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Improved Shear Design Rules for Lipped Channel Beams with Web Openings

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Abstract: Cold-formed steel Lipped Channel Beams (LCB) with web openings are commonly used as floor joists and bearers in building structures. The shear behaviour of these beams is more complicated and their shear capacities are considerably reduced by the presence of web openings. However, limited research has been undertaken on the shear behaviour and strength of LCBs with web openings. Hence a detailed numerical study was undertaken to investigate the shear behaviour and strength of LCBs with web openings. Finite element models of simply supported LCBs under a mid-span load with aspect ratios of 1.0 and 1.5 were developed and validated by comparing their results with test results. They were then used in a detailed parametric study to investigate the effects of various influential parameters. Experimental and numerical results showed that the current design rules in cold-formed steel structures design codes are very conservative. Improved design equations were therefore proposed for the shear strength of LCBs with web openings based on both experimental and numerical results. This paper presents the details of finite element modelling of LCBs with web openings, validation of finite element models, and the development of improved shear design rules. The proposed shear design rules in this paper can be considered for inclusion in the future versions of cold-formed steel design codes.

Keywords: *Lipped channel beams, Web openings, Shear capacity, Cold-formed steel structures, Finite element analyses, Design rules.*

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1. Introduction

Until recently, steel construction has recognized hot-rolled steel members as the most popular and widely used structural steel member type. However, over the past couple of decades, the use of cold-formed high strength steel members has been progressively integrated in steel construction as primary load bearing components. Cold-formed steel sections are frequently used in residential, commercial and industrial buildings due to their notable strength to weight ratio, ease of fabrication and ease of construction (see Figure 1). Cold-formed steel structural members such as the 'C', 'Z' or tubular sections are commonly used in floor and roof framing systems (i.e. purlins, joists and bearers), wall and truss systems and many other load bearing systems. The increasing use of cold-formed steel sections has led to greater interest in the design and efficiency of cold-formed steel members. Among them, lipped channel sections are commonly used due to their high strength-to-weight ratio, economy of transportation and handling, ease of fabrication, simple erection and installation. Figure 1(b) shows the use of lipped channel beams (LCB) in floor systems. Many applications in steel floor systems include openings in the web of joists or bearers so that building services can be located within them as shown in Figure 1 (b). Although different shapes can be used for these web openings, the most common shape used in floor systems is circular (Figure 1 (a)). Pokharel and Mahendran's [1] study based on finite element analyses also recommended the use of circular web openings for cold-formed steel beams such as LiteSteel beams [2].

Past research on the shear behavior and strength of cold-formed steel sections containing web openings has been limited to 'C' sections [3,4,5]. Shan et al. [4] recommended that the nominal shear capacity of cold-formed lipped channel beams with web openings can be calculated using a reduction factor (q_s) applied to the solid web strength of the shear element. Eiler [5] extended Shan et al.'s [4] work to include the behaviour of web elements with openings subjected to linearly varying shear force. In Eiler's tests, cold-formed steel beams with web openings were subjected to a uniform load (not constant shear). Eiler [5] also proposed suitable design equations for the shear strength of cold-formed steel beams with web openings. These shear strength equations have been adopted in AISI [6] and AS/NZS 4600 [7] design standards for cold-formed steel structures.

The use of web openings in cold-formed steel beams such as lipped channel beams significantly reduces their shear capacities due to the reduced web area. Many parameters affect the shear capacity of cold-formed steel beams containing web openings. They are the shape, size and location of the web openings and also the slenderness of the web element. The main aim of this research is to investigate the effect of circular web openings of varying diameters on the shear capacities of LCB sections using detailed finite element analyses (FEA) and experiments, and to investigate the accuracy of currently available design rules. This paper presents the details of the numerical study into the shear behaviour and design of LCBs with web openings. Shear capacities from FEA and experiments are compared with the predicted shear capacities using the current design rules in AS/NZS 4600 [7] and the North American Specification [6], based on which improved shear design rules are proposed.

2. Shear Behaviour and Design of LCBs with Web Openings

Current shear design rules for cold-formed steel beams with web openings are based on a reduction factor (q_s) defined as the ratio of the nominal shear capacity of LCBs with web openings (V_{nl}) to the nominal shear capacity of LCBs without web openings (V_v). Hence suitable design rules are also needed to predict $V_{v,}$ and this section presents the currently available design rules for both V_v and q_s.

