International Specialty Conference on Cold-Formed Steel Structures

(2000) - 15th International Specialty Conference on Cold-Formed Steel Structures

Oct 19th, 12:00 AM

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Fifteenth International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri U.S.A., October 19-20, 2000

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ABSTRACT

The latest edition of the American Iron and Steel Institute (AISI) *Specification for the Design of Cold-Formed Steel Structural Members* was published in 1996 (AISI, 1996). Design rules are presented in both allowable stress design (ASD) and load and resistance factor design (LRFD) formats. The LRFD rules of the latest AISI Specification form the basis of the Australian/New Zealand Standard *AS/NZS 4600:1996 Cold-Formed Steel Structures* (SA/SNZ, 1996), which was published in late 1996 and supersedes the corresponding Australian permissible stress Standard AS 1538–1988.

One of the main applications of cold-formed steel is purlins and girts in metal roof and wall systems. The design rules for these structural members have been refined over the years and procedures are now available which allow the effects of the sheeting restraint, lapped regions, and height of load application to be incorporated. Nevertheless, unlike AS/NZS 4600:1996, the AISI Specification does not explicitly allow the use of advanced numerical techniques such as rational elastic buckling analyses within Clause C3.1.2 to improve the accuracy and reliability of the design procedures.

This paper summarises the existing two approaches to purlin design (herein termed the C-factor approach and the R-factor approach) in the AISI Specification, and presents a third approach based on the use of elastic rational buckling analysis to determine the lateral buckling strength of the purlin system. The relative merits and drawbacks of each approach are discussed. The importance of distortional buckling as a failure mode to be considered (currently neglected in the AISI Specification but included in AS/NZS 4600:1996) is also highlighted. The ultimate load capacities computed using the various design models are compared with test results obtained from vacuum rig testing at the University of Sydney over a period of more than 10 years. The use of rational elastic buckling analysis in conjunction with the existing AISI beam strength curve is found to be effective as a means of assessing the lateral buckling strength of purlin systems.

1 INTRODUCTION

The limit states design of purlins attached to metal sheeting involves the consideration of many modes of failure including combined bending and shear, lateral buckling, and possibly distortional buckling. A rational consideration of the lateral buckling mode of failure should consider the inherent lateral and torsional restraint provided to the purlin system by the attached metal sheeting, the additional stiffness provided by lapped regions at the ends of the span, and the influence of the height of load application.

The AISI *Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 1996) describes two basic methods for determining lateral buckling strength. The first method is outlined in Clause C3.1.2 and involves the calculation of the elastic lateral buckling moment using classical formulae. While the effect of the moment gradient is considered through the C_b factor, the contribution of the sheeting restraint and the influence of load height are neglected. This approach is termed the C-factor approach in this paper.

The second method of determining lateral buckling strength is described in Clause C3.1.3 of the AISI Specification and accounts empirically for the sheeting restraint through the use of a reduction (R) factor which has been calibrated to test results on purlins with screw-fastened sheeting. This approach is termed the R-factor approach in this paper.

The main purpose of this paper is to present a third approach for lateral buckling strength determination which is both soundly based theoretically and incorporates the sheeting restraint when the sheeting is screw-fastened to the purlins, as well as the effect of lapped regions and load height. The approach utilises the inelastic lateral buckling strength (M_n) formulation of Clause C3.1.2 but with the elastic buckling moment (M_e) determined using a finite element elastic lateral buckling (FELB) analysis rather than classical formulae. In this way, the sheeting restraint, load height and other effects can be considered automatically and rationally. This procedure is termed the FELB approach in this paper.

The FELB approach is verified through comparisons with purlin tests conducted at the University of Sydney over a period of more than 10 years. These tests involve single, double and triple spans; zero, one and two rows of bridging; inwards and outwards load; and screw fastened and clip fastened steel sheeting (Hancock et al., 1990; 1992, 1994, 1996). While there is some conservatism for the single spans, the proposed FELB design procedure is shown to predict the double and triple span test results well. The advantage of the rational elastic buckling (FELB) approach over the R-factor approach is that the former is universally applicable.

While it is noted that the latest edition of the AISI Specification presents both allowable stress design (ASD) and load and resistance factor design (LRFD), discussion in this paper is restricted to the latter for brevity and to facilitate ready comparison with corresponding procedures in the Australian/New Zealand Standard AS/NZS 4600:1996 *Cold-Formed Steel Structures* (SA/SNZ, 1996). The latter standard is based principally on the LRFD variant of the AISI Specification.

