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Experimental investigation and design of lipped channel beams in shear

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ABSTRACT

Cold-formed high strength steel members are increasingly used as primary load bearing components in low rise buildings. Lipped channel beam (LCB) is one of the most commonly used flexural members in these applications. In this research an experimental study was undertaken to investigate the shear behaviour and strengths of LCB sections. Simply supported test specimens of back to back LCBs with aspect ratios of 1.0 and 1.5 were loaded at mid-span until failure. Test specimens were chosen such that all three types of shear failure (shear yielding, inelastic and elastic shear buckling) occurred in the tests. The ultimate shear capacity results obtained from the tests were compared with the predictions from the current design rules in Australian/NewZealand and American cold-formed steel design standards. This comparison showed that these shear design rules are very conservative as they did not include the post-buckling strength observed in the shear tests and the higher shear buckling coefficient due to the additional fixity along the web-flange juncture. Improved shear design equations are proposed in this paper by including the above beneficial effects. Suitable lower bound design rules were also developed under the direct strength method format. This paper presents the details of this experimental study and the results including the improved design rules for the shear capacity of LCBs. It also includes the details of tests of LCBs subject to combined shear and flange distortion, and combined bending and shear actions, and proposes suitable design rules to predict the capacities in these cases.

1. Introduction

During the past few decades, the use of cold-formed high strength steel members as primary load bearing components has progressively increased in building construction. In particular, light steel frame industry extensively uses cold-formed steel members such as channel, lipped channel and Z-sections due to their high strength to weight ratio, economy of transportation and handling, ease of fabrication, simple erection and installation. Availability of high strength (550 MPa) and very thin (<1 mm) steels and advanced roll-forming technologies has contributed significantly to the increasing use of cold-formed steel members in building industries. Fig. 1 shows one of the commonly used cold-formed steel sections, lipped channel beam (LCB) while Table 1 presents the currently available LCB sections and their dimensions [1].

LCBs are commonly used as flexural members in steel building systems, for example, floor joists and bearers (see Fig. 1). For LCBs to be used as flexural members, both their flexural and shear capacities must be known. In this research shear behaviour and strength of LCBs was investigated using experimental studies. This paper presents the details of a series of primarily shear tests of LCBs, and the results. Experimental shear capacities are compared with the predicted shear capacities using the current design rules. Suitable design rules are then developed based on the current shear design equations in AISI S100 [2] and the direct strength method (DSM). This paper also presents the details of the tests of LCBs subject to combined shear and flange distortion, and combined bending and shear actions, and their results.

2. Review of the shear capacities and design of LCBs

In the shear design of lipped channel beams, the conventional approach was to investigate web buckling alone and ignore the effect of flanges on the shear buckling behaviour. The shear strength of cold-formed steel lipped channel beam was first investigated by LaBoube and Yu [3] with due consideration given to the web slenderness ratio, the edge support conditions provided by the flanges with varying flat width to thickness ratios, and the mechanical properties of steel. Distortional buckling failures were not observed in LaBoube and Yu's [3] experiments since their test specimens consisted of two LCBs connected by angle sections at

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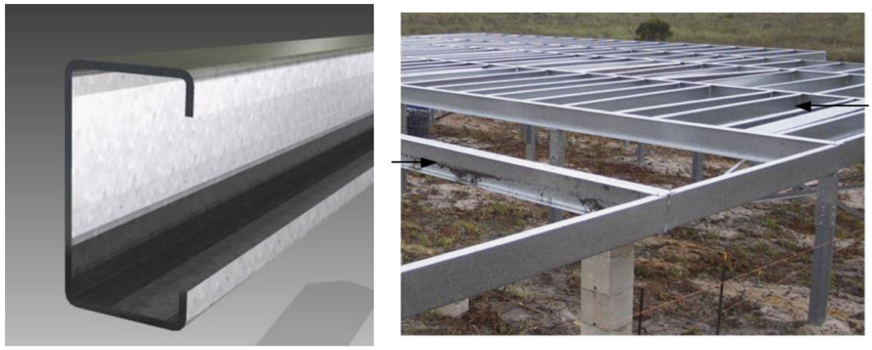
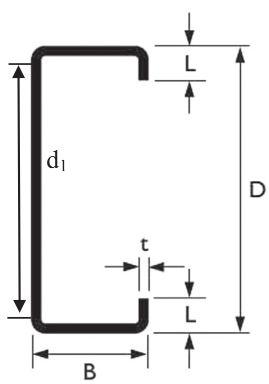


Fig. 1. Lipped channel beam.

Table 1
Currently available LCB sections and their dimensions [1].