In general the shear design of lipped channel beams considers web shear buckling behavior in isolation without the effect of flange rigidity. LaBoube and Yu [8] investigated the shear strength of lipped channel beams using single web side plate at the end supports and the loading point. Their proposed shear strength equations are based on simply supported conditions at the web-flange juncture and also without including the post-buckling strength in LCBs. Current shear capacity equations for V_v in AS/NZS 4600 [7] and AISI [6] are based on LaBoube and Yu's research. Pham and Hancock [9,10] performed both experimental and numerical studies to investigate the shear behaviour of high strength cold-formed steel channel sections. They proposed improved design equations for the shear capacity of channel sections (Equations (1) to (3)) by including the available post-buckling strength in LCBs and the effect of additional fixity at the web-flange juncture [11]. In these equations based on the direct strength method, the nominal shear capacity (V_v) is proposed using V_{cr} (elastic buckling capacity in shear) and V_y (shear yield capacity).

$$V_V = \left[1 - 0.15\left(\frac{V_{cr}}{V_y}\right)^{0.4}\right]\left(\frac{V_{cr}}{V_y}\right)^{0.4} V_y \tag{1}$$

$$V_y = 0.6 A_w f_{yw} \tag{2}$$

$$V_{cr} = \frac{k_v \pi^2 E A_w}{12(1-\upsilon^2)\left(\frac{d_1}{t_w}\right)^2} \tag{3}$$

where k_v is the elastic shear buckling coefficient of the channel section [11].

Keerthan and Mahendran [12] also proposed improved shear strength equations for the new channel sections with two rectangular hollow flanges known as LiteSteel beams (LSB) based on the current shear strength design equations in AISI [6], experimental and finite element analysis results. They then extended their research work to LCBs subjected to primarily shear action. Equations (4) to (6) present their proposed shear strength equations, which include the available post-buckling strength in LCBs and the additional fixity at the web-flange juncture. The shear capacity in kN can be obtained by multiplying the shear strength (τ_v) by its web area of d_1t_w. The increased shear buckling coefficient given by Equation 7 (k_{LCB}) is included to allow for the additional fixity in the web-flange juncture of LCBs [12].

$$\tau_v = \tau_{yw} \quad \text{for} \quad \frac{d_1}{t_w} \le \sqrt{\frac{Ek_{LCB}}{f_{yw}}} \tag{4}$$

$$\tau_v = \tau_i + 0.2(\tau_{yw} - \tau_i) \quad \text{for} \quad \sqrt{\frac{Ek_{LCB}}{f_{yw}}} < \frac{d_1}{t_w} \le 1.508\sqrt{\frac{Ek_{LCB}}{f_{yw}}} \tag{5}$$

$$\tau_v = \tau_e + 0.2(\tau_{yw} - \tau_e) \quad \text{for} \quad \frac{d_1}{t_w} > 1.508\sqrt{\frac{Ek_{LCB}}{f_{yw}}} \tag{6}$$

where

$$\tau_i = 0.6 f_{yw} \qquad \tau_i = \frac{0.6\sqrt{Ek_{LCB} f_{yw}}}{\left[\frac{d_1}{t_w}\right]} \qquad \tau_e = \frac{0.905 E k_{LCB}}{\left[\frac{d_1}{t_w}\right]^2}$$

For LCBs $k_{LCB} = k_{ss} + 0.23(k_{sf} - k_{ss})$ (7)

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

$$k_{sf} = 8.98 + \frac{5.61}{(a/d_1)^2} - \frac{1.99}{(a/d_1)^3} \quad \text{for} \quad \frac{a}{d_1} \ge 1 \tag{9}$$

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 = flat portion of clear height of web, and f_{yw} = web yield stress.

Suitable equations for V_v have been given above and now the equations for the shear capacity reduction factor q_s are given. AS/NZS 4600 [7] and AISI [6] present the following design rules for cold-formed steel beams with circular openings based on Eiler's [5] research.

$$V_{nl} = q_s V_v \tag{10}$$

$$q_s = 1 \qquad \frac{c}{t} > 54 \tag{11}$$

$$q_s = \frac{c}{54t} \qquad 5 \le \frac{c}{t} < 54 \tag{12}$$

$$c = \frac{d_1}{2} - \frac{d_{wh}}{2.83} \tag{13}$$

$$\frac{d_{wh}}{d_1} < 0.7\,,\ \frac{d_{wh}}{t_w} \le 200\,,\ d_{wh} > 15\,\text{mm},\ d_{wh} \le 150\,\text{mm}$$

where d_{wh} is the depth of web openings

Shan et al.'s [4] design equations for q_s are as follows.

