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Shear Strength Tests of Lipped Channel Beams

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Abstract

This paper presents the details of experimental studies on the shear behaviour and strength of lipped channel beams (LCBs). The LCB sections are commonly used as flexural members in residential, industrial and commercial buildings. To ensure safe and efficient designs of LCBs, many research studies have been undertaken on the flexural behaviour of LCBs. To date, however, limited research has been conducted into the strength of LCB sections subject to shear actions. Therefore a detailed experimental study involving 20 tests was undertaken to investigate the shear behaviour and strength of LCBs. This research has shown the presence of increased shear capacity of LCBs due to the additional fixity along the web to flange juncture, but the current design rules (AS/NZS 4600 and AISI) ignore this effect and were thus found to be conservative. Therefore they were modified by including a higher elastic shear buckling coefficient. Ultimate shear capacity results obtained from the shear tests were compared with the modified shear capacity design rules. It was found that they are still conservative as they ignore the presence of post-buckling strength. Hence the AS/NZS 4600 and AISI design rules were further modified to include the available post-buckling strength. Suitable design rules were also developed under the direct strength method (DSM) format. This paper presents the details of this study and the results including the modified shear design rules.

1. Introduction

Over the past couple of decades cold-formed high strength steel members have been increasingly used as primary load bearing components in residential, commercial and industrial buildings. They are used in applications such as building frames, roof trusses, purlins and girts, floor framing and many other load bearing components. The increasing use of cold-formed steel sections has enhanced interest in the design and efficiency of cold-formed steel members. Lipped channel and Z-sections are commonly used in the light gauge steel framing industry due to their high strength-to-weight ratio, economy of transportation and handling, ease of fabrication, simple erection and installation.

In steel building systems, lipped channel beams (LCBs) are commonly used as flexural members, for example, floor joists and bearers. For LCBs to be used as flexural members, their flexural and shear capacities must be known. However, past research has been limited on shear capacities in comparison with flexural capacities of LCBs. In the conventional shear design method of LCBs, the web shear buckling behavior is considered in isolation without considering the effect of flange rigidity. LaBoube and Yu (1978) obtained the ultimate strengths of LCBs by assuming that the web-flange juncture of LCB is simply supported. Here single web side plates were used at the end supports and the loading point in order to eliminate any torsional loading of test beams and web crippling of flanges and flange bearing failures. However, Keerthan and Mahendran (2013) found that the web-flange juncture in LCBs has some fixity. Pham and Hancock (2010) conducted an experimental study to determine the ultimate shear capacity of high strength cold-formed channel sections subjected to a predominantly shear action. There is a need to develop suitable design rules that include the effects of not only the fixity at the web-flange juncture, but also possible post-buckling strength.

In this research the shear behaviour of LCBs including their capacities was investigated using experimental studies. Experimental results were then used to develop suitable equations for predicting the improved shear capacities of LCBs. This paper presents the details of a series of shear tests of LCBs, and the results. Experimental shear capacities are compared with the predicted shear capacities using the current and improved design rules.

2. Shear Tests

It is important that key parameters are chosen carefully in the design of a test plan. In order to fully investigate the shear behaviour and strength of LCBs, several important issues were considered when deciding these parameters such as the aspect ratios (shear span (a)/clear height of web (d_1)) and the clear height of web to thickness of web ratios (d_1/t_w). In this experimental study, LCB test specimens were designed to fail in shear prior to reaching other section capacities.

