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The use of small scale model testing to compare connection methods of steel purlins

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Abstract. Testing of steel roof purlins is usually performed on full scale models in large vacuum test rigs. To undertake a comparison between web cleat connected purlins and flange bolted purlins a series of tests were performed on a 1:4 small scale model vacuum test rig. Various modelling issues need to be addressed to ensure reasonable comparison with actual constructed roof framing methods but still be suitable for an economical comparison between the connection methods. Model test results were supported by, and found to be in reasonable agreement with, deflection predictions from computer models based on finite element methods. This paper discusses the testing methods adopted and the value of small scale model testing programs as a means of obtaining comparisons between framing options.

Key words: cleats; connection; flange; purlins; roofs; scale model; steel; tests.

1. Introduction

Usual Australian practice is to connect steel purlins to the main roof structure by bolting to cleats welded on the beams. An alternate method of connection is to bolt the bottom flange of the purlins directly to the beam top flange. While both methods can be readily modelled and analysed by finite element or finite strip methods of analysis, it is necessary to compare these models to experimental results. It is also necessary to obtain various stiffness factors from experimental work for inclusion in such computer models.

Full scale testing of cleat mounted purlins subject to gravity and uplift loads can be economically undertaken in a vacuum test rig. A recent testing program at the University of Western Australia by Urquhart (1996) undertook a comparative study between flange and web cleat connected purlins, where the purlins were attached to screw fixed sheeting. These tests were conducted at a 1:4 scale with similar scale tests to establish parameters for use in the computer modelling of the two connection methods. Computer models based on finite element methods were used to predict deflected shapes for comparison with the experimental results, and buckling moments for both cleat and flange connected purlins.

A series of 15 tests was performed at 1:4 scale using the vacuum test rig. This rig provided uniformly distributed loads which simulate purlin uplift loads. A two span (1500 mm) continuous model purlin was tested with and without bridging for both flange bolted and cleat connected purlins. The tests are summarised in Table 1.

This paper discusses the issues that need to be considered in reduced scale testing programs.

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Table 1 Summary of test configurations

Test no	Bridging	Support system	Test no	Bridging	Support system
1	0-0	Flange	9	0-0	Flange
2	0-0	Flange	10	1-1	Flange
3	0-0	Flange	11	1-1	Web
4	0-0	Flange	12	1-1	Flange
5	0-0	Web	13	1-1	Flange
6	0-0	Web	14	1-1	Web
7	0-0	Web	15	1-1	Web
8	0-0	Web			

Note: 0-0 refers to no bridging
1-1 refers to one row of bridging in each span.

2. Reduced scale testing

Reduced scale testing programs are an acceptable method of testing for experimentally based design, provided consideration is given to the:
(1) principles of dimensional similarity;
(2) differences in yield stress;
(3) method of load application (Standards Australia 1988, 1981).

Practical issues must be considered when selecting sizes for scale model purlin tests. Various limitations apply that physically affect size options - not the least of which is ensuring the model is still large enough to be easily built and tested. Other issues include:

(1) Available base metal thicknesses and the available steel grades of sheet steel supply. Scaling of purlin sizes can be limited by suitable scaling of wall thicknesses. The ratio between available sheet steel and real purlin thicknesses can determine overall size options.
(2) Thinner sheet steel invariably has a higher yield strength than normal purlin sheet feed. Associated with a higher yield. Lack of ductility could result in different failure modes.
(3) Screw head sizes should reflect sizes of bolts for flange and cleat connections and maintain relative shear capacities. Screws must still be of a practical size for erection.
(4) Available sheeting profiles should model normal roof sheeting profiles. These profiles must have properties that are not out of proportion with the selected model purlins.
(5) Full size purlins are roll formed, while model sections invariably are pressed into the required profile shape. Associated with this are size limitations, e.g., overall lengths, lip lengths and a likelihood of increased variation in purlin dimensions.

3. Test layout

Trimdek$^{(TM)}$ roof sheeting screw fastened to Z150 purlins, spanning 6000 mm, was selected as the full scale equivalent of the test configuration.

