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Ultimate Shear Strength of LiteSteel Beams

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***Abstract* — This paper presents the details of an investigation on the shear behaviour of a recently developed, cold-formed steel beam known as LiteSteel Beam (LSB). The LSB section has a unique shape of a channel beam with two rectangular hollow flanges and is produced by a patented manufacturing process involving simultaneous cold-forming and dual electric resistance welding. In the present investigation, a series of numerical analyses based on three-dimensional finite element modeling and an experimental study were carried out to investigate the shear behaviour of 10 different LSB sections. It was found that the current design rules in cold-formed steel structures design codes are very conservative for the shear design of LiteSteel beams. Significant improvements to web shear buckling occurred due to the presence of rectangular hollow flanges while considerable post-buckling strength was also observed. Therefore the design rules were further modified to include the available post-buckling strength. Suitable design rules were also developed under the direct strength method format. This paper presents the details of this investigation and the results including the final design rules for the shear capacity of LSBs. It also presents new shear strength formulae for lipped channel beams based on the current design equations for shear strength given in AISI (2007) using the same approach used for LSBs.**

***key words** – Ultimate shear strength, Post-buckling, LiteSteel Beams, Direct strength method, Cold-formed steel*

I. INTRODUCTION

Advanced roll-forming technologies and very thin and high strength steel have significantly increased the use of thin-walled, cold-formed steel products in the building industry. Australian Tube Mills has recently developed a new hollow flange channel cold-formed section, known as the LiteSteel Beam (LSB) shown in Figure 1. The innovative LSB sections have the beneficial characteristics of torsionally rigid closed rectangular flanges combined with economical fabrication processes from a single strip of high strength steel. They combine the stability of hot-rolled steel sections with the high strength to weight ratio of conventional cold-formed steel sections. The integral benefits of lightweight, strength, and ease of constructability offer a new choice of using LSBs for structural engineers. The LiteSteel beam has a wide range of applications in residential, commercial and industrial buildings (Figure 1), and is on average 40% lighter than traditional hot-rolled structural sections of equivalent bending strength [1]. In the building systems, LSB sections are commonly used as flexural members, for example, floor joists and bearers. For LSBs to be used as flexural members, their flexural and shear capacities must be known.

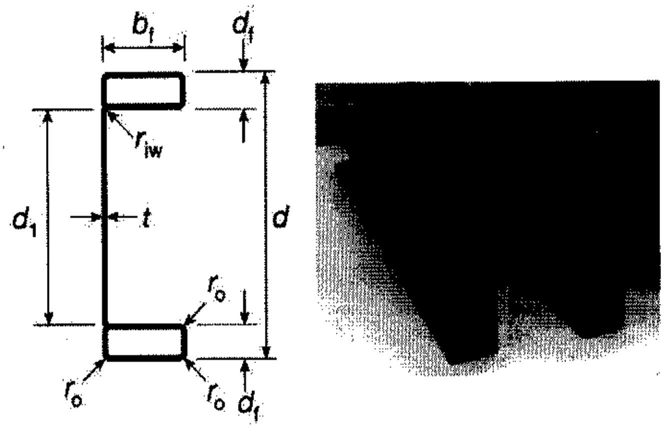


Figure 1: LiteSteel Beams

Flexural behaviour of LSBs has been investigated recently by Mahaarachchi and Mahendran [2], and hence the moment capacities of LSBs are available. However, the shear behaviour of LSBs has not been investigated yet. Past research [3,4] was restricted to plate girders. Therefore experimental and numerical studies were undertaken to investigate the shear buckling and strength behaviour of LSB sections, and to develop improved shear design rules that take into account the effects of additional fixity along the web to flange juncture of LSBs and post-buckling strength. This paper describes the details of this investigation on the ultimate shear strength of LSBs and presents the results including the new shear design equations for LSBs. It also includes similarly improved shear design equations for lipped channel beams.

II. SHEAR TESTS

Shear behaviour of LSBs was investigated using a series of shear tests of simply supported LiteSteel beams subjected to a mid-span load (see Figure 2). In order to simulate a primarily shear condition, relatively short test beams of span based on aspect ratio (shear span a/ clear web height d_1) of 1.0 and 1.5 were selected. Two LSB sections were bolted back to back using three T-shaped stiffeners located at the end supports and the loading point in order to eliminate any torsional loading of test beams. Test specimens were chosen such that all three types of shear failure (shear yielding, inelastic and shear buckling) occurred in the tests. A 20 mm gap (see Figure 2) was included between the LSB sections to allow the test beams to behave independently while remaining together to resist torsional effects. The stiffeners were used to avoid eccentric loading and web crippling. Figure 2 shows the shear test set-up used here. Table 1 presents the experimental results while Figure 3 shows the typical shear failure modes LSBs.

