Shear Strength and Stiffness of Crest-fixed Steel Claddings

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SUMMARY Diaphragm action of **crest-fixed** profiled steel claddings is present in low-rise buildings whether the designer acknowledges it or not. For the designers to take advantage of the diaphragm strength of the crest-fixed steel claddings in the design of low-rise buildings in a similar manner to valley-fixed claddings, and to design the buildings based on the true behaviour rather than the assumed behaviour, shear/racking behaviour of the three trapezoidal and corrugated steel claddings commonly used at present was investigated using large scale experiments. Crest-fixed claddings (up to a maximum size of 6 x 6.2m) with different aspect ratio and fastening systems were tested to failure, based on which suitable shear strength and stiffness values have been proposed for these claddings as they are used at present. A simple analytical model combined with basic connection data from small scale experiments was used to predict the shear strength of tested panels. Currently attempts are being made to develop general design formulae to determine shear strength and stiffness of crest-fixed steel claddings.

1. INTRODUCTION

Low rise buildings such as commercial and industrial buildings and houses are subjected to uplift pressures on the roof claddings and lateral pressures on the wall claddings under wind loading. Lateral pressures on wall claddings lead to diaphragm action (stressed skin) of profiled steel claddings in the buildings even when they are crest-fixed. In Australia, where the roof claddings are crest-fixed in order to avoid possible water leakage, designers ignore the diaphragm action of **crest-fixed** profiled steel claddings in buildings, and instead provide separate bracing systems to carry the racking forces on the building. However, the 'real' behaviour of buildings under wind loads is affected by the diaphragm action of steel claddings whether the designer acknowledges it or not. Full scale tests on houses (Reardon and Mahendran (1)) and steel portal frame buildings (Moor and Mahendran (2)) have shown that both roof and wall claddings acted as diaphragms in transferring the racking forces to the end bracing walls. Therefore it is important that building design is based on the 'real' behaviour instead of an assumed behaviour. This means that the shear strength and stiffness of steel claddings should be included in the building design, and the claddings should be designed to withstand both wind uplift *and* in-plane racking (shear) forces which occur during high wind events such as storms and cyclones.

In Europe and USA where claddings are valley-fixed, diaphragm strength of claddings is included in building designs, and the required data on diaphragm strength of valley-fixed steel roof claddings and design methods are available (Davies and Bryan (3), Bryan and Davies (4)). In contrast, only limited research (Nash and Boughton (5), Boughton (6), Nash (7), Mahendran (8)) has been conducted to date and available design information is inadequate for crest-fixed steel claddings.

Brian and Davies (4) showed that the level of membrane stress in the valley-fixed roof sheeting is usually very low (10 to 20% of the normal bending stress), and that there is insignificant loss in static strength in racking or uplift due to the action of combined uplift and racking forces. Static tests on full scale houses and claddings (Mahendran(8)) have shown that these observations are also valid for crest-fixed claddings. Therefore it is believed that the overall behaviour of crest-fixed steel roof cladding under combined wind uplift and racking will not be affected, and the claddings can carry the required racking forces and transfer them to the bracing walls. However, the localised stress concentrations and the resulting local static and fatigue pull-through and tearing failures around the fastener holes (Mahendran (8,9,10)) could get worse during fluctuating uplift and racking forces, particularly for crest-fixed claddings. However, Mahendran's (8) fatigue tests have shown that these local effects do not get worse under combined loading unless the racking force component is rather high and fluctuating. Effect of combined racking and uplift loading on individual uplift and racking strength of claddings was insignificant as for valley-fixed claddings. Fluctuations of shear forces during high wind events are insignificant compared to uplift forces and it has been shown that cyclic shear loading does not reduce the static shear strength until load levels reach yielding (Beck (11)). All these observations give confidence to the claim that shear strength and stiffness of crest-fixed claddings could be taken into account in the building design.

