of the lap connection corresponding to one purlin ($F_{tearing\text{-}lap}$). The panel will then start to tear along the lap connection and the corresponding load is defined as the onset of tearing load. The onset of tearing load P_{on} can be derived as follows.

Applied moment for each purlin = $P\, h/n$

Resisting moment $= \sum_{i=1}^{m} F_i X_i = \sum_{i=1}^{m} \left[\frac{2F_m X_i}{h} \right] X_i = \frac{2F_m}{h} \sum_{i=1}^{m} X_i^2$

By equating applied moment Ph/n to resisting moment $\frac{2F_m}{h} \sum_{i=1}^{m} X_i^2$,

F_m can be found as follows: $F_m = \frac{Ph^2}{2n} \Big/ \sum_{i=1}^{m} X_i^2$

At onset of tearing $P = P_{on}$. If tearing occurs at the p[th] lap connection, ie., the critical lap connection (see Figure 6 (a))

$$F_{tearing\text{-}lap} = \sum_{i=1}^{p} F_i = \sum_{i=1}^{p} 2F_m X_i \ /h = \sum_{i=1}^{p} 2\left[\frac{P_{on}h^2}{2n} \Big/ \sum_{i=1}^{m} X_i^2 \right] \frac{X_i}{h}$$

$$= P_{on} h \sum_{i=1}^{p} X_i \Big/ n \sum_{i=1}^{m} X_i^{\ 2}$$

$$P_{on} = F_{tearing-lap} n \sum_{i=1}^{m} X_i^{\ 2} \Big/ h \sum_{i=1}^{p} X_i \qquad (1)$$

where: m is the total number of crests fastened on each purlin

p is the number of crests fastened from the critical lap (the lap on which tearing begins) to the nearest edge of panel system

It must be noted that $F_{tearing\text{-}lap}$ in Equation (1) is the tearing capacity of the lap connection corresponding to one purlin and will depend on the number and size of fasteners along the lap. The lap tearing load of each fastener depends mainly on the steel thickness and the diameter of the screw fastener shaft. This can be determined using simple tension tests simulating fastener connections in a similar manner to that used by Nash and Boughton (1981). The experimental work conducted in this investigation to determine $F_{tearing\text{-}lap}$ is described in Section 3.3.

Increasing the load beyond the onset of tearing causes the failure of other lap connections in the panel system. By this time, all the lap joints have completely failed and panels behaved independently. The force distribution at this point is shown in Figure 6 (c). As the load is further increased, the failure load for the panel to purlin connection is reached at extreme edge connections on every panel. At this stage, these extreme main fastener edge connections reached the ultimate tearing load capacity, referred to as tearing load capacity at the edge. Further load was then carried by main fastener edge connections closer to the centre of each panel. Eventually all these edge connections will reach $F_{tearing\text{-}edge}$ and the entire panel system will fail at the failure load, P_{ult}. Figure 6 (d) shows the final force distribution across the panel. The edge tearing load capacity $F_{tearing\text{-}edge}$ is the total capacity of all the main fasteners at each edge connection and was determined using small scale tests (see Section 3.3) in a similar manner to that for $F_{tearing\text{-}lap}$. The ultimate load P_{ult} can be derived as follows.

Applied moment at ultimate load for each purlin $= P_{ult}\dfrac{h}{n}$

At ultimate stage, the lap and edge connections have completely failed and the resisting moment of the panel system is the sum of resisting moment of each individual roof panel.

Resisting moment per individual panel considering lap at only one side of the panel

$$= F_{tearing-lap} \times x_{lap} + F_{tearing-edge} \sum_{i=1}^{q} x_i$$

where: q is the total number of crests fastened on a panel except the lap crests.

x_{lap} is the distance of the lap connection from the centre of each panel.

x_i is the distance of fastener from the centre of each panel.