2 LATERAL BUCKLING DESIGN CRITERIA

For conventional purlin systems which are fastened to rafters by bolting through the web to cleat plates (as was the case for the majority of the tests performed at the University of Sydney in the 1990s), the relevant LRFD design checks to the AISI Specification include:

- section capacity in combined bending and shear (Clause C3.3.2),
- member capacity governed by lateral buckling (Clause C3.1.2);
- bolts in shear at the cleat connection (Clause E3.4); and
- ply in bearing at the cleat connection (Clause E3.3).

In the United States, standard practice is to attach purlins to rafters by bolting directly through the bottom flange, thereby avoiding the need for cleats. In this case, combined bending and web crippling (Clause C3.5.2) at the supports, fastener strength in tension (Clause E3.4), and pull-out and pull-over strengths comprise the additional design checks to be conducted. It is noted that the current edition of the AISI Specification contains no provisions for a distortional buckling strength check. Such a requirement is included in AS/NZS 4600:1996, Clause 3.3.3.3 (Hancock, 1998). The relevance of distortional buckling as a failure mode for purlins is investigated in this paper.

Of all the design criteria mentioned previously, the one which is most influenced by the restraint provided by sheeting is that of lateral buckling strength. Design provisions for beam lateral buckling based on the C-factor approach are prescribed in Clause C3.1.2 of the AISI Specification. More specific rules for purlin systems with screw fastened sheeting are given in Clause C3.1.3 (R-factor approach).

C-factor approach

The basis of the C-factor approach is that for each segment of the purlin system between brace positions, a C_b factor based on the shape of the bending moment diagram is determined. This C_b factor is then used in conjunction with beam effective lengths to determine the elastic buckling moment (M_e) of the segment. The corresponding beam strength curve is shown labelled as Clause C3.1.2 in Figure 1. The advantage of the C-factor approach is that it is universally applicable to all purlin systems, including all section shapes, loading types, sheeting types and bridging layouts. It does not, however, consider the effect of load height or the lateral and torsional restraint provided by sheeting on the buckling moment. This latter aspect may be a source of considerable conservatism when it is applied to purlin systems.

As presented in Clause C3.1.2, the nominal strength (M_n) of a laterally unbraced purlin segment is given as

$$M_n = S_c \frac{M_c}{S_f} \tag{1}$$

where
S_f = the elastic section modulus of the full unreduced section for the extreme compression fibre,
S_c = the elastic section modulus of the effective section calculated at a stress M_c/S_f in the extreme compression fibre, and
M_c = the elastic critical moment calculated as follows

$$\begin{aligned} M_c &= M_y && \text{for } M_e \geq 2.78M_y \\ M_c &= \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_e}\right) && \text{for } 0.56M_y < M_e < 2.78M_y \\ M_c &= M_e && \text{for } M_e \leq 0.56M_y \end{aligned} \tag{2}$$

where M_y is the moment causing initial yield at the extreme compression fibre of the full section, and M_e is the elastic critical moment.

Clause C3.1.2 provides formulae for the determination of the elastic critical moment M_e. These formulae consider the influence of moment gradient on the buckling strength through the C_b factor,

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \quad (3)$$

where:

M_{max} = absolute value of maximum moment in the unbraced segment
M_A = absolute value of moment at quarter point of unbraced segment
M_B = absolute value of moment at centreline of unbraced segment
M_C = absolute value of moment at three-quarter point of unbraced segment.