The static and fatigue uplift behaviour of crest-fixed claddings has already been investigated and adequate design information is currently available (Mahendran (9,10); LBI (12)). Therefore in this investigation large scale static shear experiments were carried out to determine the required shear strength and stiffness of crest-fixed claddings commonly used in Australia (Figure 1). This paper presents the details of the shear / racking test rig, test methods and the results and compares them with some simple analytical predictions.

2. EXPERIMENTAL METHOD

2.1 General

Nash and Boughton (1981) were the first to investigate the behaviour of crest-fixed roof claddings under racking loads. They tested a number of roof claddings which were essentially designed to resist uplift loading only. Therefore insufficient fasteners were used to utilise the full shear capacity of the roof cladding. Accordingly, they observed tearing of roofing at the fastener holes and bending of fasteners, instead of buckling of roofing commonly observed in valley-fixed roof claddings (Bryan and Davies (4)). They used a smaller panel without lap joints and a larger panel with a lap joint, and developed a simple theory to predict the ultimate static failure load in shear (see Section 3). In this investigation a similar approach using large scale cladding panels with a number of lap joints was used. It is considered that if racking/shear capacity of cladding is to be increased, more fasteners have to be added along the lap joint, which will eliminate the shear failure along the lap joint. Seam/side-lap fasteners and shear connectors are always used with valley-fixed claddings to improve their shear capacity. However, the main objective of this investigation is to assess the shear capacity of the commonly used crest-fixed steel claddings as they are used at present, i.e., without seam fasteners and shear connectors. This was attempted because firstly it was considered that even without seam/lap fasteners and shear connectors it may be possible to obtain adequate shear capacity from these crest-fixed claddings. Secondly, additional cost due to seam fasteners and shear connectors can be avoided.

2.2 Steel Cladding Systems

A series of laboratory experiments simulating racking due to wind loading was carried out on three common trapezoidal and corrugated roof claddings used in the country (see Figure 1). All three claddings had a 0.42 mm base metal thickness and a minimum guaranteed yield strength of 550 MPa (G550 steel - measured yield stress is 690 MPa). The corrugated and trapezoidal Type B claddings were fastened at alternate crests to steel purlins by screw fasteners (LBI (12)) whereas trapezoidal Type A cladding was fastened at every crest as shown in Figure 1. The steel purlins were 200Z20 sections which are commonly used in industrial and commercial buildings. The screw fasteners had three different sizes, viz. No.14 x 50, No.12 x 50 and No.10 x 16, and were all of the self-tapping type. Maximum size of cladding panel tested was approximately 6.0 m x 6.2 m.

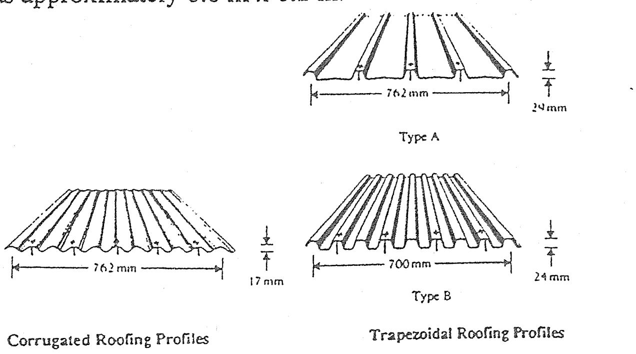


Figure 1. Steel Cladding Profiles considered in this investigation

2.3 Shear Test Rig

A shear/racking test rig was designed and constructed to be able to test crest-fixed steel claddings up to a maximum size of 6.0 x 6.2 m. The same principles used by Davies and Bryan (3) in the case of valley-fixed claddings were used for this purpose. Attempts were made to model a cladding system between two rafters as they are commonly used in the industrial and commercial buildings. As shown in the figures the test rig had two rafters, one of which was free to move under the racking load. Large number of Z-purlins (200Z20) were then fixed to these rafters at approximately 1 m spacing. This somewhat smaller spacing was chosen so that results can be used in another project (Moor and Mahendran (2)) on a full scale steel portal frame building which had the same purlins at that spacing. The purlins were fixed to the rafters via special joints which allowed free rotation of purlin under the racking load on one of the rafters. Once the sheeting was screw-fastened to the purlins as shown in Figure 2, the shear resistance was provided by the sheeting. Figure 2 shows the shear test rig used in this investigation.