Resisting moment per individual panel considering lap at both sides of the panel

$$= F_{tearing-lap} \times x_{lap} \times 2 + F_{tearing-edge} \sum_{i=1}^{q} x_i$$

Now, consider the panel system with s number of panels, out of which two panels will have a lap connection only on one side whereas others will have one at both sides. Therefore, the resisting moment for the panel system is

$$(s\text{-}2)[\, F_{tearing-lap} \times x_{lap} \times 2 + F_{tearing-edge} \sum_{i=1}^{q} x_i \,] + 2\,[\, F_{tearing-lap} \times x_{lap} + F_{tearing-edge} \sum_{i=1}^{q} x_i \,]$$

Equating the resisting moment above to the applied moment ($= P_{ult}\dfrac{h}{n}$), the ultimate tearing load P_{ult} can be found as follows:

$$P_{ult} = \frac{n}{h} \left\{ (s\text{-}2)[\, F_{tearing-lap} \times x_{lap} \times 2 + F_{tearing-edge} \sum_{i=1}^{q} x_i \,] + 2\,[\, F_{tearing-lap} \times x_{lap} + F_{tearing-edge} \sum_{i=1}^{q} x_i \,] \right\} \tag{2}$$

It must be noted that $F_{tearing\text{-}edge}$ in Equation (2) is the tearing capacity of edge connection corresponding to one purlin which consists of one or more main fasteners whereas $F_{tearing\text{-}lap}$ is the tearing capacity of lap connection corresponding to one purlin which often consists of many seam fasteners and some main fasteners. In these equations (1) and (2), all parameters are known except $F_{tearing\text{-}lap}$ and $F_{tearing\text{-}edge}$. These values can be obtained using small-scale connection tests described in Section 3.3.

3.3 Small Scale Connection Testing

The failure loads of the large scale sandwich panel systems can be predicted using the simple analytical method described in Section 3.2 provided the basic connection tearing loads ($F_{tearing\text{-}lap}$ and $F_{tearing\text{-}edge}$) are determined in a similar manner to that of profiled steel cladding systems (Mahendran and Subaaharan, 1995).

3.3.1 Determination of Lap Connection Tearing Load

Figure 7 (a) shows the test set-up used to determine the lap tearing load of the sandwich panel systems. Lap connection of sandwich panel systems consists of main fasteners and seam fasteners. The main fasteners at the lap connect the longitudinal edges of adjacent panel widths through purlins (see Figure 3 (a)). Seam fasteners along the lap connect only the top faces of adjacent panels along the longitudinal edge (see Figures 1 and 3 (a)). To determine the total lap connection tearing load, tearing loads of a main fastener and that of a seam fastener are required.

Five tests were carried out with a main fastener to determine the tearing load of a main fastener. Other five tests were carried with both main and seam fasteners. From the above two types of tests, tearing load of a seam fastener was determined. This was attempted as the same test rig could be used for both types of tests. As seen in Figure 7 (a), a lap connection was made with two panels of approx. 150 mm x 300 mm and were fastened with a main fastener of No.14-10 x 135 mm to a piece of steel RHS for the first case and with a main fastener of No.14-10 x 135 mm and a seam fastener of No.12-11 x 25 mm in the second case. A tension load was applied until large tearing deformations occurred without any increase in load.

For the 50 mm thick sandwich panel, the mean and characteristic strengths of a main fastener were 4.22 and 4.07 kN, respectively, whereas for both main and seam fasteners, they were 4.57 and 4.45 kN. The mean strength of a seam fastener can be determined by subtracting one from the other, ie 4.57 - 4.22 = 0.35 kN. The corresponding characteristic strength can be determined to be the same, considering the small coefficient of variation. Therefore, the tearing capacity of each connection, $F_{tearing-lap}$, can then be determined using these results. Sample calculations are given in Section 3.4.

3.3.2 Determination of Edge Connection Tearing Load

Figure 7 (b) shows the test set-up used to determine the edge connection tearing load for a main fastener of the sandwich panel system. An edge connection was made of a panel of approx. 150 mm x 300 mm and was fastened to a piece of steel RHS with a main fastener of No.14-10 x 50 mm. A tension load was applied until large tearing deformations occurred without any increase in load. The mean ultimate tearing load obtained from five tests was 2.27 kN and the corresponding characteristic load was 2.13 kN.