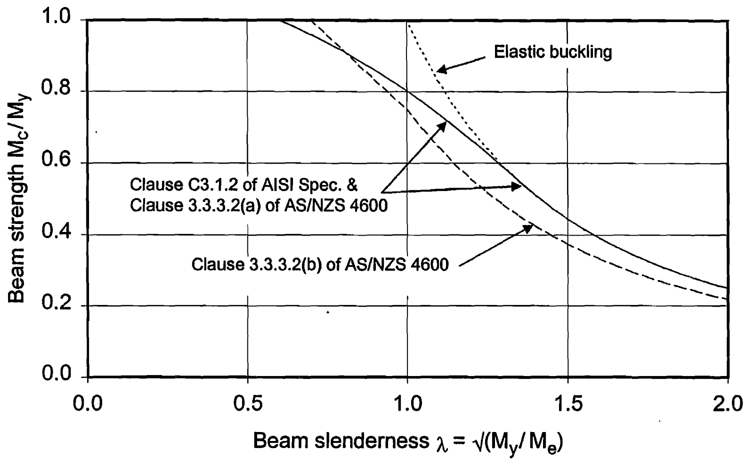

Figure 1. Beam strength curves in AISI Specification and AS/NZS 4600:1996

R-factor approach

The so-called R-factor approach (Clause C3.1.3) is an empirically based procedure whereby a single reduction (R) factor is used to account for the lateral buckling behaviour of purlin systems with screw fastened sheeting. No lateral buckling calculations are required as the R factor is applied directly to the bending section capacity (S_eF_y). It is interesting to note that while the list of qualifications applying to the R-factor approach is similar in both the AISI Specification and AS/NZS 4600:1996, the actual values of the R-factors vary between the two specifications as shown in Table 1.

In the AISI Specification, it is noted in Clause C3.1.3 that the R-factor approach applies to purlin systems with the tension flange attached to deck or sheathing and with the compression flange laterally unbraced. This is the reason why there are no AISI R-factors for downwards loading given in Table 1. Also, since the AISI R-factors are calibrated for a laterally unbraced compression flange (zero rows of bridging), these R-factors will be conservative when applied to purlin systems with intermediate bridging. In AS/NZS 4600:1996, the R-factors vary with the number of bridgings as indicated in Table 1 since the use of bridging is standard practice in Australia and sufficient full-scale tests have been performed to justify the delineation.

The advantage of the R-factor approach is that it is calibrated directly from test results and therefore generally provides higher design capacities than the C-factor approach (at least for zero rows of bridging in the case of the AISI Specification). The major disadvantage of the R-factor approach is that it apples only to specific purlin system configurations (concealed fixed sheeting is excluded for example), and consequently is quite restrictive in its use unless additional standard tests are performed. In the AISI Specification, the beneficial effect of providing one or two rows of bridging per span is not reflected in the R-factors.

Table 1. R-Factors in the AISI Specification and AS/NZS 4600:1996

Loading	Span Configuration	Bridging	AISI Spec. R-Factor	AS/NZS 4600 R-Factor
Uplift	Simple	0	0.4(C), 0.5(Z)	0.75
		1	0.4(C), 0.5(Z)	0.85
		2+	0.4(C), 0.5(Z)	1.00
	Double/double lapped	0 per span	0.6(C), 0.7(Z)	0.60
		1 per span	0.6(C), 0.7(Z)	0.70
		2+ per span	0.6(C), 0.7(Z)	0.80
	Three or more lapped spans	0 per span	0.6(C), 0.7(Z)	0.75
		1 per span	0.6(C), 0.7(Z)	0.85
		2+ per end span, 1+ per interior span	0.6(C), 0.7(Z)	0.95
Downwards	Simple	0	N.A.	N.A.
		1	N.A.	N.A.
		2+	N.A.	N.A.
	Double/double lapped	0 per span	N.A.	N.A.
		1 per span	N.A.	N.A.
		2+ per span	N.A.	N.A.
	Three or more lapped spans	0 per span	N.A.	0.85
		1 per span	N.A.	0.85
		2+ per end span, 1+ per interior span	N.A.	0.85

Lateral Buckling Strength Based on Rational Elastic Buckling Analysis (FELB Approach)

In AS/NZS 4600:1996, it is permitted to use rational elastic buckling analysis to calculate member buckling loads and moments, and cross-section buckling stresses, in lieu of simplified formulae. The use of rational buckling analysis often leads to more accurate determination of buckling quantities and a consequent reduction in design conservatism.

The use of rational buckling analysis in design is not embraced by the current edition of the AISI Specification. Specifically, there are no provisions in the AISI Specification for the use of rational elastic buckling analysis to determine the elastic buckling moment of a purlin system when applying Clause C3.1.2.

Whereas the C-factor approach requires the calculation of the C_b factor separately for each unbraced segment of the purlin system, the output from a rational elastic buckling analysis is a single quantity λ which represents the scalar value by which the distribution of in-plane design bending moments (M_u) should be multiplied to cause lateral buckling of the purlin system. In general, from span-to-span, a purlin system may incorporate different sized members, lapped regions, and may be subjected to loads of different magnitudes. Some care is therefore required to identify the segment which is critical from a strength viewpoint, despite the fact that all segments are tacitly assumed to elastically buckle simultaneously. The critical segment is the one for which the ratio of the buckling moment to the section strength (M_e/M_{no}, in which $M_{no} = S_eF_y$) is a maximum.