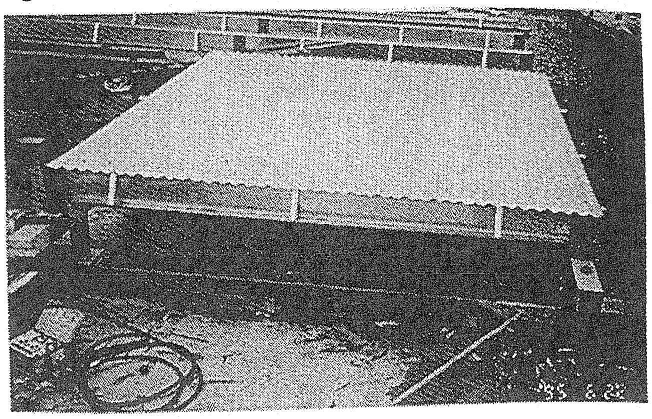


Figure 2. Shear Test Rig

The first test panel was a 6.0 x 6.2 m (approx. square) large scale cladding, but both the height (or width) and depth of the panel were varied in the following tests in order to investigate the effect of aspect ratio (height / depth = 0.3 to 3) on the stiffness of the panel (see Table 3 in Section 3).

2.4 Shear Test Method

During each test, a racking load was applied to one of the rafters using a hydraulic jack as shown in Figure 2. The load was increased in steps and the corresponding load cell and deflection readings at a number of points around the cladding were then taken until the cladding failed. In most cases, the failure was due to the lap failure when large shear deflections of the sheeting occurred without any increase in the load. In addition to the maximum load, the load at which the sheeting commenced tearing at the laps was also noted.

2.5 Small Scale Connection Testing

It was found that large scale claddings systems failed due to sheet tearing at the lap joints because of insufficient lap fasteners (see Section 4.1) and at the edge fasteners when there were no lap joints (Mahendran (8)). The failure loads of these large scale cladding systems can be predicted using a simple analytical method (described in Section 3) provided the basic tearing load at the lap joint and at the edge fastener are available. Therefore small scale cladding connection models were tested to determine these tearing loads using a method similar to that of Nash and Boughton (5). Figures 3 and 4 show the simple tension test set-up used to determine the basic tearing load at the lap joints and at the edge fastener, respectively. Two sheets of approx. 150 mm x 300 mm were used with two screw fasteners for the lap joints and with four fasteners for the edge fastener. It was assumed that the applied load was shared equally among the screw fasteners. Tests were continued until large tearing deformations occurred without any increase in load. A minimum of three tests was conducted in each case, and the results are given in Section 4.2. To date these tests were conducted only for the combinations of cladding profile and screw fasteners used in the large scale shear tests. However, tests will be continued to include other combinations.