As seen from the results, the lap tearing load of a main fastener is higher than the edge tearing load of the same main fastener. This is because the lap fastener connects the two top skins of the panels to purlin compared with a single top skin of a panel to purlin in the case of edge connection and hence the contact area between fastener and panel in lap connection is higher than that of edge fastener.

3.4 Calculation of Analytical Shear Strength

A sample calculation of analytical prediction of ultimate failure load and onset of tearing load of a sandwich panel cladding system of 3 m x 3 m using small scale connection tearing loads and Equations (1) and (2) are given next. For this purpose, the measured mean tearing loads were used instead of the characteristic strength values.

Consider Test 5, which had 2 main fasteners at each crest and seam fasteners at 100 mm spacing along the lap joints (Figure 8).

Number of purlins, n = 2

Panel dimension, h * b = 3000 mm x 3000 mm

Number of Panels = 3

Therefore, there were 2 lap connections

Number of main fasteners at crest = 2

Number of seam fasteners along lap = 29

Tearing capacity of the lap connection in the system

$F_{tearing\text{-}lap}$, corresponds to one purlin = $4.22 \times 2 + (4.57 - 4.22)\dfrac{29}{2} = 13.52kN$

Tearing capacity of the edge connection in the system

$F_{tearing\text{-}edge}$, corresponds to one purlin = $2 \times 2.27 = 4.54kN$

Spacing between fasteners = 250 mm

Distance from the centre of the panel system

to the first fastener = 125 mm

P_{on}= {13.52x2[125^2+375^2+625^2+ 875^2+1125^2+1375^2] x 2}/{3000[625+875+1125+1375]}

P_{on} = 20.1 kN

P_{ult} = 2{[13.52x500x2 + 4.54x250x2] + [13.52x500 + 4.54x500 + 4.54x250x2]

+ [13.52x375 + 4.54x375 + 4.54x125x2]}/3000 = 23.3 kN

Since the two edge panels (ie. with only one lap connection) in Test 5 were not identical, different x_i values were used in determining P_{ult} using Equation (2).

Analytical predictions for the onset of tearing and ultimate loads of the sandwich panel system using Equations (1) and (2) and the measured basic tearing loads are compared with the corresponding experimental results in Table 1. These results agree well in all the cases as seen in Table 1.

4. CONCLUSIONS

The following conclusions have been drawn from this investigation.

1. Shear/racking behaviour of sandwich panel systems used commonly in Australia was studied using large scale experiments of 3m x 3m panel systems to obtain the shear

strength and flexibility values. It was found that the current fastening system adopted for sandwich panel claddings was not utilising the full shear capacity of the composite sandwich panel. Also, it led to a brittle failure of fasteners under the shear/racking load. Therefore an improved fastening system was developed.

2. Despite the fact the sandwich panels were crest-fixed and had no shear connectors, they appeared to have considerable shear strength and stiffness. They may be adequate for designers to include the diaphragm action of such panels in building design. Shear strength of the commonly used sandwich panel system with improved fastener arrangement was approximately 2.5 times higher than the shear strength with the current fastening system. Shear flexibility of the commonly used sandwich panel system with improved fastening arrangement is approximately 1/4 of the flexibility value of conventional crest-fixed profiled cladding systems of similar dimensions.
3. Shear flexibility values reported in this paper could be used in the three dimensional analyses of buildings incorporating the effects of commonly used sandwich panels. Shear tests on a cladding system of 3m x 3m will be adequate to determine the shear strength and flexibility values of those cladding systems not included in this paper.
4. Formulae for analytical prediction of onset of tearing and ultimate shear failure loads of crest-fixed sandwich panel systems have been developed based on Nash and Boughton's (1981) theory and are given by Equations (1) and (2). Small-scale connection tests for sandwich panel systems were conducted to determine the lap connection and edge connection tearing loads. These values were used in the analytical formulae to predict the onset of tearing and the ultimate shear failure loads of the cladding system. The ultimate failure and onset of tearing loads from large scale shear tests agreed reasonably well with analytically predicted failure loads for sandwich panel systems.
5. Designers could use the analytical formulae developed in this paper and the characteristic tearing loads to determine the onset of tearing and ultimate loads for the sandwich panel systems. These loads could be used in the bracing design of roofs and walls using these cladding systems. For the cladding systems and fastening systems not considered in this investigation, small connection tests described in this paper can be used first to determine the tearing loads. The analytical formulae can then be used to predict the shear strength of the cladding system. This will eliminate the need for testing of large scale cladding systems.