3.1. Purlins

Selection of the model purlin size was dependent on the available sheet base metal thickness. At a nominal 1:4 scale model ratio, Z15019 purlin sections could be formed from available 0.48 base metal thickness sheet steel. See Fig. 1.

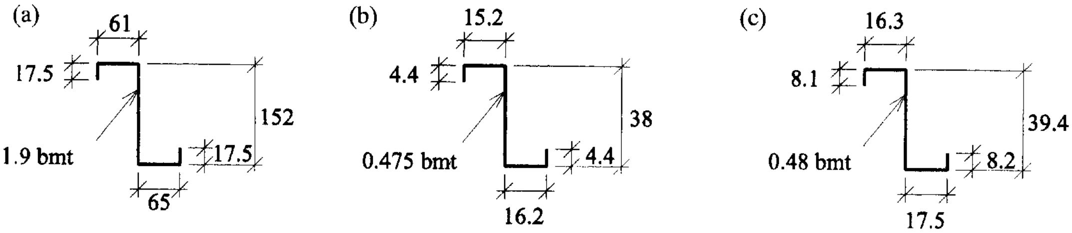


Fig. 1 Scaled purlin model dimensions. (a) Full Size Z15019, (b) Exact 1:4 scale of Z15019, (c) Average model dimensions

With a 1:4 scale based on available sheet thicknesses, the purlin model dimensions were appropriately scaled to represent the Z15019. Roll forming of sections of this size is not an option and all sections had to be pressed from 0.48 mm sheet. The required 4.5 mm lip lengths were too small to be pressed, and larger lip sizes were used. The impact of the slightly larger lips was considered negligible in tests developed for comparisons between connection methods. The larger resulting I_y, J, and I_w properties did result in lower lateral deflections than would be expected in strict 1:4 scaled sections. As the problem applied to both connection methods, the comparision was still acceptable.

With pressed shaping rather than roll forming, corner radii were sharper than in a conventional purlin. However, the difference in a scaled 1.25 mm radius and the sharp corner bend is not significant particularly as corner radii are often ignored in the development of section properties (Bradford 1990, Toma and Wittemann 1994).

Pressed sections result in a greater variation of purlin dimensions than in the repeatable roll forming process. To account for the higher variation in pressed purlin dimensions the I_x and I_y for each purlin were normalised against the "average" purlin (based on the dimensions of the 60 purlin samples used in the complete test program). Vertical deflections for each purlin were adjusted by the ratio of the purlin's I_x to the average dimension purlin I_x value. Horizontal deflections similarly adjusted by the I_y ratio. No attempt was made to correct for variations in J or I_w values for each individual purlin.

Test purlins were pressed from G550 steel plate sheeting complying with AS 1397-1984 (Standards Australia 1984) with a yield stress of 550 MPa. Tension coupon tests gave failure stresses between 680 and 690 MPa. The near brittle failure of the samples showed the yield point and tension failure point to be almost the same. As such, the purlins did not exhibit the same ductile failure patterns as would be expected from normal 450 MPa full size purlins.

3.2. Sheeting

Sheeting lateral restraint is high when the sheeting system is anchored in-plane and the sheets have positive connections that inhibit sliding. Sheeting torsional restraint is dependent on its out-of-plane flexural stiffness and is significant, particularly in unbraced purlin systems (Hancock, Celeban and Healy 1993). A problem remains in assessing suitable values for the rotational stiffness, especially when considering the possible variations in sheeting profile shape.

The torsional restraint provided to the free purlin flange is dependent on three separate stiffnesses, namely the:

(1) rotational stiffness of the connection between the sheeting and the purlin;
(2) distortion of the cross-section of the purlin;

(3) bending stiffness of the sheeting (Hancock 1994).

As the Trimdek bending flexibility can usually be neglected in comparison to the purlin/sheeting connection flexibility and the distortion of the purlin cross section (Toma and Wittemann 1994), accurate modelling of its stiffness was not a major concern in model selection.