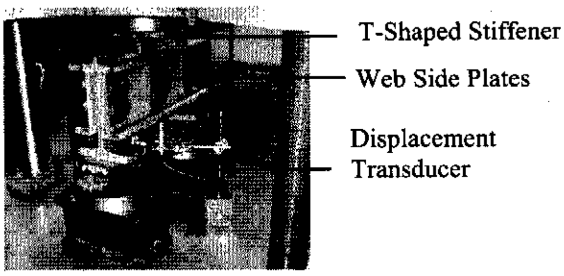
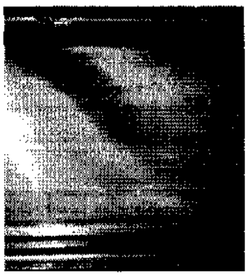
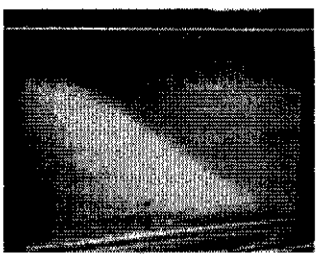


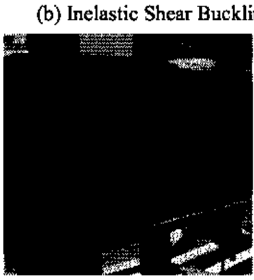
Figure 2: Shear Test Set-up



(a) Shear Yielding Failure



(b) Inelastic Shear Buckling

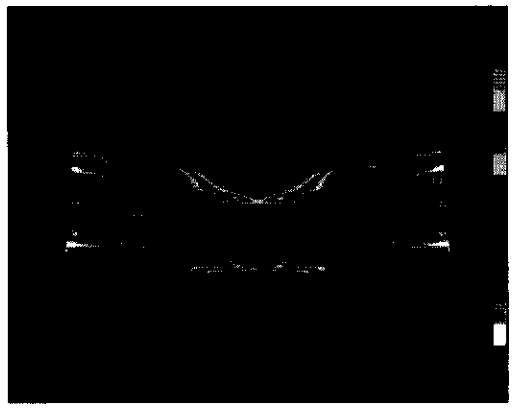


(c) Elastic Shear Buckling

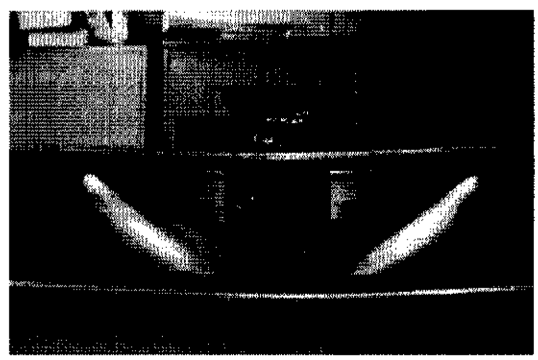
Figure 3: Shear Failure Modes of LSBs

III. NUMERICAL MODEL

This section describes the development of finite element models to investigate the ultimate shear strength behaviour of LSBs (Figures 4 (a)). For this purpose, a general purpose finite element program ABAQUS Version 6.7 [5], which has the capability of undertaking nonlinear geometric and material analyses of three dimensional structures, was used. Finite element models of tested LSBs were developed with the objective of simulating the actual test members' physical geometry, loads, constraints, mechanical properties, residual stresses and initial geometric imperfections as closely as possible. The shell element S4R5 in ABAQUS was used to model the shear behaviour of LSBs. R3D4 rigid body elements were used to simulate the restraints and loading in the finite element models. The elastic modulus and Poisson's ratio were taken as 200 GPa and 0.3, respectively. Simply supported boundary conditions were implemented under a three-point loading arrangement. Figures 4 (a) and (b) show the shear yielding of 125x45x2.0 LSB (Aspect Ratio = 1.5) from finite element analyses (FEA) and shear tests.