The application of the rational buckling analysis is relatively straightforward for systems comprising channel section purlins since the plane of loading (parallel to the web) produces bending about the major principal (symmetry) axis, and lateral buckling about the minor axis is a distinct physical possibility. In the case of Z-section purlin systems restrained by sheeting against lateral movement, and therefore constrained to bend about the non-principal axis perpendicular to the web, the section properties employed in the rational elastic buckling analysis should pertain to an *equivalent* channel where the direction of the flange of the Z-section attached to the sheeting is reversed (Ings & Trahair, 1984).

For Z-sections comprising flanges which are of equal dimensions, as is typically the case in the North America, it is obvious how the equivalent channel should be defined. However, in Australia, the Z-sections have lips perpendicular to the flange and consequently the top and bottom flanges are of slightly different length to facilitate lapping. Nevertheless, it is sufficient to average the flange dimensions of these standard Z-sections for the purpose of defining an equivalent channel section with equal flanges. However, so-called new generation Z-section purlin shapes with *significantly* different top and bottom flanges are starting to appear on the market, and in this case it is appropriate to consider a more rigorous approach to the definition of the equivalent channel section. The following approach is suggested in this paper pending further research.

In essence, if the top and bottom flanges are significantly different, *two* equivalent channel sections need to be considered in the buckling analysis. The first equivalent channel, which may be termed the *top flange equivalent channel*, comprises flanges which are identical to the top flange of the Z-section. The second equivalent channel, which may be termed the *bottom flange equivalent channel*, comprises flanges which are identical to the bottom flange of the Z-section. The section properties used in the rational buckling analysis should relate to the equivalent channel for which the flanges correspond to the free flange in the physical purlin system. The free flange is unrestrained by sheeting and is therefore able to buckle laterally and twist. For a continuous lapped purlin system, the alternating section orientation from span to span results in the corresponding alternation of the equivalent channel section properties employed in the rational buckling analysis. This clearly adds some complexity to the buckling analysis but automated procedures can be developed to overcome this (CASE, 1999).

The advantages of the FELB approach are that the load height and sheeting restraint effects can be accounted for in the buckling analysis. It is interesting to note that, although the FELB approach is permitted in AS/NZS 4600:1996, the corresponding beam strength curve (shown in Fig. 1) is generally lower than the curve used with the C-factor approach. The reason for the use of two beam strength curves in AS/NZS 4600:1996 is more historical than rationally based, with Clause 3.3.3.2(a) reproduced from the AISI (1996) Specification and Clause 3.3.3.2(b) reproduced from AS 1538–1988 (SA, 1988). In this paper, the use of rational buckling analysis is proposed with the (higher) beam strength curve defined in the AISI Specification, which is equivalent to the beam strength curve defined in Clause 3.3.3.2(a) of AS/NZS 4600:1996.

3 DISTORTIONAL BUCKLING DESIGN CRITERIA

For completeness, the distortional buckling design criteria defined in AS/NZS 4600:1996 (but which are absent in the AISI Specification) are presented in this section since load capacities based on the distortional buckling mode are presented later in this paper.

As presented in Clause 3.3.3.3 of AS/NZS 4600:1996, the nominal moment capacity (M_n) of sections subject to distortional buckling is generally calculated from

$$M_b = M_c \tag{4}$$

where

$$\begin{aligned} M_c &= M_y && \text{for } \lambda_d \geq 0.674 \\ M_c &= \frac{M_y}{\lambda_d}\left(1 - \frac{0.22}{\lambda_d}\right) && \text{for } \lambda_d > 0.674 \end{aligned} \tag{5}$$

and λ_d is the non-dimensional slenderness

$$\lambda_d = \sqrt{\frac{f_y}{f_{od}}} \tag{6}$$

where f_{od} is the elastic distortional buckling stress.

4 SUMMARY OF TEST DATA ON PURLIN–SHEETING SYSTEMS

In 1988, a large vacuum test rig was commissioned in the Centre for Advanced Structural Engineering at the University of Sydney using funds provided by the Metal Building Products Manufacturers Association (MBPMA) for the purpose of providing test data on metal roofing systems. The test rig uses a conventional vacuum box to simulate wind uplift or inwards load. While the early series of tests were "generic" by virtue of their funding through the MBPMA, later test programs have been performed specifically for individual companies who have nevertheless made their results available in the public domain. The test programs which have been conducted are summarised in Table 2. More detailed information on specific tests is listed in Table 3 presented later.