5. ACKNOWLEDGEMENTS

The authors wish to thank Mr C. Heath for his help with laboratory experiments, and the National Panel Pty. Ltd. for donating the sandwich panels for the experiments.

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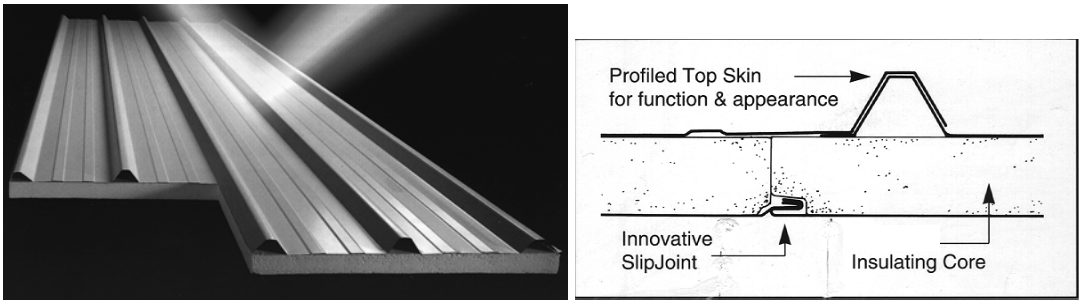
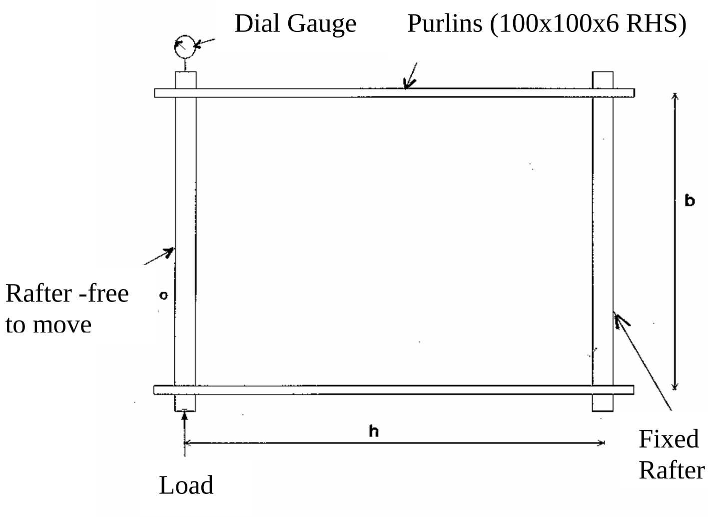
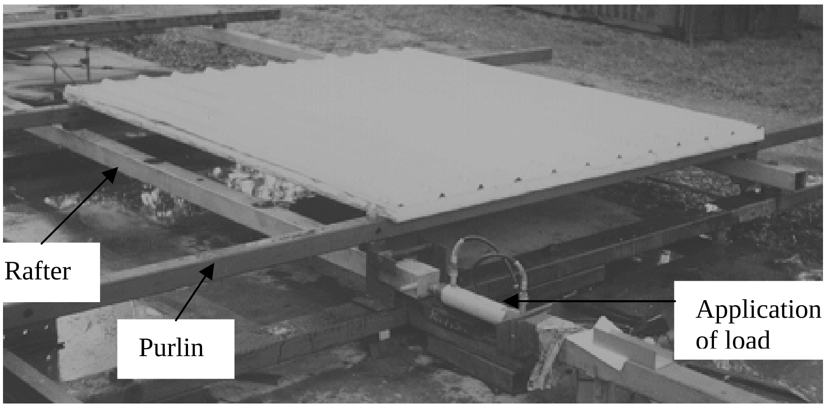