(a) FEA



(b) Shear Test

Figure 4: Shear Yielding of 125x45x2.0 LSB (Aspect Ratio = 1.5)

IV. VALIDATION OF FINITE ELEMENT MODEL

It is important to validate the developed finite element model for non-linear analyses of LSBs. This was achieved by comparing the non-linear finite element analysis results with the results obtained from shear tests. Shear yielding and failure modes from finite element analyses (FEA) agreed well with experimental failure modes as shown in Figures 4(a) and (b). Table 1 presents a summary of the ultimate shear capacity results of the non-linear static analyses using the finite element model developed in this research and a comparison of these results with corresponding experimental results. The mean and COV of the ratio of ultimate shear capacities from experiments and FEA are 0.99 and 0.028 as shown in Table 1. This indicates that the finite element model predicts the ultimate shear behaviour and capacities of LSBs with very good accuracy. Figure 5 shows the FEA results in the form of load versus mid-span vertical deflection for 125x45x2.0 LSB (Aspect Ratio =1.5) and compares them with experimental results. It shows that the finite element model predicts the failure load and mid-span deflection of LSBs with good accuracy.

TABLE 1 COMPARISON OF ULTIMATE SHEAR CAPACITIES FROM FEA AND TESTS

No.	LSB Sections	Aspect Ratio	Ultimate Shear Capacities (kN)		Test/FEA Ult. Shear Capacity
			Test	FEA	
1	150x45x2.0	1.0	68.5	70.0	0.98
2	200x45x1.6	1.0	63.6	63.5	1.00
3	200x60x2.0	1.0	88.2	88.5	1.00
4	200x60x2.5	1.0	119.3	118.0	1.01
5	250x60x2.0	1.0	90.1	93.0	0.97
6	250x75x2.5	1.0	139.6	136.5	1.02
7	300x60x2.0	1.0	93.0	96.0	0.97
8	300x75x2.5	1.0	143.7	151.5	0.95
9	125x45x2.0	1.5	56.9	56.0	1.02
10	150x45x1.6	1.5	45.8	47.8	0.96
11	150x45x1.6	1.5	47.1	47.8	0.99
12	150x45x1.6	1.5	47.0	47.8	0.98
13	150x45x2.0	1.5	61.1	61.0	1.00
14	150x45x2.0	1.5	58.8	61.0	0.96
15	150x45x2.0	1.5	59.5	61.0	0.98
16	200x60x1.6	1.5	56.8	55.0	1.03
17	200x45x1.6	1.5	54.2	55.0	0.99
18	200x60x2.0	1.5	74.0	76.0	0.97
19	200x60x2.5	1.5	110.0	109.0	1.01
20	250x60x2.0	1.5	>75.0	83.0	NA
21	250x75x2.5	1.5	118.9	121.0	0.98
22	300x60x2.0	1.5	> 75.0	82.0	NA
23	300x75x2.5	1.5	125.1	131.0	0.95
24	200x60x2.0	1.6	79.4	75.0	1.06
25	200x60x2.5	1.6	107.9	106.0	1.02
					Mean = 0.99
					COV = 0.028

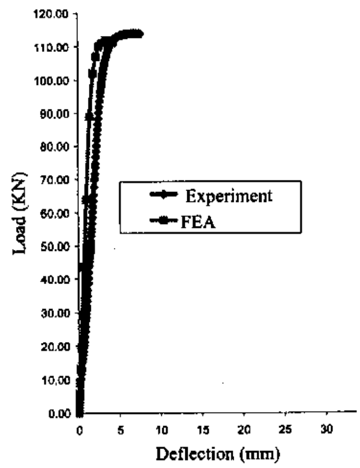


Figure 5: Load versus Vertical Deflection at Mid-span for 125x45x2.0 LSB (Aspect Ratio = 1.5)

V. PROPOSED DESIGN EQUATIONS FOR THE SHEAR STRENGTH OF LITESTEEL BEAMS

New shear strength formulae (τ_v) were developed for LSBs based on their experimental and FEA results and the current design equations for shear strength given in AISI [6]. Equations 1 to 3 present the new proposed design equations. The increased shear buckling coefficient given by Equation 4 (k_{LSB}) is included to allow for the additional fixity in the web-flange juncture [7]. It is to be noted that Equations 2 and 3 were not modified to include the reserve post-buckling strength.