Table 2. Purlin–Sheeting Test Programs Performed at the University of Sydney

Series	Loading	Spans*	Bridging†	Sheeting Type	Rafter Fixing
S1	Outwards	3-span lapped	0, 1, 2	Screw fastened	Cleats
S2	Outwards	2-span lapped	0, 1, 2	Screw fastened	Cleats
S3	Outwards	Simply supported	0, 1, 2	Screw fastened	Cleats
S4	Inwards	3-span lapped	0, 1	Screw fastened	Cleats
S5	Outwards	Simply supported	0, 1, 2	Concealed fixed	Cleats
S6	Outwards	3-span lapped	1	Concealed fixed	Cleats
S7	Outwards	Simply supported	0, 1, 2	Screw fastened	Cleats
S8	Outwards	Simply supported 3-span lapped	1, 2	Screw fastened	Cleats
CP	Outwards	3-span lapped	0, 1	Screw fastened	Flange

* 3 × 7.0 m spans with 900 mm laps between bolt centres for 3-span lapped configuration.
2 × 10.5 m spans with 1500 mm laps between bolt centres for 2-span lapped configuration.
1 × 7.0 m span for simply supported configuration.

† 0: Zero rows of bridging in each span
1: One row of bridging in each span
2: Single and double spans: Two rows of bridging in each span
Triple spans: Two rows of bridging in the end spans, one row in the central span.

4 COMPARISON OF LATERAL BUCKLING APPROACHES

All cleat fastened purlin tests performed at the University of Sydney have been analysed using the design approaches outlined in Sections 2 and 3. The results of the studies are summarised in Table 3, in which the symbol q refers to the uniformly distributed load capacity of the purlin system. The measured yield stress (f_y) and cross-section dimensions, together with resistance (capacity factors) (ϕ) of unity, were used in all comparisons. The symbols used in Table 3 to define sheeting and bridging configurations are defined in Table 4. The columns (9) to (11) of Table 3 headed "C-Factor" "R-Factor" and 'FELB" refer to the direct application of the methods as described in Section 3. The ratios of test load (q_T) to predicted load are provided for all methods in columns (12) to (14) of Table 3. The following definitions apply:

$$\begin{aligned}\hat{q}_C &= \min(q_D, q_{MV}, q_C)\\ \hat{q}_R &= \min(q_D, q_{MV}, q_R)\\ \hat{q}_F &= \min(q_D, q_{MV}, q_F)\end{aligned} \qquad (7)$$

In the finite element lateral buckling analyses (FELB approach), the laps over internal supports and the load height were modelled, and a minor axis rotational restraint (k_{ry}) of 1000 kN (representing the elastic restraint provided by the sheeting to the purlins) was employed. The final three columns of Table 3 are reproduced graphically in Figs. 2 to 4.

As foreshadowed in Section 2, it can be seen in Table 3 and Figs 2 to 4 that the strength predictions based on the C-factor approach are generally the most inferior and have the highest variability. With the exception of the zero rows of bridging cases, the FELB results are superior to the AISI R-factor approach. The R-factor approach provides the best predictions of the test results with zero rows of bridging since it is for these cases it has been calibrated. In view of its universal applicability and generally superior results, the FELB approach may be viewed as the most practically useful design methodology, since it is applicable to all purlin system configurations and incorporates the beneficial effects of sheeting restraint on the elastic buckling moment.

It can be observed in Table 3 that the predictions of the C-factor and FELB approaches improve as the number of rows of bridging increases. This conclusion is also clearly apparent in Figure 3 which has been plotted specifically for the FELB results in conjunction with the AISI beam curve. This trend is a reflection of the fact that as the purlin segments between braces become less slender, the mode of failure shifts from lateral buckling to distortional buckling or even combined bending and shear. However, the conservatism of the strength predictions when failure is clearly premised on lateral buckling is due to the fact that there is apparently more lateral and torsional restraint being provided by the sheeting to the purlins than is included in the models (no torsional restraint from sheeting is included in the FELB predictions, for example).