Figure 1. Sandwich Panels



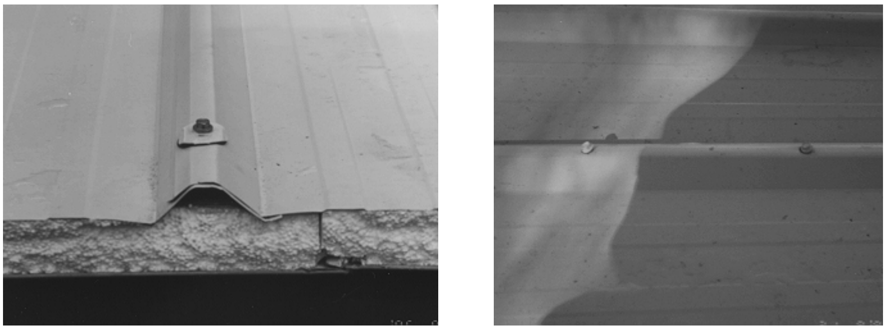
(a). Rafter/Purlin Arrangement – Schematic Diagram



(b) Shear Test Rig

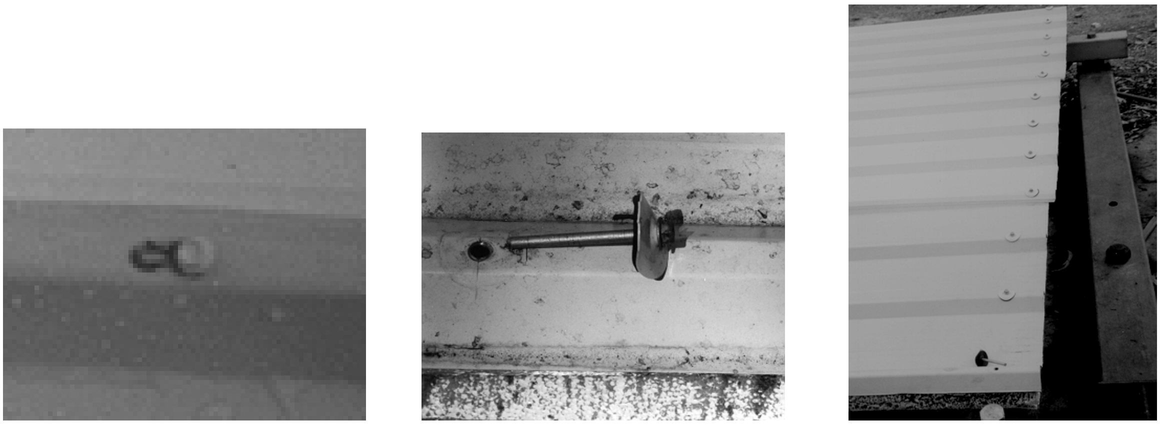
(c). Purlin to Rafter Joint

Figure 2. Shear Test Set-up



Main fastener Seam fasteners

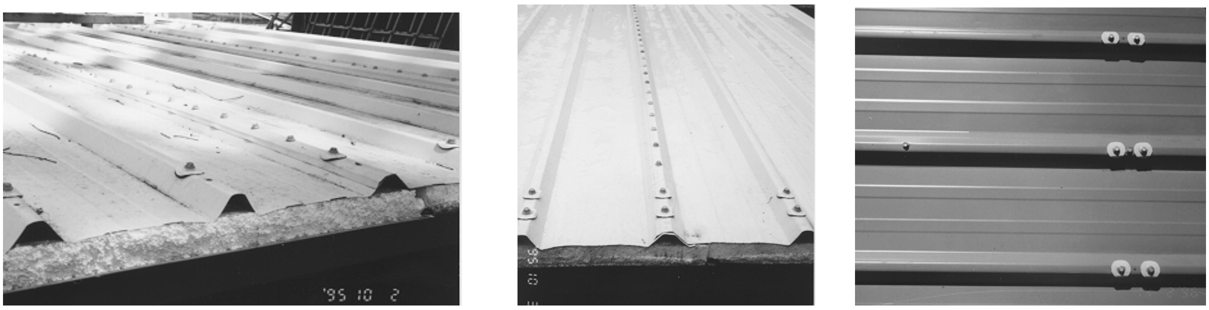
(a) Main and Seam Fasteners