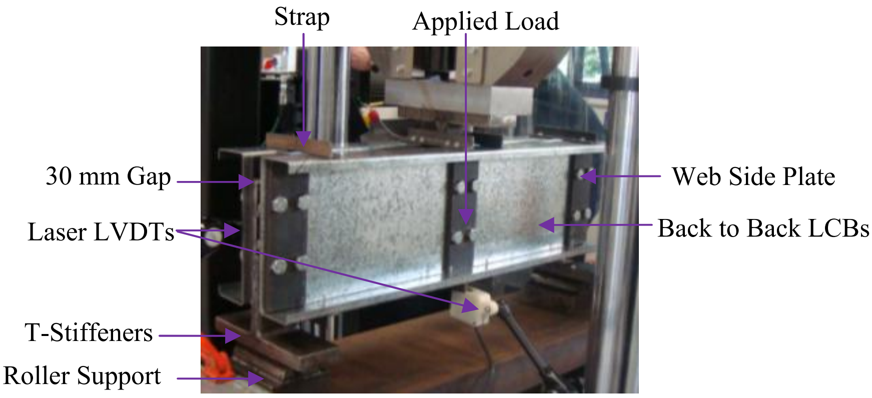


Figure 1: Test set-up

Tests were undertaken using an Instron testing machine. Figure 1 shows the shear test set-up. In order to simulate a primarily shear condition, relatively short test beams of span based on two aspect ratios (a/d_1) of 1.0 and 1.5 were selected. Two LCB sections were bolted back to back using three T-shaped stiffeners and web side plates located at the end supports and the loading point in order to eliminate any torsional loading of test beams and possible web crippling of flanges and flange bearing failures. A 30 mm gap was included between the two LCB sections (Figure 1) to allow the test beams to behave independently while remaining together to resist torsional effects. Flanges were restrained by straps to eliminate any flange distortion due to distortional buckling or unbalanced shear flow. The measuring system was set-up to record the applied load and associated test beam deflections. Two laser displacement transducers were located on the test beam under the loading point and web panel to measure the vertical and lateral deflections, respectively (see Figure 1). Test beam was loaded at midspan by moving the cross-head of the testing machine at a constant rate of 0.7 mm/minute until the test beam failed. Table 1 shows the lipped channel beam specimens tested in this experimental study and the shear capacities obtained from them.

Table 1: Shear capacities of LCBs with straps

LCB Sections	d_1 (mm)	t_w (mm)	f_y (MPa)	a/d_1	Shear capacity (kN)		
					Test	Eqs. (1) to (3)	AS4600
200x75x15x1.9	197.0	1.92	515	1.0	75.0	75.8	60.7
250x75x18x1.9	245.0	1.90	515	1.0	69.4	69.6	47.3
160x65x15x1.9	156.8	1.92	515	1.0	73.8	75.6	73.2
200x75x15x1.5	197.0	1.51	537	1.0	57.0	44.7	29.5
250x75x18x1.5	247.3	1.49	537	1.0	53.2	43.3	22.6
160x65x15x1.5	157.5	1.51	537	1.0	54.5	47.2	37.0
120x50x18x1.5	116.8	1.49	537	1.0	43.3	46.3	45.0
200x75x15x1.95	198.0	1.93	271	1.0	55.1	54.2	53.6
250x75x18x1.95	248.3	1.94	271	1.0	60.3	57.9	49.7
160x65x15x1.95	158.0	1.94	271	1.0	52.2	49.9	53.2
120x50x18x1.95	118.6	1.95	271	1.0	38.1	37.6	40.1
250x75x18x1.9	247.3	1.90	515	1.5	60.7	61.1	35.7
200x75x15x1.5	196.8	1.49	537	1.5	38.1	38.3	21.7
250x75x18x1.5	247.2	1.50	537	1.5	42.9	39.7	17.6
160x65x15x1.5	156.4	1.51	537	1.5	39.7	40.7	28.4

Figure 2 shows the load-deflection curves for the shear test of 250x75x18x1.5 LCB section (aspect ratio =1.0). At Point 1, the web began to deflect out of plane and the beam reached the ultimate shear capacity of 53.2 kN (applied load of 212.8 kN/4) at Point 2. This confirms that LCBs have post-buckling strength in shear due to tension field action. Figures 3 (a) and (b) show the failure modes of 160x65x15x1.9 LCB with an aspect ratio of 1.0 and 250x75x18x1.9 LCB with an aspect ratio of 1.5, respectively.