For the scale model tests Panelrib$^{(TM)}$ sheet, with a profile pitch of 50 mm, was adopted. This made it possible to fix at each "crest" at the required centres with No 8-15 × 15 mm hex slot head needle point (nominal 4.2 mm thread diameter) self tapping tek screws, with a neoprene washer under the head. Intermediate sheet connectors were not needed due to the relative size of sheet to normal panel widths.

Screw sizes were selected by physical size considerations rather than a true 1:4 ratio. Absolute rescaling of tek screw heads was impractical. The relatively larger screw head size may have provided a higher sheeting/purlin connection stiffness, than would occur in full scale conditions. Uniformity between the tests was considered more appropriate than a strict 1:4 modelling of the sheeting/purlin connection. The same fixing condition was also used in testing for rotational stiffness values for use in computer models.

3.3. *Test rig and frame layout*

A 3100 mm long, 1140 mm wide and 200 mm deep vacuum chamber was constructed with solid panels on the bottom and side walls with the top forming the test plane. The tested panel consisted of profiled metal sheeting held in the top horizontal plane by four purlins, representing a complete roofing system.

The Panelrib was supported by four lines of purlins spaced at varying centres to achieve an equal uniformly distributed loading to all purlins. The aim of varying the purlin spacing was to achieve a uniform deflection across the total roof area. Exterior purlins were located 110 mm from the sheeting edges and the interior purlins spaced at 200 and 360 mm centres (refer Fig. 2). This spacing arrangement was selected by treating the sheeting as a continuous beam over four supports. The support locations were adjusted until the loads were approximately the same for each purlin.

Purlins were oriented in an east-west arrangement and marked A to D from north to south. Each purlin span is individually designated by a test number, position and either an east or west span, e.g., 9A-W denotes Test 9, purlin A, western span.

The Panelrib was kept free of the sides and ends of the chamber to prevent constraint of the roof panel deflections. Plastic sheeting was placed between the purlins and the Panelrib as a seal, to assist in maintaining pressure differences, and fixed at the edges in a manner to avoid constraining roof panel movement.

The purlin sections were connected to 75 × 40 mm channels, acting as rafters. Uplift loads were simulated by progressively pumping the air out of the chamber and allowing the pressure differential to load the roof sheeting and purlins.

3.4. *Connection between purlin and beams*

Most purlin connections are made with proprietary M12 purlin bolts that incorporate 30 mm diameter washers in the bolt head and nut (Lysaght Building Industries 1993). The 30 mm washer dimension is the critical dimension to model, as it dictates the amount of rotational restraint provided by the bolt to the purlins. The model purlins were connected to the channel, through the bottom flange, using 4.0 mm bolts with 7.9 mm diameter circular