$$\tau_v = \tau_{yw} = 0.6 f_{yw} \qquad \frac{d_1}{t_w} \le \sqrt{\frac{Ek_{LSB}}{f_{yw}}} \tag{1}$$

$$\tau_v = 0.6 \frac{\sqrt{Ek_{LSB} f_{yw}}}{\left[\frac{d_1}{t_w}\right]} \qquad \sqrt{\frac{Ek_{LSB}}{f_{yw}}} < \frac{d_1}{t_w} \le 1.508\sqrt{\frac{Ek_{LSB}}{f_{yw}}} \tag{2}$$

$$\tau_v = \frac{0.905 Ek_{LSB}}{\left(\frac{d_1}{t_w}\right)^2} \qquad \frac{d_1}{t_w} > 1.508\sqrt{\frac{Ek_{LSB}}{f_{yw}}} \tag{3}$$

For LSBs $k_{LSB} = k_{ss} + 0.87(k_{sf} - k_{ss})$ for $a/d_1 \ge 1$ (4)

where d_1, t_w = clear height and thickness of web; τ_{yw} = shear yield strength and f_{yw} = yield strength, k_{ss} and k_{sf} are shear buckling coefficients of plates with simple-simple and simple-fixed boundary conditions.

Equations 5 to 7 are now proposed in which post-buckling strength is included. Here post-buckling is included in the inelastic and elastic buckling regions to replace Equations 2 and 3. New design Equations for shear strength (Eqs. 6 and 7) are based on Lee et al. [8], who used a similar approach for plate girders. The nominal shear capacities (V_v) can be calculated by multiplying the shear strengths (τ_v) from Equations 5 to 7 by the area of web element ($d_1 t_w$).

$$\tau_v = \tau_{yw} \quad \text{for} \quad \frac{d_1}{t_w} \le \sqrt{\frac{Ek_{LSB}}{f_{yw}}} \tag{5}$$

$$\tau_v = \tau_i + 0.2(\tau_{yw} - \tau_i) \quad \text{for} \quad \sqrt{\frac{Ek_{LSB}}{f_{yw}}} < \frac{d_1}{t_w} \le 1.508\sqrt{\frac{Ek_{LSB}}{f_{yw}}} \tag{6}$$

$$\tau_v = \tau_e + 0.2(\tau_{yw} - \tau_e) \quad \text{for} \quad \frac{d_1}{t_w} > 1.508\sqrt{\frac{Ek_{LSB}}{f_{yw}}} \tag{7}$$

$$\text{where} \quad \tau_{yw} = 0.6 f_{yw} \tag{8}$$

$$\tau_i = 0.6\frac{\sqrt{Ek_{LSB} f_{yw}}}{\left[\frac{d_1}{t_w}\right]} \tag{9}$$

$$\tau_e = \frac{0.905Ek_{LSB}}{\left[\frac{d_1}{t_w}\right]^2} \tag{10}$$

VI. DIRECT STRENGTH METHOD

The new direct strength method (DSM) provides simple design procedures for cold-formed steel members. Proposed design equations (Eqs. 1 to 3 and 5 to 7) are therefore recast in the DSM format and are given as Equations 11 to 13 and 16 to 18 [9]. The ultimate shear stress (τ_u) was calculated as the ultimate shear capacity from tests or FEA divided the by web area of d_1t_w whereas the slenderness (λ) was calculated using Equation 15. Equations 11 to 13 present the proposed direct strength method (DSM) design equations in which post-buckling strength is not included. Experimental and FEA results are compared with non-dimensional shear strength curve based on the proposed DSM equations in Figure 6. It is to be noted that in the non-dimensional shear strength curve with slenderness (λ) as the horizontal axis all the results can be plotted together.

$$\frac{\tau_v}{\tau_{yw}} = 1 \quad \text{for } \lambda \le 0.815 \tag{11}$$

$$\frac{\tau_v}{\tau_{yw}} = \frac{0.815}{\lambda} \quad \text{for } 0.815 < \lambda \le 1.23 \tag{12}$$

$$\frac{\tau_v}{\tau_{yw}} = \frac{1}{\lambda^2} \quad \text{for } \lambda < 1.23 \quad (5) \tag{13}$$

where k_{LSB} and τ_{yw} = as defined in Eqs.4 and 8, respectively

$$\tau_{cr} = \frac{k_{LSB}\pi^2 E}{12(1-\nu^2)}\left(\frac{t_w}{d_1}\right)^2 \tag{14}$$

$$\lambda = \sqrt{\left(\frac{\tau_{yw}}{\tau_{cr}}\right)} = 0.815\left(\frac{d_1}{t_w}\right)\sqrt{\left(\frac{f_{yw}}{Ek_{LSB}}\right)} \tag{15}$$