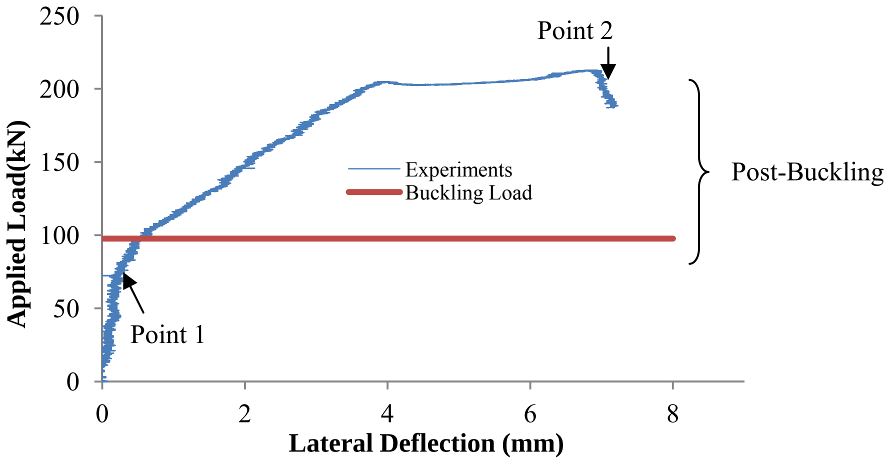
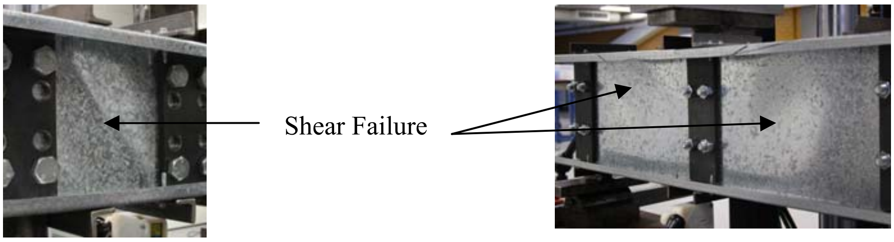


Figure 2: Plot of applied load versus lateral deflection (250x75x18x1.5)



(a) 160x65x15x1.90 (Aspect Ratio = 1.0)

(b) 250x75x18x1.90 (Aspect Ratio = 1.5)

Figure 3: Shear failure modes of LCBs

In some building applications, LCBs are not used with straps at the supports. To investigate the effect of straps on the shear behaviour and strength of LCBs, five more shear tests were conducted. Table 2 shows the shear capacities of LCBs when straps were not used and compared them with the corresponding shear capacities when straps were used. It confirms that there is about 9 to 20% reduction in the shear capacity of LCB when straps are not attached to its flanges. Here flange distortion occurs due to distortional buckling or unbalanced shear flow in LCBs.

Table 2: Shear capacities of LCBs without straps

LCB section	Aspect ratio (a/d_1)	Shear capacity (kN)		% Reduction
		With straps	Without straps	
200x75x15x1.50	1.0	57.0	45.5	20
120x50x18x1.50	1.0	43.3	39.6	9
120x50x18x1.95	1.0	38.1	33.4	12
250x75x18x1.50	1.5	42.9	37.9	12
200x75x15x1.95	1.5	39.5	32.4	18

3. Design Equations for the Shear Capacity of LCBs

In Table 1 experimentally obtained shear capacities were compared with the corresponding shear capacities predicted by the current design rules in AS/NZS 4600. This comparison clearly shows that the shear capacities predicted by AS/NZS 4600 design rules are less than experimental shear capacities. AS/NZS 4600 design rules are very conservative as they do not include the post-buckling strength observed in the shear tests and the increased shear buckling coefficient determined from numerical studies of Keerthan and Mahendran (2013).