It can be seen in Table 3 that the distortional buckling failure mode undercuts the combined bending and shear failure mode in the majority of cases. While on the one hand this might point towards a need to include distortional buckling criteria in future editions of the AISI Specification, it should be noted that the latter failure mode is seldom more than 15 per cent below the combined bending and shear failure mode. Evidently, the rules in Clause B4.2 of the AISI Specification for uniformly compressed elements with edge stiffeners partially account for distortional buckling if the latter is considered to be quasi-local buckling.

Table 3a. Purlin Test Results and Comparison with Design Models — Single and Double Span Tests

Test (1)	Section (2)	Bridging (3)	Sheeting (4)	Test f_y (MPa) (5)	Test q_T (kN/m) (6)	Distort. q_D (kN/m) (7)	Bend/Shear q_{MV} (kN/m) (8)	C-Factor q_C (kN/m) (9)	R-Factor q_R (kN/m) (10)	FELB q_F (kN/m) (11)	C-Factor $q_T/\hat{q}_C$ (12)	R-Factor $q_T/\hat{q}_R$ (13)	FELB $q_T/\hat{q}_F$ (14)
S3T1R	Z-20024	0	TD	529	3.28	3.72	4.25	0.63	2.01	1.31	5.21	1.63	2.50
S3T4	C-20024	0	MC	518	3.63	3.82	4.23	0.64	1.61	1.16	5.67	2.26	3.13
S5L1	Z-20025	0	KL	525	2.57	3.89	4.35	0.62	2.06	1.29	4.15	1.25	1.99
S5S1	Z-20019	0	SD	517	2.17	2.54	3.05	0.43	1.45	0.88	5.05	1.50	2.47
S7T1	Z-20015	0	SH	527	1.85	1.77	2.06	0.32	0.97	0.64	5.78	1.90	2.89
S7T2	C-20015	0	SH	548	1.70	1.72	1.99	0.30	0.94	0.54	5.67	1.81	3.15
										Mean	5.25	1.72	2.69
										SD	0.62	0.35	0.45
S3T2	Z-20024	1	TD	529	3.69	3.72	4.25	2.41	2.01	2.90	1.53	1.83	1.27
S3T5	C-20024	1	MC	518	3.63	3.82	4.23	2.44	1.60	2.84	1.49	2.26	1.28
S5L2	Z-20025	1	KL	525	4.19	3.89	4.35	2.42	2.06	2.98	1.73	2.04	1.41
S5S2	Z-20019	1	SD	517	2.28	2.54	3.05	1.61	1.45	2.04	1.42	1.58	1.12
S7T3	C-20015	1	SH	512	1.77	1.65	1.91	1.10	0.72	1.28	1.61	2.44	1.38
S8T2	C-20015	1	TD	480	1.71	1.59	1.84	1.10	0.70	1.28	1.55	2.46	1.34
S8T3	C-15012	1	TD	582	0.83	0.90	1.05	0.42	0.40	0.49	1.98	2.08	1.69
										Mean	1.62	2.10	1.36
										SD	0.19	0.32	0.18
S3T3	Z-20024	2	TD	529	4.76	3.72	4.25	3.58	2.01	3.85	1.33	2.37	1.28
S3T6	C-20024	2	MC	518	4.71	3.82	4.23	3.52	1.60	3.67	1.34	2.94	1.28
S5L3	Z-20025	2	KL	525	4.90	3.89	4.35	3.54	2.06	3.77	1.38	2.38	1.30
S5S3	Z-20019	2	SD	517	2.74	2.54	3.05	2.54	1.45	2.74	1.08	1.90	1.08
S7T5	C-20015	2	SH	510	1.95	1.65	1.91	1.65	0.72	1.72	1.18	2.69	1.18
S8T1	C-20015	2	TD	500	1.98	1.63	1.88	1.64	0.72	1.70	1.21	2.77	1.21
S8T4	C-15012	2	TD	578	0.93	0.90	1.05	0.78	0.40	0.84	1.19	2.35	1.11
										Mean	1.25	2.48	1.21
										SD	0.11	0.35	0.09
S2T1	Z-30025	0-0	MC	485	4.33	4.01	4.07	1.23	3.02	2.71	3.52	1.43	1.60
S2T2	Z-30025	1-1	MC	485	4.93	4.01	4.07	3.20	3.02	3.93	1.54	1.63	1.25
S2T3	Z-30025	2-2	TD	485	5.77	4.01	4.07	4.55	3.02	4.42	1.44	1.91	1.44
										Mean	2.17	1.66	1.43
										SD	1.17	0.24	0.17