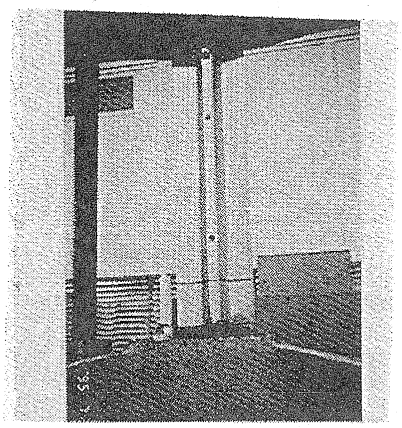


Figure 3. Test Set-up to Determine Tearing Load at the Lap Joints

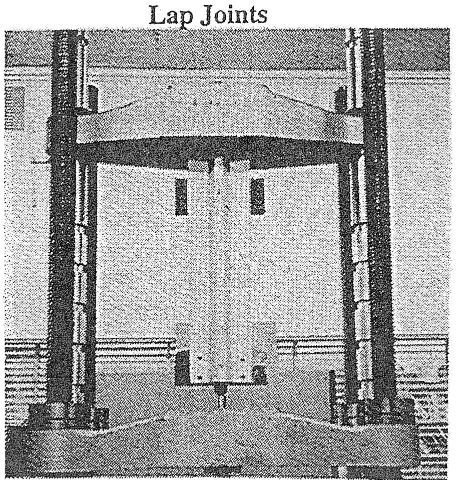


Figure 4. Test Set-up to Determine Tearing Load at the Edge Fastener

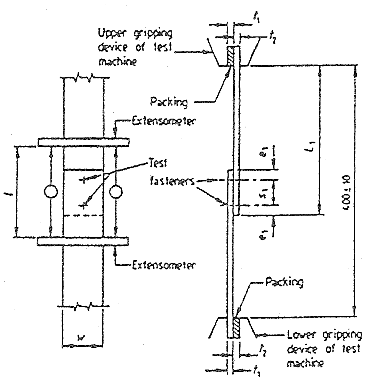


Figure 5. Test Set-up recommended by Davies and bryan (3)

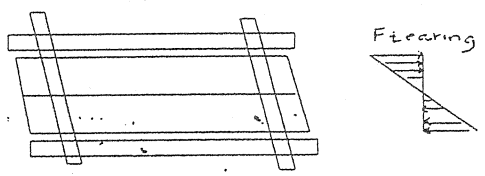
Davies and Bryan (3) recommend a simple shear test of a lapped joint as shown in Figure 5. This test set-up does not include the effect of cladding profile or type of fixing. It assumes that the tearing load based on hole bearing ($F_{tearing}$) depends only on the thickness (t) and ultimate tensile strength of steel (f_u) and screw shaft diameter (d). A design formula (Equation (1)) has been developed based on this (Toma et al. (13)) for inclusion in Eurocode 3. The corresponding American design formula (Pekoz (14)) recalibrated to suit the USA practice is given by the same equation, but with a different k value. However, in this investigation, testing was based on Figures 3 and 4, but test results were then compared with the predictions from Equation (1). Since this design equation is for lower grade steels (G300), it is necessary that their accuracy is verified for high strength steels (G550).

$$F_{tearing} = k\ t\ d\ f_u \qquad (1)$$

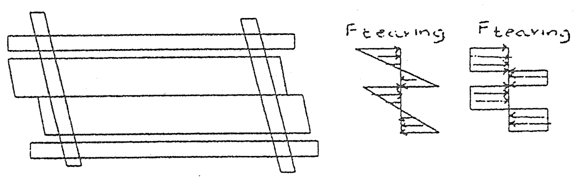
where $k = 2.1$ European design formula
$k = 2.7$ American design formula

3. ANALYTICAL METHOD

Nash and Boughton (5) developed a simple theory which 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 has produced reasonable predictions for their experiments with corrugated roofing (Nash and Boughton (5)) and for the recent experiments on corrugated roofing without laps (Mahendran (8)). In this investigation the same theory was used to predict the failure loads, and therefore for the sake of completeness, the basis of this theory and the formulae are presented here.



(a) Onset of tearing



(b) Ultimate failure

Figure 6. Assumed Failure Modes of Cladding Panel and Force Distribution for the Analysis

This theory is based on a number of assumptions which are described in this section. Figures 6 (a) and (b) show the onset of tearing and the ultimate failure modes of cladding panels and the associated force distribution at each purlin 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. If the shear forces at the fasteners on each purlin are F_1, F_2,---, F_m where m is the number of fasteners at each purlin, the maximum shear force is on the edge fasteners (F_1 or F_m). Despite the complexity of the structure, it is assumed that for elastic conditions the force at the fastener is proportional to the distance from the neutral axis, that is, the centre of each panel. Therefore, the force at the ith fastener = $F_m \cdot x_i / 0.5\ h$ where x_i is the distance from the neutral axis and h is the height (or width) of panel. If there is no lap joint, onset of tearing occurs when the edge fastener load F_m reaches the tearing load $F_{tearing}$. This tearing load depends mainly on the thickness of sheeting and the diameter of the screw shaft and can be determined using simple tension tests on small scale connections. In this investigation it was determined using a test method similar to that used by Nash and Boughton (5) (see Section 2.5).