Equations 16 to 18 present the proposed DSM design equations in which post-buckling strength is included.

$$\frac{\tau_v}{\tau_{yw}} = 1 \quad \text{for } \lambda \le 0.815 \tag{16}$$

$$\frac{\tau_v}{\tau_{yw}} = \frac{0.815}{\lambda} + 0.2\left(1 - \frac{0.815}{\lambda}\right) \quad \text{for } 0.815 < \lambda \le 1.23 \tag{17}$$

$$\frac{\tau_v}{\tau_{yw}} = \frac{1}{\lambda^2} + 0.2\left(1 - \frac{1}{\lambda^2}\right) \quad \text{for } \lambda > 1.23 \tag{18}$$

VII. COMPARISON WITH PROPOSED SHEAR STRENGTH EQUATIONS

In this section, the shear strengths from FEA and shear tests are compared with the predictions of proposed shear strength equations. Both FEA and test results are plotted in Figure 6 and compared with the new DSM based shear design equations. Figure 6 shows that there is considerable amount of post-buckling strength for LSBs subjected to shear, particularly in the case of large clear web height to thickness ratios (d_1/t_w). The proposed DSM based design equations including the post-buckling strength (Eqs.17 and 18) are able to predict the shear strengths of LSBs accurately as seen in Figure 6.

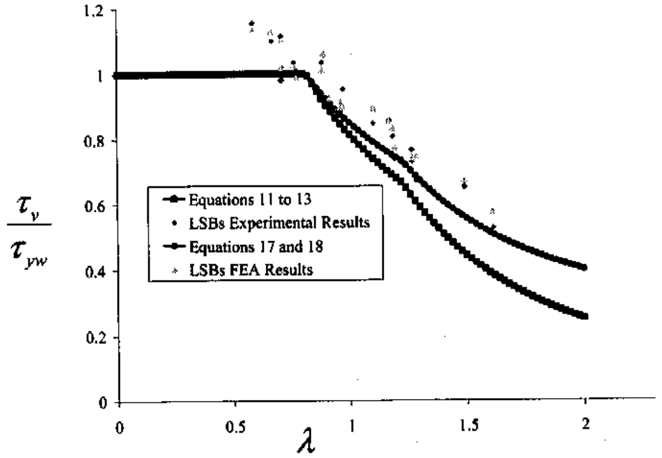


Figure 6: Comparison of Shear Strengths of LSBs with DSM based Design Equations

VIII. PROPOSED DESIGN FORMULAE FOR LONG SPAN LITESTEEL BEAMS (INFINITY ASPECT RATIO)

Long span LSBs are used in practical applications and do not have transverse stiffeners. In order to simulate this practical application, LSBs with an infinity aspect ratio were considered. Figure 7 shows the new design curves based on the proposed equations with a ϕ_v factor of 0.95 (Eqs. 1 to 3 and Eqs. 5 to 7) for the aspect ratio of infinity ($k_{LSB} = 8.5$), and compares them with the AS/NZS 4600 [10] design equations with a ϕ_v factor of 0.90. Figure 7 shows that the shear capacities predicted by the current design rules in AS/NZS 4600 are conservative because AS/NZS 4600 (SA, 2005) assumes that the web panel is simply supported at the juncture between the flange and web elements and uses a shear buckling coefficient of 5.34. However in this study it was found that the realistic support condition at the web-flange juncture of LSB is closer to a fixed support condition. Therefore the assumption considered by Clause 3.3.4 of AS/NZS 4600 may result in an overly conservative shear design for LSBs.

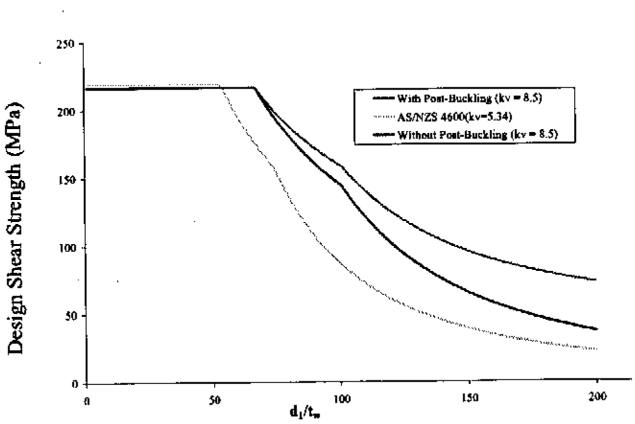


Figure 7: Design Shear Strength of LSB versus Clear Web Height to Thickness Ratio (d_1/t_w) for Long Span LSBs (f_{yw}= 380 MPa)