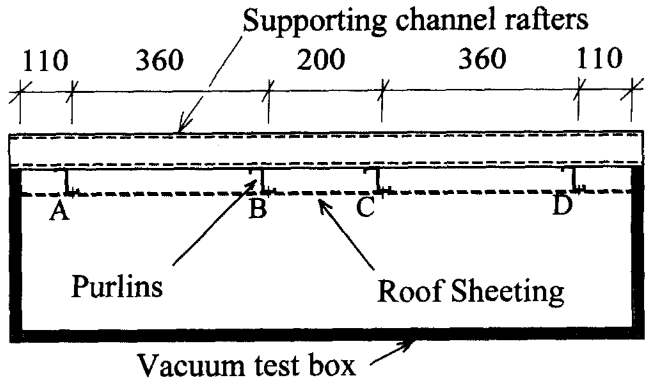


Fig. 2 (a) Purlin layouts - section

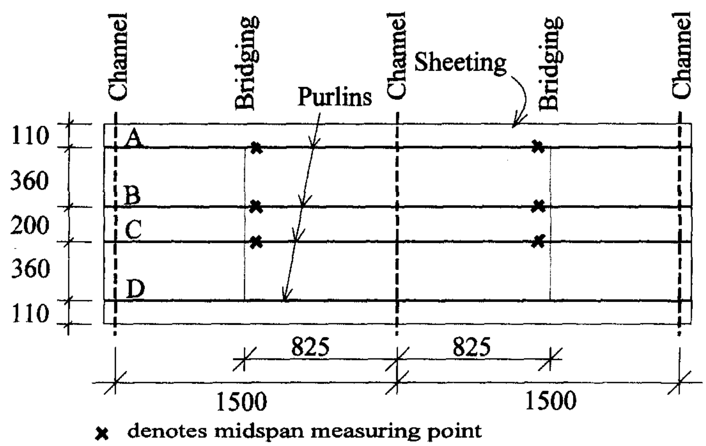


Fig. 2 (b) Purlin layouts - plan

heads, the closest size available at 1:4 scale. Two bolts were used at an 18 mm pitch to model the standard 70 mm pitch for purlin connections.

For the control test cases, purlins were bolted to correctly scaled cleats with two 5.0 mm bolts. A slightly larger bolt, than direct 1:4 scale, was used to avoid bolt shear or bearing failure problems, which are not governing criteria for the purlin configuration being modelled (Lysaght Building Industries 1993). Cleat bolt holes were set at standard gauge lines (i.e., 60 mm) scaled to 15 mm.

3.5. Bridging

Bridged and unbridged 6000 mm spans were modelled in the test program. The unbridged length of 6000 mm is beyond the AS 1538 recommended limit for unbraced purlin lengths of 20D, where *D* is the depth of the purlin.

To provide the required rotational restraint, 12 mm diameter threaded rods were used as bridging elements. A nut and washer were fixed either side of the web and tightened into position, to act as the bridging connection.

Bridging spanned between purlins and was not connected to external supports in recognition that screw fixed sheeting normally possesses sufficient diaphragm action for the bracing force to be transferred through the roof system to the purlin cleats (Hancock, Celeban and Healy 1993).

3.6. Main test procedure

Simulated uplift loads were applied by progressively reducing the air pressure in the vacuum box. Pressure levels were controlled by a manually operated vent at one end of the vacuum box and measured by a water manometer placed at the opposite end. Pressure in the vacuum box was initially reduced in approximately 0.2 kPa increments and then in 0.1 kPa increments closer to the expected failure loads.

Before the application of load, and then at each pressure increase, the horizontal and vertical deflections of the purlin midspan points were measured using a Mitutoyo vernier dial scale (increments marked to 0.05 mm). Horizontal and vertical deflections were measured at the purlin's top flange (in the test position).

Deflections were measured at midspan of both the east and west spans of purlins "A" and "B", representing an internal and external purlin. After the first two tests, deflections were also measured at purlin "C".

4. Repeatability of results from tests

In most cases, deflection readings taken from each of the purlins in any one test exhibited a high degree of repeatability. For example, see Figs. 3 and 4.

Failure pressures for the unbridged flange connected purlins (Tests 1, 2, 3, 4 and 9) ranged from 3.87 to 4.12 kPa with an average of 4.00 kPa while the unbridged web cleat connected purlin (Tests 5, 6, 7 and 8) ranged from 3.83 to 4.18 kPa, with an average of 4.02 kPa.

In the bridged purlin tests, the flange connected purlins (Tests 10, 12 and 13) had a nominally higher average failure pressure 4.18 kPa (range 4.06 to 4.32 kPa) compared with the web cleat connected purlins (Tests 11, 14 and 15) at an average of 4.14 kPa (range 4.06 to 4.22 kPa). The difference in the bridged and unbridged purlins, irrespective of the connection method, was far less than expected. The difference is not statistically significant,

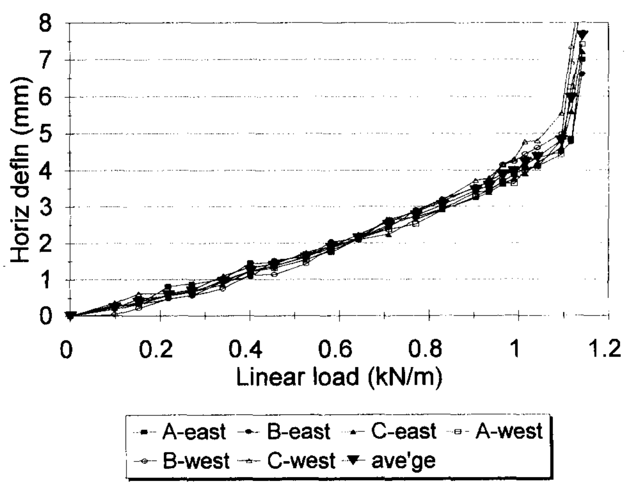


Fig. 3 Typical horizontal deflection v's purlin line load Test 7 - unbridged web connected