By equating the applied moment of Ph/n to the resisting moment $2\ F_m\ \Sigma x_i^2 / h$ provided by the forces on the fasteners and assuming $F_m = F_{tearing}$, the onset tearing load can be found.

$$P_{on} = 2\ n \sum_{i=1}^{m} x_i^2\ F_{tearing} / h^2 \qquad (2)$$

In the presence of lap joints,

$$P_{on} = n \sum_{i=1}^{m} x_i^2\ F_{tearing} / h \sum_{i=1}^{p} x_i \qquad (3)$$

where m is the total number of fasteners on each purlin
p is the number of fasteners from the edge to critical lap

For the case of ultimate failure, it was assumed that tearing occurs at all the fasteners. All the lap joints have completely failed and sheets behaved rather independently. The tearing that occurs at the laps and at other locations was assumed to occur at approximately the same load $F_{tearing}$. This was determined using small scale tests (see Section 2.2). Therefore the resisting moment is number of sheets s times the resisting moment provided by each sheet $F_{tearing}\ \Sigma x_i$. Equating this to the applied moment Ph/n gives the ultimate failure load of the panel P_{ult}.

$$P_{ult} = n\ s \sum_{i=1}^{m} x_i\ F_{tearing} / h \qquad (4)$$

where s is the number of sheets

Equations (2) to (4) were derived for crest-fixed claddings which undergo tearing at the lap joints or edge fasteners, but they are also applicable to valley-fixed claddings provided appropriate $F_{tearing}$ values are used.

Davies and Bryan (3) and BS5950 (15) have developed simple design formulae to determine the ultimate failure loads of valley-fixed cladding panels for a range of failure modes. For the case of tearing between sheeting and purlin fasteners, the following formula (Equation (5)) has been recommended. Although Equation (5) was derived for valley-fixed claddings, it can be used for the crest-fixed claddings tested here with appropriate F_p values.

$$P_{ult} = \beta_2\ n_p\ F_p \qquad (5)$$

where β_2 = Factor to allow for the number of sheet to purlin fasteners per sheet width
n_p = Number of purlins
F_p = Design strength of an individual sheet to purlin fastener connection in kN (same as $F_{tearing}$).

4. RESULTS AND DISCUSSION

4.1 Connection Test Results

Tables 1 and 2 present the results obtained to date. The columns $F_{tearing}$ in Tables 1 and 2 give the characteristic value based on 95% confidence level.

Table 1. Tearing Failure Loads at the Lap Joints

Cladding Profile bmt=0.42mm	Screw Shaft Dia. (mm)	Measured Failure Load Tearing	Ult.	$F_{tearing}$ (kN)	k Eq.1
Trapez. A	5.16 (No.14)	1.30	1.84	1.27	0.9
Trapez. B	5.16 (No.14)	1.34	1.88	1.28	0.9
Corrugated	5.16 (No.14)	1.30	1.72	1.27	0.9
Trapez. A	4.64 (No.12)	1.20	1.66	1.17	0.9

Table 2. Tearing Failure Loads at the Edge Fasteners

Cladding Profile bmt=0.42mm	Screw shaft Dia. (mm)	Measured Failure Load (kN)	$F_{tearing}$ (kN)	k Eq. 1
Trapez. A	5.16 (No.14)	1.29	1.26	0.8
Trapez. B	5.16 (No.14)	1.33	1.27	0.9
Corrugated	5.16 (No.14)	1.29	1.26	0.8
Trapez. A	4.64 (No.12)	1.20	1.17	0.9