IX. PROPOSED DESIGN EQUATIONS FOR COLD-FORMED STEEL MEMBERS

Proposed design Equations (Eqs. 1 to 3 and Eqs. 5 to 7; 11 to 13 and 16 to 18) can be used for conventional cold-formed steel members with suitable shear buckling coefficients. An appropriate shear buckling coefficient should be included to allow for the additional fixity in the web-flange juncture of cold-formed steel members.

This research has shown that realistic support condition of LSBs at the web-flange juncture is equivalent to 87% fixed condition whereas the realistic support condition of cold-formed lipped channel beams (LCBs) at the web-flange juncture is equivalent to 40% fixed condition. The shear buckling coefficients of LSBs and LCBs can be calculated by using Equations 19 and 20, respectively. For the sake of completeness, k_{LSB} equation is presented again.

$$k_{LSB} = k_{ss} + 0.87(k_{sf} - k_{ss}) \qquad (19)$$

$$k_{LCB} = k_{ss} + 0.40(k_{sf} - k_{ss}) \qquad (20)$$

$$k_{ss} = 5.34 + 4/(a/d1)^2 \qquad a/d_1 \geq 1 \qquad (21)$$

$$k_{sf} = 8.98 + 5.61/(a/d_1)^2 - 1.99/(a/d_1)^3 \qquad a/d_1 \geq 1 \qquad (22)$$

Figure 8 shows the non-dimensional shear strength curve for cold-formed steel members. Here the ultimate shear capacities of LSBs from shear tests and FEA, and the ultimate shear capacities from experiments [11] are shown in Figure 8 in the DSM format.

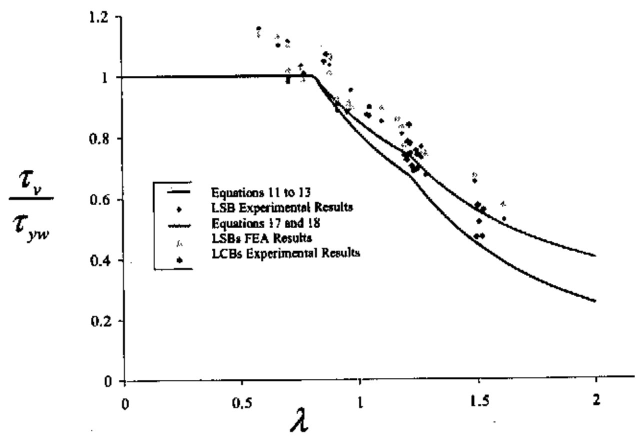


Figure 8: Comparison of Shear Strengths of Cold-formed Steel Members with DSM Based Design Equations

In general, the available limited shear test results of LCBs confirm the applicability of improved shear strength equations proposed in this research. However, unlike with LSBs, the shear strengths from two tests in the elastic buckling region are below the design curve predicted by Eq. 18. Finite element analyses of LCBs should be undertaken to investigate this further.

X. CONCLUSIONS

This paper has presented the details of an investigation into the ultimate shear strength behaviour of a new cold-formed steel beam known as LiteSteel beams. Finite element models of LSBs in shear were developed and validated by comparing their results with shear test results. Nonlinear finite element analyses were able to predict the ultimate shear capacities of LSBs with very good accuracy.

Both tests and finite element analyses showed the presence of significant reserve strength beyond elastic buckling for LSBs in shear. Therefore new shear strength design equations were proposed within the guidelines of the current Australian and American cold-formed steel structures codes and the new Direct Strength Method. The ultimate shear capacities of LSBs from tests and nonlinear finite element analyses were compared with the current AS/NZS 4600 design equations and the proposed shear design equations. This comparison shows that the current design rules in AS/NZS 4600 are conservative for the shear buckling design of LSBs. The proposed shear design equations are able to predict the shear capacities of LSBs accurately.

This research has also proposed new shear strength formulae for lipped channel beams based on the current design equations for shear strength given in AISI (2007) using the same approach used for LSBs.

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