Improved shear capacity equations were therefore proposed based on the current shear capacity design equations in AISI (2007) and experimental results. Equations (1) to (3) present the shear capacity (V_v) which include the available post-buckling strength in LCBs and the additional fixity at the web-flange juncture. In these equations, V_y = shear yield capacity, V_{cr} = elastic shear buckling capacity and V_{icr} = inelastic shear buckling capacity. The increased shear buckling coefficient given by Equation 7 (k_{LCB}) is also included to allow for the additional fixity in the web-flange juncture of LCBs.

$$V_v = V_y \quad \text{for} \quad \frac{d_1}{t_w} \le \sqrt{\frac{Ek_{LCB}}{f_y}} \quad \text{(Shear yielding)} \tag{1}$$

$$V_v = V_{icr} + 0.2(V_y - V_{icr}) \text{ for } \sqrt{\frac{Ek_{LCB}}{f_y}} < \frac{d_1}{t_w} \le 1.508\sqrt{\frac{Ek_{LCB}}{f_y}} \text{ (Inelastic shear buckling)} \tag{2}$$

$$V_v = V_{cr} + 0.2(V_y - V_{cr}) \quad \text{for} \quad \frac{d_1}{t_w} > 1.508\sqrt{\frac{Ek_{LCB}}{f_y}} \quad \text{(Elastic shear buckling)} \tag{3}$$

$$V_y = 0.6 f_y d_1 t_w \tag{4}$$

$$V_{cr} = \frac{k_{LCB}\pi^2 E t_w^3}{12(1-\upsilon^2)d_1} \tag{5}$$

$$V_{icr} = 0.6 t_w^2 \sqrt{Ek_{LCB} f_y} \tag{6}$$

$$k_{LCB} = k_{ss} + 0.23(k_{sf} - k_{ss}) \tag{7}$$

$$k_{ss} = 5.34 + \frac{4}{(a/d_1)^2} \quad \text{for } \frac{a}{d_1} \geq 1 \tag{8}$$

$$k_{ss} = 4 + \frac{5.34}{(a/d_1)^2} \quad \text{for } \frac{a}{d_1} < 1 \tag{9}$$

$$k_{sf} = \frac{5.34}{(a/d_1)^2} + \frac{2.31}{(a/d_1)} - 3.44 + 8.39(a/d_1) \quad \text{for} \quad \frac{a}{d_1} < 1 \tag{10}$$

$$k_{sf} = 8.98 + \frac{5.61}{(a/d^1)^2} - \frac{1.99}{(a/d_1)^3} \quad \text{fo} \frac{a}{d_1} \geq 1 \tag{11}$$

where k_{ss}, k_{sf} = shear buckling coefficients of plates with simple-simple and simple-fixed boundary conditions. a = shear span of web, d_1 = clear height of web, f_y = web yield stress.

New design equations were also proposed for the shear capacity of LCBs in a similar manner to those of the section moment capacity of beams subject to local buckling (Equations 12 and 13) using test results. In these equations the Direct Strength Method based nominal shear capacity (V_v) is proposed using the local buckling (M_{sl}) equation where M_{sl}, M_{ol} and M_y are replaced by V_v, V_{cr} (elastic buckling capacity in shear) and V_y (shear yield capacity), respectively. As for hot-rolled I-sections, only two regions based on shear yielding, and elastic and inelastic shear buckling, were considered. In these equations, a power coefficient of 0.55 was used instead of 0.4 based on the experimental results of LCBs.

$$V_v = 0.6 f_y d_1 t_w \qquad \frac{d_1}{t_w} \leq \sqrt{\frac{Ek_{LCB}}{f_y}} \tag{12}$$

$$V_v = \left[1 - 0.15\left(\frac{V_{cr}}{V_y}\right)^{0.55}\right]\left(\frac{V_{cr}}{V_y}\right)^{0.55} V_y \qquad \frac{d_1}{t_w} > \sqrt{\frac{Ek_{LCB}}{f_y}} \tag{13}$$