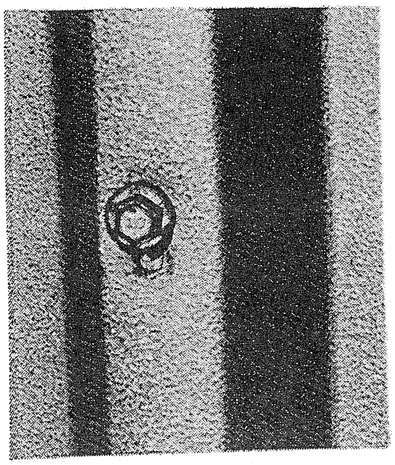
As seen from the results, tearing loads do not appear to depend on the profile geometry. This is similar to the valley-fixed claddings and thus Equation (1) can be used for crest-fixed claddings with an appropriate k value. Tables 1 and 2 present the k value in each case. It can be seen that the mean tearing failure loads (and thus k values) are approximately the same at the lap joints and edge fasteners. This verifies the assumption used in the derivation of Equations (2) to (4). Measured mean tearing load was used in the calculation of ultimate failure load of each panel, and they are compared with experimental results in Table 3. However, for design purposes a k value of 0.8 can be used for tearing load for all the crest-fixed claddings. Further tests are required to refine this value.

4.2 Shear Test Results

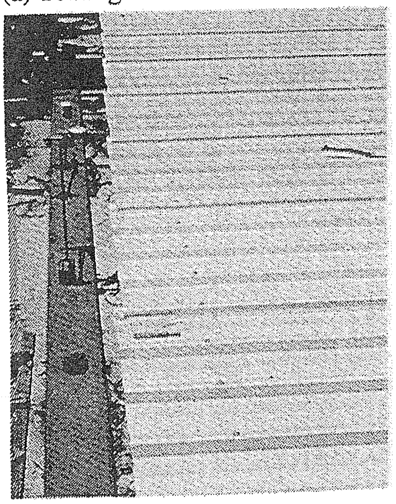
Table 3 presents the results of 11 shear tests on claddings of different size and fastening. Results include both the onset of tearing load and the ultimate load in kN. As anticipated, all the claddings failed due to tearing along the lap joints except when additional fasteners were used at the lap joints. Figure 7 shows this typical failure of claddings observed during the tests. Both crest-fixed and valley-fixed claddings failed in a similar way, but the latter had greater strength. Test 11 was carried out on trapezoidal claddings which were fastened inadequately (fasteners neither located in the middle of the crests nor vertical). Despite this, the strength and stiffness have not decreased.

Load-deflection curve for the 3.0 m x 3.1 m crest-fixed trapezoidal-Type A cladding is presented in Figure 8. Elastic stiffness of each cladding system was obtained and is included in Table 3. These elastic stiffness values of claddings can be used in the three-dimensional analysis of buildings attempting to include the stiffening effect of claddings (Moor and Mahendran (2)). From the first five tests the effect of aspect ratio on stiffness of trapezoidal cladding - Type A was investigated, and their results are shown in Figure 9. Similar trend was anticipated for other cladding profiles. This investigation has produced stiffness values only for the commonly used claddings. Davies and Bryan (3) has a general formula to predict the stiffness of valley-fixed claddings. In a similar manner, attempts have to be made to develop general design formulae for the shear stiffness/flexibility of crest-fixed cladding systems.

Analytical predictions for the ultimate load using Nash and Boughton's (5) theory and measured basic tearing loads agreed well with the experimental results. Davies and Bryan's (3) formula appears to produce conservative results. Nash and Boughton's method can also be used to determine the onset of tearing load, and its predictions agreed reasonably well with experimental results (see Table 3). Ultimate and onset tearing loads were not calculated for the claddings with valley-fixing, more lap screws and inadequate fixing method (Tests 7, 8, 11) since basic tearing loads are not available for these cases. Therefore these analytical methods with appropriate basic tearing loads ($F_{tearing}$ in Tables 1&2) are recommended to determine the design strength of both crest- and valley-fixed claddings undergoing tearing failure.