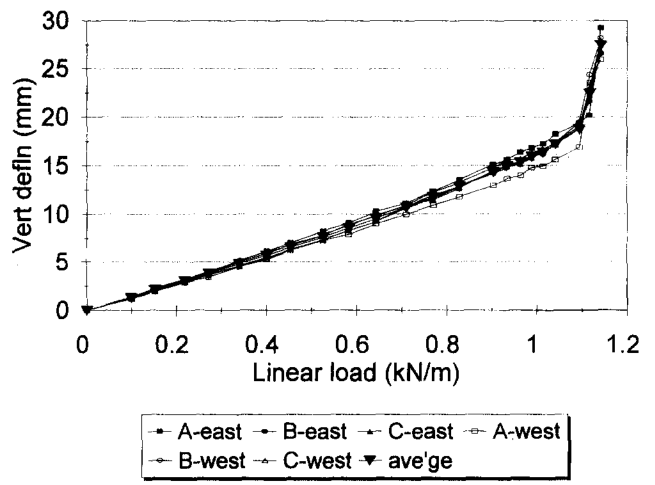


Fig. 4 Typical vertical deflection v's purlin line load Test 7 - unbridged web connected

as the variation in the results within any one test series exceeds the variation between the averages of the different series.

5. Computer modelling and spring supports

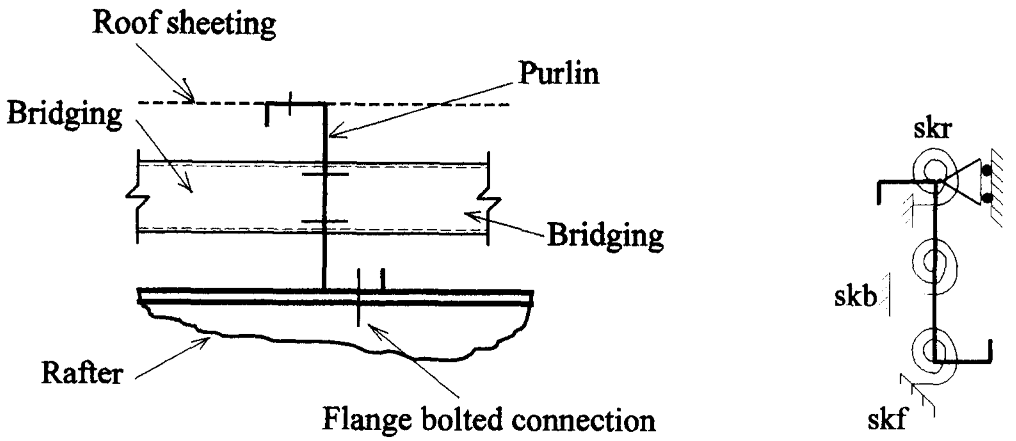
A computer modelling based on cubic warping rotations (Chajes 1974) was used to model the purlin tests. The model coupled rotation and displacement via the equations of lateral torsional buckling (Kavanagh 1996) and a solution was sought by applying incremental loads. The effective section, to AS 1538 was computed at each load increment.

The model shown in Fig. 5 required calibration of the 'effective' spring constants for the roof, web and flange restraints. These were obtained by tests shown in Fig. 6. The computer buckling model predicted buckling loads significantly greater than the yield load of the member. By applying vertical loads to the model, predictions of horizontal and vertical deflections for the cleat and flange bolted purlins were also calculated.