Tearing along the lap joint Main fastener fracture Overall shear failure

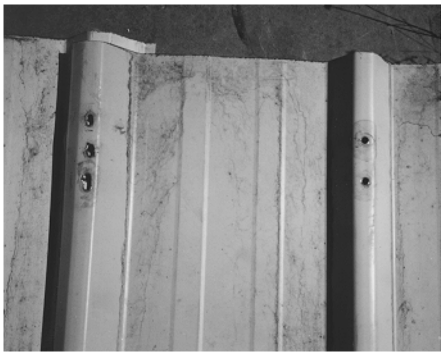
(b) Failure Modes of Test 1 Panel

Figure 3. Details and Failure of Test Panels



Seam fasteners at closer spacing   Test 5 Panel   Test 6 Panel

(c) Different Fastener Arrangements



(d) Tearing Failure at Main fastener

Figure 3.   Details and Failure of Test Panels

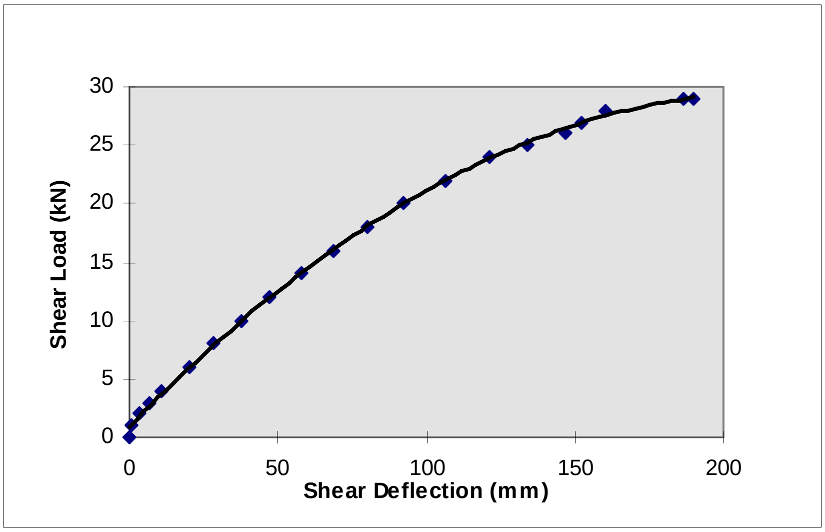


Figure 4. Typical Load-deflection Curve for Shear Tests

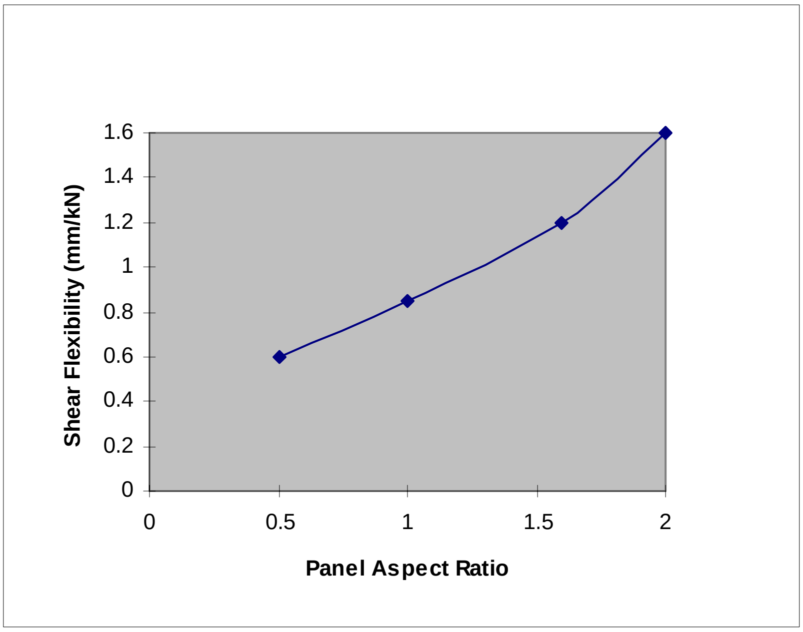
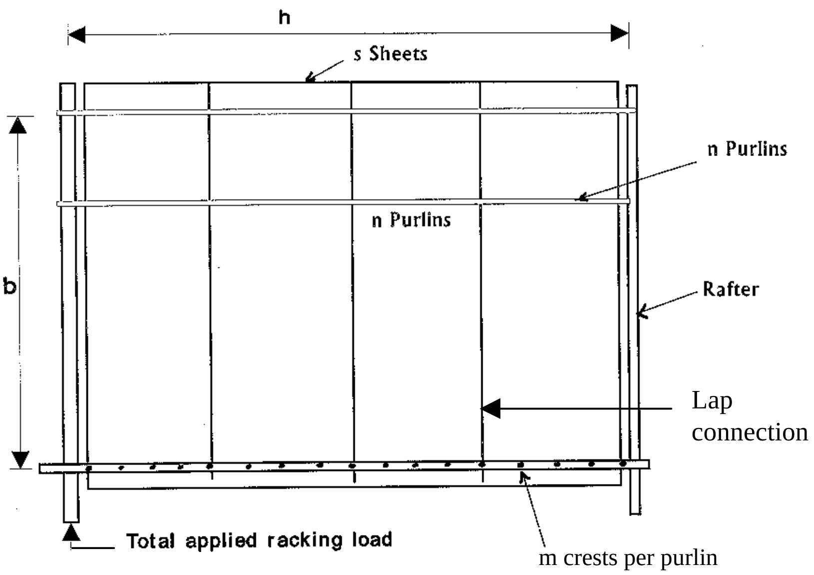
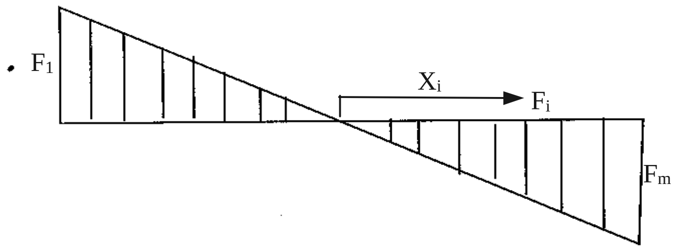


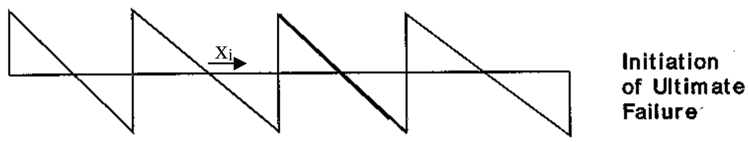
Figure 5. Effect of Aspect Ratio on the Shear flexibility of Sandwich Panels