Table 3b. Purlin Test Results and Comparison with Design Models — Triple Span Tests

Test (1)	Section (2)	Bridging (3)	Sheeting (4)	Test f_y (MPa) (5)	Test q_T (kN/m) (6)	Distort. q_D (kN/m) (7)	Bend/Shear q_{MV} (kN/m) (8)	C-Factor q_C (kN/m) (9)	R-Factor q_R (kN/m) (10)	FELB q_F (kN/m) (11)	C-Factor $q_T/\hat{q}_C$ (12)	R-Factor $q_T/\hat{q}_R$ (13)	FELB $q_T/\hat{q}_F$ (14)
S1T1	Z-15019	0-0-0	MC	487	2.31	2.66	2.91	0.48	1.94	1.30	4.81	1.19	1.78
S1T4A	Z-20015	0-0-0	TD	520	2.58	2.94	2.85	0.65	2.16	1.57	3.97	1.20	1.64
S1T7	Z-20019	0-0-0	TD	495	3.51	3.96	4.36	0.86	3.07	2.14	4.08	1.14	1.64
										Mean	4.29	1.18	1.69
										SD	0.46	0.03	0.08
S1T2	Z-15019	1-1-1B	MC	487	2.63	2.66	2.91	1.67	1.94	2.14	1.57	1.35	1.23
S1T5	Z-20015	1-1-1A	TD	520	2.94	2.94	2.85	2.36	2.16	2.64	1.25	1.36	1.11
S1T8	Z-20019	1-1-1A	TD	495	4.28	3.96	4.36	3.05	3.07	3.56	1.40	1.39	1.20
S6L1	Z-15019	1-1-1A	KL	615	2.56	3.11	3.52	1.84	2.37	2.29	1.39	1.08	1.12
S6L2	Z-20019	1-1-1A	KL	517	3.81	4.11	4.57	3.60	3.27	4.07	1.06	1.17	0.94
S6S1	Z-20015	1-1-1A	SD	529	2.64	2.65	2.74	2.40	2.07	2.64	1.10	1.28	1.00
S6S2	Z-15019	1-1-1A	SD	527	2.71	2.86	3.14	1.85	2.11	2.25	1.46	1.29	1.20
										Mean	1.32	1.27	1.11
										SD	0.19	0.11	0.11
S1T3	Z-15019	2-1-2B	TD	487	2.98	2.72	2.84	2.11	1.90	2.47	1.41	1.57	1.21
S1T9	Z-20019	2-1-2B	TD	495	4.55	3.96	4.36	3.77	2.92	4.20	1.21	1.56	1.15
S8T5	Z-20015	2-1-2A	TD	529	2.93	2.65	2.74	2.65	1.95	2.94	1.11	1.50	1.11
S8T6	Z-15019	2-1-2B	SR	546	3.37	2.85	3.15	2.25	2.10	2.74	1.50	1.60	1.23
										Mean	1.31	1.56	1.17
										SD	0.18	0.04	0.06
S4T3	Z-20015	0-0-0	TD/MC	480	2.90	2.98	2.90	3.28	2.20	2.97	1.00	1.32	1.00
S4T4	Z-20015	0-0-0	MC	480	2.94	2.98	2.90	3.28	2.20	2.97	1.01	1.34	1.01
S4T5	Z-15019	0-0-0	MC	480	2.92	2.66	2.88	2.84	1.93	2.40	1.10	1.52	1.22
										Mean	1.04	1.39	1.08
										SD	0.05	0.11	0.12
S4T1	Z-20019	1-1-1A	TD	480	3.97	4.22	4.49	4.22	3.20	4.83	0.94	1.24	0.94
S4T2	Z-20019	1-1-1A	MC	480	4.42	4.22	4.49	4.22	3.20	4.83	1.05	1.38	1.05
S4T6	Z-20019	1-1-1A	MC	480	2.69	2.66	2.88	2.66	1.93	2.91	1.01	1.40	1.01
										Mean	1.00	1.34	1.00
										SD	0.05	0.09	0.05

Table 4. Explanation of Sheeting and Bridging Symbols used in Table 3

Sheeting Symbol in Table 2	Sheeting Name	Bridging Symbol in Table 2	Bridging Locations
KL	Klip-Lok	1-1-1A	2800 mm from end supports
MC	Monoclad	1-1-1B	2890 mm from end supports
TD	Trimdeck	2-1-2A	2370 mm and 4195 mm from end supports
SD	Speed Deck	2-1-2B	2410 mm and 4250 mm from end supports
SH	Spandek Hi-Ten	Note: All bridging is located centrally in the centre span	
SR	Spanrib		