The average of the beam/purlin flange connection tests gave a rotational stiffness value ("skr" in Fig. 5) of 25.5 Nm/rad. Panelrib sheeting/purlin connection test segments yielded a rotational stiffness value ("skb" in Fig. 5) of 16.9 Nm/rad for the 100 mm long samples. By adding the flexibilities of the sheeting spans to the flexibility of the sheeting connection/purlin web system, the rotational stiffness provided by the roof sheeting system ("skf" in Fig. 5) was taken as 160 Nm/rad/m length of panelrib sheeting. Averages for the bridging/purlin connection tests gave a bridging rotational stiffness of 150 Nm/rad.

Buckling moments for the scale model purlins, and the exact 1:4 scale Z15019 purlin, were determined, by taking the lowest mode value calculated by the programs, for the various boundary conditions. Results are shown in Table 2.

From Table 2, it can be seen that the theoretical lateral torsional buckling load is well in excess of the actual experimental failure loads. The failure loads are remarkably similar for all cases, indicating that all specimens failed by general yielding of the cross section. Calculated stresses (based on the net section properties) produced 632 MPa under vertical bending, and 860 MPa under combined vertical and horizontal bending. The calculated stresses confirm the



(a) Purlin restraints (b) Spring models

Fig. 5 Spring stiffness models for purlin boundary conditions

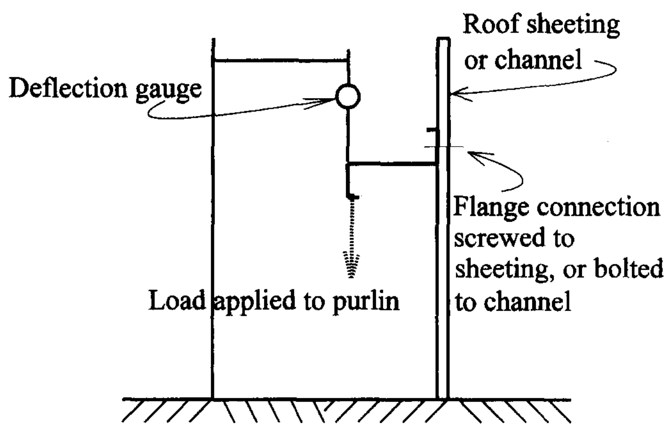


Fig. 6 Layout for rotational stiffness tests

Table 2 Lateral torsional buckling loads and experimental failure loads

Lateral torsional buckling loads and experimental failure loads				
Case	Buckling load (kN/m)	Buckling moment (kNm)	Failure load (kN/m)	Failure moment (kNm)
Web no bridge	3.80	1.07	1.06	0.30
Flange no bridge	2.99	0.841	1.06	0.30
Web one bridge	3.91	1.10	1.06	0.30
Flange one bridge	3.04	0.856	1.06	0.30

observed yield behaviour in Figs. 7 and 8.

6. Comparison of results

The difference in the average capacities of the bridged and unbridged purlins, irrespective of the connection method, was far less pronounced than expected or suggested by purlin load