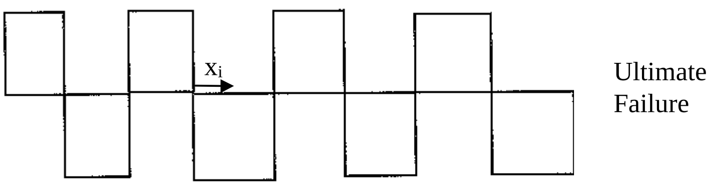
(a) Sandwich panel system



(b) Fastener load at onset of tearing

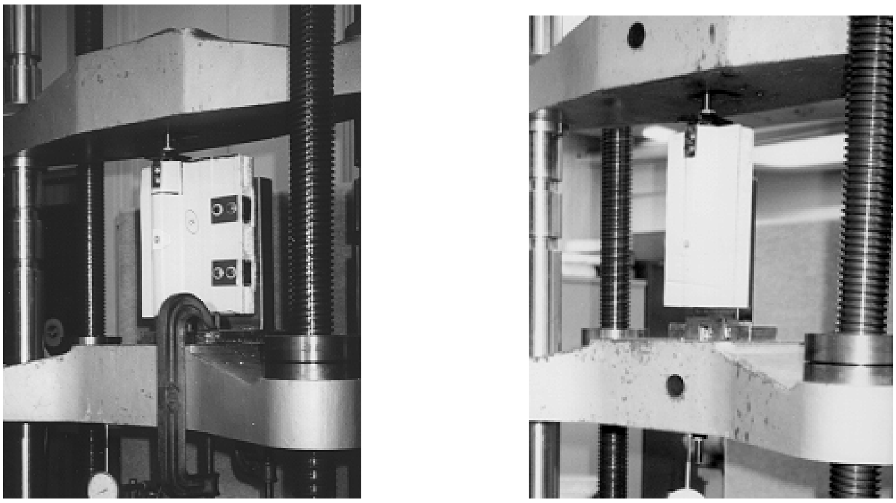


(c) Fastener load at Initiation of Ultimate Failure



(d) Fastener Load at Ultimate Failure

Figure 6. Distribution of Fastener Loads in a Sandwich Panel System



(a) Lap connections **(b) Edge connections**

Figure 7. Small Scale Connection Testing

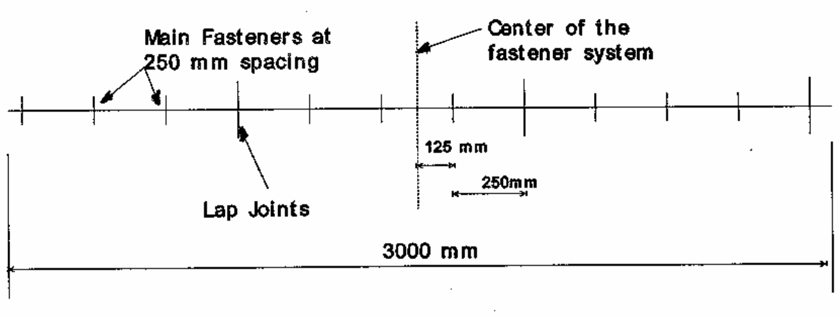


Figure 8. Locations of Fasteners on a Purlin in Test 5

Table 1. Analytical and Experimental Shear Strength Values

Test No.	Panel Size (m)	Panel Thick (mm)	Main Fasteners at crest	Main Fasteners at Lap crest	Lap fastenr Spacing (mm)	Aspect Ratio	Shear flexibility (mm/kN)	Onset Tearg Expt. (kN)	Onset Tearg theory (kN)	Ultimate Load Expt. (kN)	Ultimate Load theory (kN)
1	3 x 3	50	1	1	1000	1	0.95	6.0	6.8	10	9.9
2	3 x 3	50	1	1	500	1	1.0	7.5	7.6	10.5	10.7
3	3 x 3	50	1	1	250	1	0.88	8.5	9.1	11.5	12.0
4	3 x 3	50	1	1	100	1	0.88	13.0	13.8	16.5	16.3
5	3 x 3	50	2	2	100	1	0.90	20.0	20.1	27	23.3
6	3 x 3	50	2	3	500	1	0.90	20.0	20.1	24	23.3
7	3 x 6	50	2	3	500	0.5	0.50	31.0	30.1	36	35.1
8	4.7 x 3	50	2	3	500	1.6	1.20	18.0	17.2	27	26.1
9	6 x 3	50	2	3	500	2.0	1.60	19.0	18.0	29	26.4
10	3 x 3	75	2	3	500	1	0.80	17.0	16.2	24	23.3

Biodata

M. Mahendran

Dr Mahen Mahendran is an Associate Professor of Civil Engineering at Queensland University of Technology. He obtained his BScEng degree with a first class honours from the University of Sri Lanka in 1980 and his PhD from Monash University in 1985. He has since worked as an academic and applied researcher at various universities including James Cook University Cyclone Testing Station. His current research interests include behaviour and design of profiled steel cladding and sandwich panel systems under high wind forces, full scale behaviour of steel building systems and components, cyclone/storm-resistant building systems, and buckling and collapse behaviour of thin-walled steel structures.

S. Subaaharan

Mr S. Subaaharan was a postgraduate research scholar at Queensland University of Technology during 1996-98, where he completed a Masters degree in 1998. Prior to this, he has worked as a structural engineer in Sri Lanka and Australia. He is currently a structural engineer with Multiframe Pty Ltd. in Brisbane.