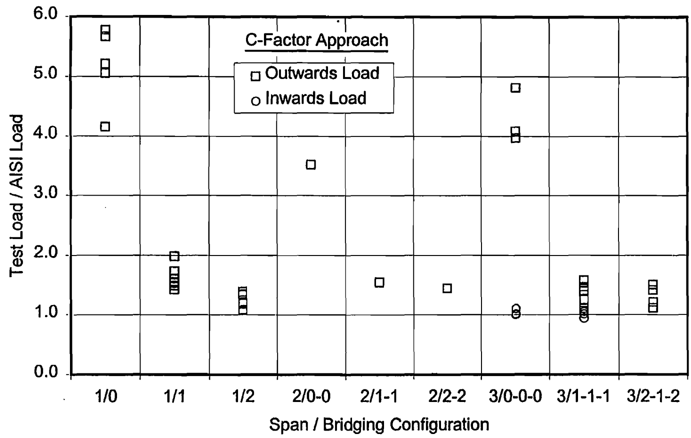

Figure 2. Comparison of C-factor design model with test data

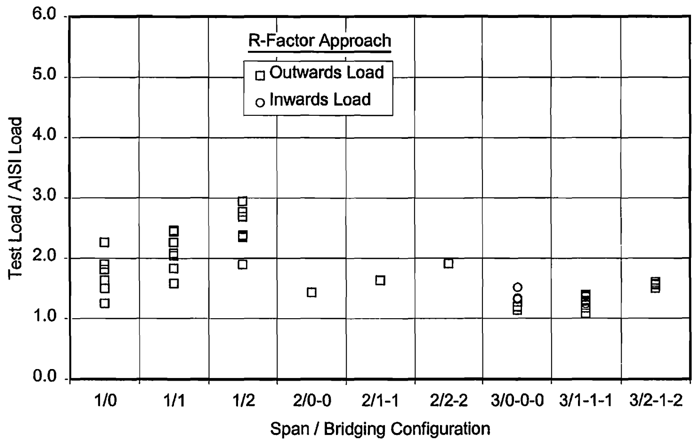

Figure 3. Comparison of R-factor design model with test data

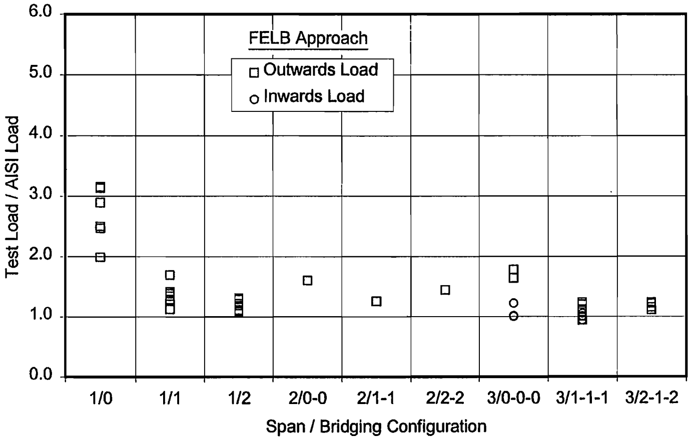

Figure 4. Comparison of FELB design model with test data

CONCLUSIONS

This paper has outlined three current approaches to the design of purlin systems which are referred to herein as the C-factor, the R-factor and the FELB approaches. While the C-factor and R-factor approaches are already available in the AISI Specification, the FELB approach is proposed as an alternative, universally applicable design procedure for purlin systems which gives improved capacities compared to the C-factor approach.

In the proposed FELB approach, the existing AISI beam strength curve is used but a finite element lateral buckling analysis is used to calculate the elastic critical moment (M_e) in lieu of classical formulae which incorporate approximate adjustments for moment gradient. It is only through such numerical means that the effect of sheeting restraint, load height and the interactions between adjacent segments of the purlin system can be properly accounted for in the elastic buckling moment calculation.

The rationality and effectiveness of the FELB approach has been confirmed by comparing the predicted strengths with the results of all cleated purlin tests performed at the University of Sydney since the late 1980s.

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