Section	D (mm)	B (mm)	L (mm)	t (mm)	f_{yw} (MPa)
C100-10	102	51	12.5	1.0	550
C100-12	102	51	13.0	1.2	500
C100-15	102	51	14.0	1.5	450
C100-19	102	51	15.0	1.9	450
C150-10	152	64	14.5	1.0	550
C150-12	152	64	15.0	1.2	500
C150-15	152	64	16.0	1.5	450
C150-19	152	64	17.0	1.9	450
C150-24	152	64	18.5	2.4	450
C200-15	203	76	16.0	1.5	450
C200-19	203	76	19.5	1.9	450
C200-24	203	76	21.0	2.4	450
C250-19	254	76	19.0	1.9	450
C250-24	254	76	20.5	2.4	450
C300-24	300	96	28.0	2.4	450
C300-30	300	96	31.5	3.0	450
C350-30	350	125	30.0	3.0	450



their compression and tension flanges. Their test set-up was similar to that shown in Fig. 2 with the main difference in terms of the number of bolt rows at the supports. As shown in Fig. 3(a), their LCB specimens were connected together using only three rows of bolts at their supports with heavy web side plates on one side only. LaBoube and Yu [3] obtained the ultimate strengths of LCBs by assuming that the web-flange juncture of LCB is simply supported. Single web side plates were used at the end supports and the loading point to eliminate any torsional loading of test beams, web crippling of flanges and flange bearing failures. LaBoube and Yu [3] also proposed suitable design equations for the shear strength of cold-formed steel beams, which have been adopted in AISI S100 [2] and AS/NZS 4600 [4]. AS/NZS 4600 and AISI S100 shear design equations are based on simply supported conditions at the web-flange juncture and do not include the post-buckling shear strength in LCBs. These equations for the shear capacity (V_v) are given next.

$$V_v = V_y = 0.6 f_{yw} d_1 t_w \quad \text{for} \quad \frac{d_1}{t_w} \le \sqrt{\frac{Ek_v}{f_{yw}}} \tag{1}$$

$$V_v = V_i = 0.6 t_w^2 \sqrt{Ek_v f_{yw}} \quad \text{for} \quad \sqrt{\frac{Ek_v}{f_{yw}}} < \frac{d_1}{t_w} \le 1.508\sqrt{\frac{Ek_v}{f_{yw}}} \tag{2}$$

$$V_v = V_{cr} = \frac{k_v \pi^2 E t_w^3}{12(1-\nu^2) d_1} \quad \text{for} \quad \frac{d_1}{t_w} > 1.508\sqrt{\frac{Ek_v}{f_{yw}}} \tag{3}$$

where V_y=Shear yield capacity, V_i=Inelastic shear buckling capacity, V_{cr}=Elastic shear bucking capacity, d_1=depth of the flat portion of web measured along the plane of the web, t_w=web thickness, f_{yw}=web yield stress, E=Young's modulus, ν=Poisson's ratio and k_v is the elastic shear buckling coefficient, which is determined as follows.

For beams with transverse stiffeners

$$k_v = 5.34 + \frac{4}{(a/d_1)^2} \text{ for } \frac{a}{d_1} \ge 1 \tag{4}$$

$$k_v = 4 + \frac{5.34}{(a/d_1)^2} \text{ for } \frac{a}{d_1} < 1 \tag{5}$$

where 'a'=shear panel length and (a/d_1)=aspect ratio. k_v is taken as 5.34 for unstiffened webs.

Keerthan and Mahendran [5] investigated the elastic shear buckling behaviour of a cold-formed steel hollow flange channel section known as LiteSteel beams (LSBs) [6] and developed a simple predictive equation for the increased shear buckling coefficient (k_v) due to the presence of higher fixity along the web to flange juncture. Keerthan and Mahendran [4,7–9] continued their research using experimental and numerical studies and developed suitable design equations for the shear capacity of hollow flange channel beams (V_v) by including the available post-buckling strength and the increased shear buckling coefficient (k_v). They also developed suitable DSM based design equations for the shear capacity of hollow flange channel beams.

Pham and Hancock [10] investigated the elastic buckling of unlipped and lipped channel section members subject to shear using an isoparametric spline finite strip method. They found that flanges can have a significant influence on the shear buckling capacity of thin-walled channel sections and that lack of lateral restraint for sections with narrow flanges can lead to premature buckling of the section in a twisting and lateral buckling mode. However, they did not propose a simple equation to determine the shear buckling coefficient of LCBs. Hence Keerthan and Mahendran [8,9] developed a simple predictive equation for the shear buckling coefficient of LCBs based on finite element analyses (FEA) to allow for the available web to flange fixity in LCBs.

Pham and Hancock [11] conducted both experimental and numerical studies to investigate the shear behaviour of high strength cold-formed steel lipped channel sections. Suitable design equations for the shear capacity of LCBs (Eqs. (6) and (7)) were then proposed in Pham and Hancock [12]. These equations predict the shear strength of LCBs which include their available post-buckling strength and the effect of additional fixity at the web-flange juncture. In these equations the DSM 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.

$$V_v = V_y \quad \text{for} \quad \frac{d_1}{t_w} \le \sqrt{\frac{Ek_v}{f_{yw}}} \tag{6}$$