(a) Tearing at the fastener holes



(b) Steel Cladding after the Test

Figure 7. Typical Failure Modes Observed during Shear Tests

Table 3. Shear Test Results for Steel Claddings

Test No.	Cladding Panel	Screw Fasteners	Type of Fixing	Panel height h (m)	Panel Depth b (m)	No. of Purlins	No. of Sheet	Expt Stiffness kN/mm	Onset Tearg Load (kN) Expt.	Theory	Ultimate Load (kN) Expt.	Theory
1	Trapez. A	No.14	Crest	6.0	6.2	7	8	0.25	11	9.2	20	19.7
2	Trapez. A	No.14	Crest	6.0	3.1	4	8	0.12	5.5	5.2	10	11.2
3	Trapez. A	No.14	Crest	6.0	2.0	3	8	0.06	4.5	3.9	7	8.4
4	Trapez. A	No.14	Crest	3.0	6.2	7	4	0.45	10	9.3	19	19.7
5	Trapez. A	No.14	Crest	3.0	3.1	4	4	0.25	6	5.3	10	11.2
6	Trapez. A	No.12	Crest	3.0	3.1	4	4	0.22	5	4.8	9	10.1
7	Trapez. A	No.14	Crest more lap screws	3.0	3.1	4	4	0.25	7	-	16	-
8	Trapez. A	No.10	Valley	3.0	3.1	4	4	0.23	6.5	-	13	-
9	Trapez. B	No.14	Crest	3.0	3.1	4	4	0.15	6	5.0	11	11.1
10	Corrugated	No.14	Crest	3.0	3.1	4	4	0.23	6	4.9	13	12.6
11	Trapez. A	No.14 & 12	Crest inadeq. fixing	3.0	3.1	4	4	0.21	6	-	10	-

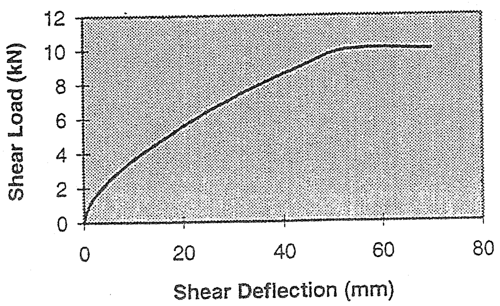


Figure 8. Typical Load-deflection Curve for Shear Tests

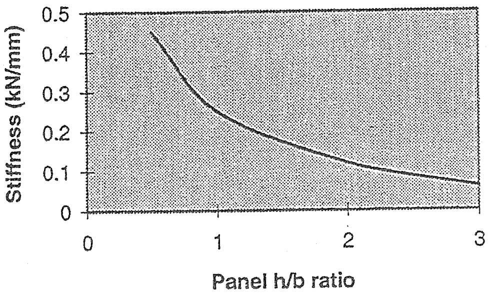


Figure 9. Effect of Aspect Ratio on the Shear Stiffness of Trapezoidal Cladding (Type A)

6. CONCLUSIONS

The following conclusions have been drawn from this investigation.

1. Shear / racking behaviour of crest-fixed steel claddings used commonly in Australia was studied using large scale experiments. Both shear strength and stiffness were determined from these experiments. The failure of the claddings was in most cases due to tearing failure at the lap joints. The basic tearing loads for the lap joints and also for the edge fastener were also determined using small scale connection tests. They were then used to develop simple design formulae. An available theory (Nash and Boughton, 1981) was then used adequately to predict the strength of these claddings using the basic tearing loads.

2. Designers could use the basic tearing load formulae developed in this paper and the available theory (Nash and Boughton, 1981) to determine the shear strength of steel cladding systems used without shear connectors and lap fasteners. Shear stiffness values reported could be used in three dimensional analyses of buildings incorporating the effects of commonly used steel claddings. More experiments will be conducted in the future, based on which simple design formulae will be developed for shear strength and stiffness of crest-fixed claddings.

3. Despite the fact the claddings were crest-fixed and had no shear connectors and seam fasteners, they appeared to have considerable shear strength and stiffness. They may be adequate for designers to include diaphragm action of such claddings in building design. Even the cladding which was installed inadequately showed only a slight reduction in strength and stiffness.

4. Limited number of tests on valley-fixed claddings revealed that the shear strength was increased by about 50%, but stiffness was not. This may have been because shear connectors and seam/lap fasteners were not used.

7. ACKNOWLEDGEMENTS

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