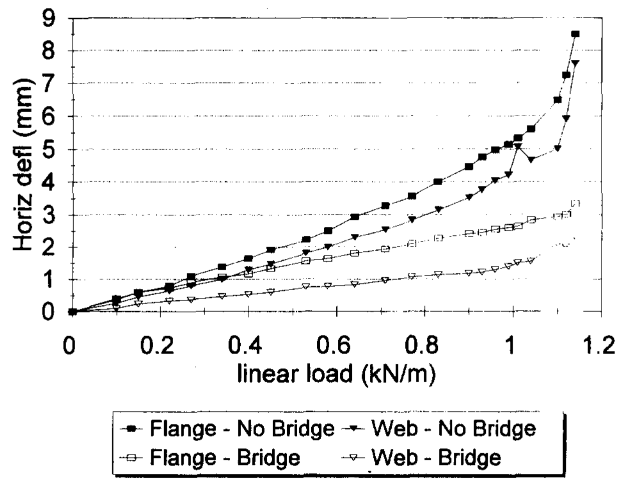


Fig. 7 Comparison of average horizontal deflections

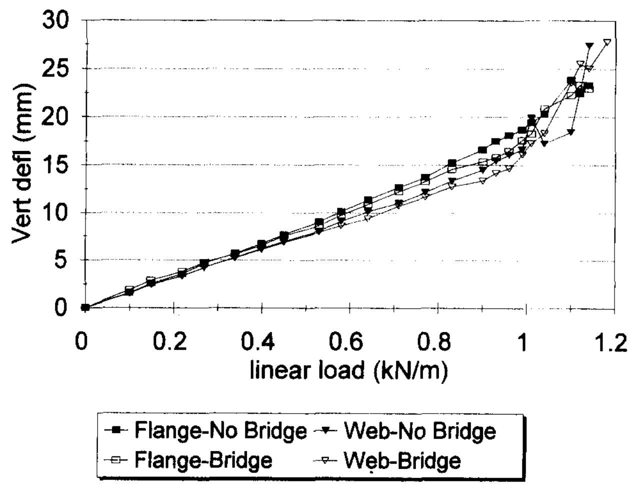


Fig. 8 Comparison of average vertical deflections

tables. This behaviour is also reflected in the theoretical buckling calculations.

Figs. 7 and 8 summarise the average midspan horizontal and vertical deflections, measured at the top flange, for the bridged and unbridged flange and cleat connected purlins.

Bridging significantly reduced the measured top flange horizontal deflections at a given load for both purlin connection methods. There was also a less pronounced increase in horizontal deflection just before failure, compared with the unbridged purlins.

Stress calculations from the computer model indicate that at failure loads near 1.18 kN/m the purlin section stresses at the internal support are in excess of the yield and tensile strength of the steel. The reduced section stiffness, due to yielding at the internal support, has contributed to the purlins' non-linear deflection behaviour seen at the higher load levels.

6.1. Unbridged tests

The unbridged purlins generally failed by a single buckle occurring at the flange - web junction near the middle of the span. In Tests 3 and 4, web buckling occurred at the internal supports about 0.2 kPa before overall purlin failure.

In all five tests, the midspan buckling failure and purlin collapse was followed by tearing in the webs at the internal support. Tearing commenced at the bolt holes in the flange connected to the beam and continued the full depth of the web to the opposite flange. Tension coupon tests found that the base metal experiences brittle failure and the drastically reduced material ductility contributed to the purlin tearing failure.

6.2. Bridged tests

With a single line of bridging purlins tended to fail with a web-flange local buckle at or near the bridging point. The bridging reduced purlin horizontal deflection by more than half. A slight reduction in vertical deflection also occurred, until just before failure. Horizontal deflections, in the top flange, at failure varied from 2.1 mm (purlin 13A-W) to 5.1 mm (purlin 12C-E) over the three tests. Reductions in horizontal and vertical deflections, with the introduction of midspan bridging, are consistent with the findings of Zetlin and Winter (1955).

6.3. Comparison with computer models

Deflections in the tests were measured at the top flange in the test position, i.e., a height 'h' from the sheeting, while the computer model shear centre deflection is calculated at $h/2$. Assuming a rigid rotation of the section, the measured horizontal deflection at the top flange is twice the shear centre deflection. By dividing the measured top flange horizontal deflections by two, the test deflections have been converted, to shear centre deflections for comparison with the computer model results.

A comparison of the test and the computer model midspan horizontal deflections are shown in Fig. 9. The computer model underestimates the unbridged purlin horizontal deflections. The scatter in the horizontal deflections measured in the tests is more pronounced than the scatter in the vertical deflection results. Variations in the purlin/sheeting connection may have contributed to the scatter in the horizontal deflection measurements.