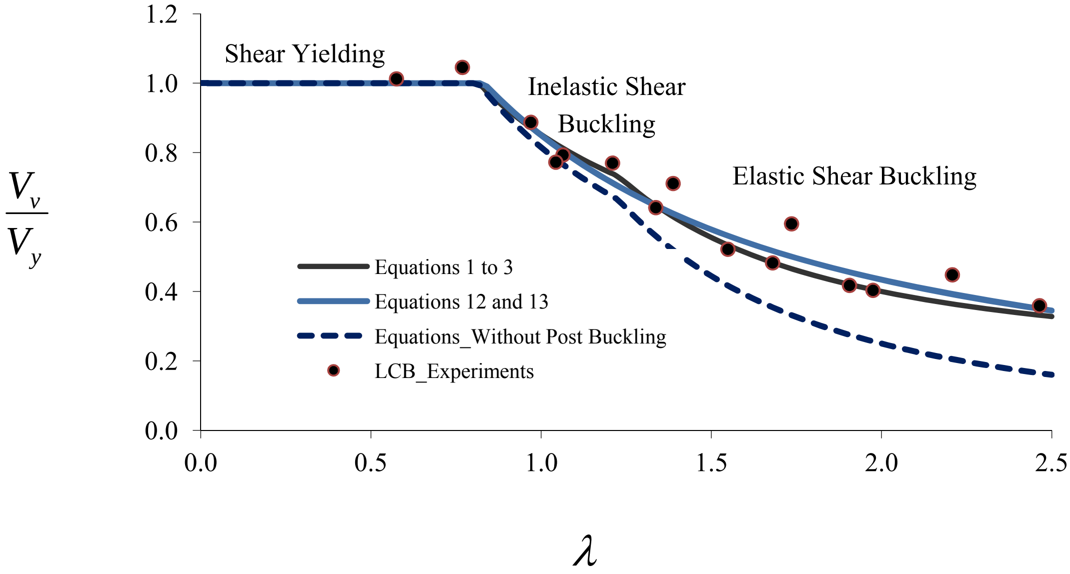


Figure 4: Comparison of experimental shear capacities of LCBs with DSM based design equations

Experimental ultimate shear capacity results are also calculated within the DSM format and are shown in Figure 4. Here slenderness (λ) was calculated using Equation 14. Figure 4 shows the non-dimensional shear capacity curve for LCBs and compares with experimental results. Equations (1) to (3) and (13) predict the shear capacities accurately as they include the available post-buckling strength and additional fixity at the web-flange juncture.

$$\lambda = \sqrt{\left(\frac{V_y}{V_{cr}}\right)} = 0.815\left(\frac{d_1}{t_w}\right)\sqrt{\left(\frac{f_y}{Ek_{LCB}}\right)} \tag{14}$$

Due to lack of experimental evidence on the shear capacity of plates without stiffeners, AS/NZS 4600 (SA, 2005) design equations do not include the post-buckling strength in shear, and the design shear stress in webs is therefore limited by the elastic buckling capacity. It was found that AS/NZS 4600 design equations are very conservative for the shear capacity of LCBs. Test results shown in Figure 4 show that there is considerable amount of post-buckling strength for LCBs subjected to shear. Hence post-buckling shear strength can be taken into account in the design of LCBs, in particular for those with larger clear web height to thickness ratios (d_1/t_w). This research has shown that the currently available deign rules (SA, 2005) for the shear capacity of LCBs must be modified.

4. Conclusions

This paper has presented the details of an experimental study of 20 shear tests into the shear behaviour and strength of lipped channel beams (LCB). Comparison of ultimate shear capacities from tests showed that AS/NZS 4600 (SA, 2005) and AISI (2007) design equations are very conservative for the shear design of LCBs. It was found that there is considerable amount of post-buckling strength for LCBs subjected to shear. New shear capacity equations for LCBs were proposed based on the current shear strength design equations in AISI (2007) and experimental results. Suitable shear design rules were also developed under the direct strength method (DSM) format. Test results show that there is about 9 to 20% reduction in the shear capacity of LCBs when straps are not attached to their flanges.

5. References

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