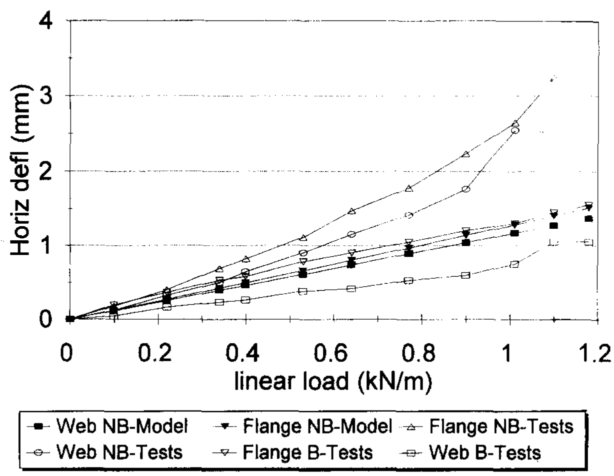


Fig. 9 Comparison of test and model horizontal deflections

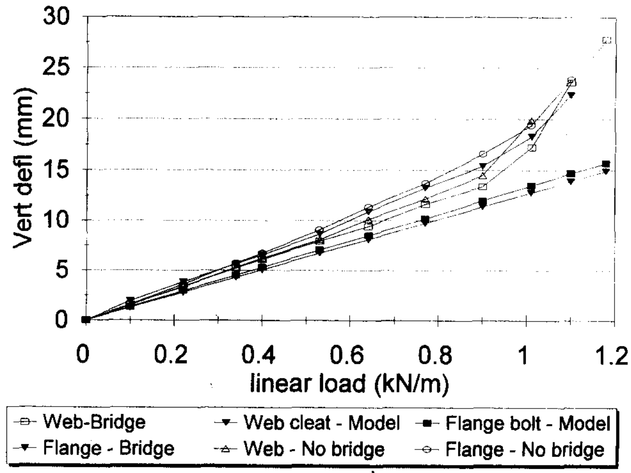


Fig. 10 Comparison of test and model vertical deflections

A 10% variation in sheeting/purlin rotational restraint (skr) causes a 10% variation in horizontal deflection, while vertical deflections only vary 1.0%. Sheeting/purlin rotational restraint is directly related to the screw fixing - its position relative to the flange centre line, and local sheeting deformation around the screw head. These factors help explain the variation in horizontal deflection test results.

A similar exercise to assess the sensitivity of horizontal and vertical deflections to variations in the purlin/beam rotational restraint (skf) value showed a 20% variation in restraint stiffness caused only a 1% variation in the horizontal deflection and negligible difference in the vertical deflection values.

A comparison of the calculated vertical deflections, with the scale model, is shown in Fig. 10. The computer models underestimated the deflections near the failure load range (1.0 to 1.18 kN/m) because they do not allow for purlin yielding which would occur at loads over 0.90 kN/m. Expected reductions in test result deflections, with the introduction of bridging, are reflected in results from computer models (refer Table 2). Computer models that allow for changes in purlin geometry before each new load increment may produce closer agreement between test and calculated deflections.

7. Conclusions

Experiments performed on model scale roof purlin systems have been shown to produce good simulations of full scale roof and purlin systems. With minor limitations produced by available sheet profiles and thicknesses, the models are capable of reproducing the behaviour for a wide range of bridging options and for variations in roof sheet restraint. Comparison of the experimental results with computed results has shown to be good.

Test results indicate that the behaviour of both cleat restrained and flange connected purlins is similar when the roof diaphragm is stiff. Both connection systems exhibited a general yield type of failure at a yield stress approaching 680 MPa. The behaviour has been shown to be remarkably linear up to failure. Ultimate failure of the cross section occurred by buckling or tearing after the onset of cross-section yield.

Model tests using the higher strength (550 MPa) material result in higher elastic stresses than in the commercial (450 MPa) grade material. These elastic stresses can cause a combination of lateral-torsional buckling or local flange/web buckling, which could not be observed in full-scale testing. In the current test series, local buckling did not occur until the cross-section had yielded.

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Notation

The following notation is used in this paper:

bmt	base metal thickness
D	depth of a purlin
h	purlin height
I_w	warping section constant
I_x	second moments of area of the full section about the x-axes
I_y	second moments of area of the full section about the y-axes
J	torsion section constant
skb	bridging/purlin connection rotational stiffness
skf	purlin/beam rotational restraint spring stiffness
skr	sheeting/purlin rotational spring stiffness