$$q_s = -3.66\left(\frac{d_{wh}}{d_1}\right) + 1.71 \qquad \text{for} \qquad \frac{d_{wh}}{d_1} \le 0.38 \tag{14}$$

$$q_s = -0.38\left(\frac{d_{wh}}{d_1}\right) + 0.46 \qquad \text{for} \quad 0.38 < \frac{d_{wh}}{d_1} \le 1.0 \tag{15}$$

Equations 16 to 18 show the proposed design equations for the shear capacity reduction factor for LiteSteel beams developed by Keerthan and Mahendran [13].

$$q_s = 1 - 0.6\left[\frac{d_{wh}}{d_1}\right] \qquad \text{for} \quad 0 < \frac{d_{wh}}{d_1} \le 0.25 \tag{16}$$

$$q_s = 1.1 - \left[\frac{d_{wh}}{d_1}\right] \qquad \text{for} \quad 0.25 < \frac{d_{wh}}{d_1} \le 0.65 \tag{17}$$

$$q_s = 0.84 - 0.6\left[\frac{d_{wh}}{d_1}\right] \qquad \text{for} \quad 0.65 < \frac{d_{wh}}{d_1} \le 0.85 \tag{18}$$

McMahon et al. [14] also recommended a suitable linear equation for calculating q_s of LiteSteel beams with web openings as follows.

$$q_s = 1 - 0.5\left[\frac{d_{wh}}{d_1}\right] \quad \text{for} \quad 0.40 < \frac{d_{wh}}{d_1} \leq 0.80 \tag{19}$$

3. Experimental Studies of LCBs with Web Openings

Forty shear tests were conducted to investigate the shear behaviour of LCBs with web openings as shown in Table 1. In order to fully understand the shear behaviour of LCB sections with web openings several important issues were considered when deciding these parameters such as the ratio of the depth of web openings (d_{wh}) to clear height of web (d_1) and the clear web height (d_1) to web thickness (t_w). In this experimental study, test specimens of LCBs with circular web openings were designed to fail in shear prior to reaching other section capacities.

All LCB specimens were tested using the experimental set-up shown in Figure 2. 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. In order to simulate a primarily shear condition, relatively short test beams of span based on two aspect ratios (shear span a/ clear web height d_1) of 1.0 and 1.5 were selected. A 30 mm gap was included between the two LCB sections (Figure 2) to allow the test beams to behave independently while remaining together to resist torsional effects. High strength steel bolts (M16 8.8S) were used to avoid any bolt failure during testing. In the first 32 tests flanges were restrained by angle straps at the loading and support points to eliminate any flange distortion due to distortional buckling or unbalanced shear flow (see Figure 2). However, eight tests were also conducted without straps to study the effects of not using them. The support system was designed to ensure that the test specimen acted as a simply supported beam with pinned supports at each end. The measuring system was set-up to record the applied load and associated test beam deflections. The cross-head of the testing machine was moved 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 including their measured dimensions and web yield stresses. Further details of this experimental study and the results are presented in Keerthan and Mahendran [15].

Combined shear and bending actions can influence the failures in longer beams with low grade steel beams, ie. beams with a higher aspect ratio and a lower flange yield stress. Test results showed that shear capacities of 120x50x18x1.9 LCB, 120x50x18x1.95 LCB and 200x75x15x1.9 LCB (aspect ratio = 1.5) were reduced by 21%, 10%, and 10%, respectively, due to this combined action. In these three cases their shear capacities were calculated using Equations 4 to 6 that were developed to predict the capacities of beams subjected to a primarily shear action. In other cases, they were obtained from experiments or finite element analyses.

4. Finite Element Analyses of LCBs with Web Openings

4.1. General

This section describes the finite element model that was used to investigate the shear behaviour of LCBs with web openings. For this purpose, a general purpose finite element program, ABAQUS Version 6.7 [16], was used. Appropriate parameters were chosen for the geometry, mechanical properties, loading and support conditions, and initial geometric imperfections. Experimental study [15] included 40 shear tests of simply supported back to back LCBs with web openings under a three-point loading arrangement as shown in Figure 2. Finite element models of single LCBs with shear centre loading and simply supported boundary conditions were used to simulate these shear tests of back to back LCBs with web openings. The cross-section geometry of the finite element model was based on the measured dimensions and yield stresses of tested LCBs. Table 1 gives the measured dimensions and web yield stresses of tested specimens. In this table t_w and d_1 are the base metal thickness and the clear web height, and f_{yw} is the web yield stress of LCBs. Shear test results of back to back LSBs were similar to those obtained from single LSBs with a shear centre loading [12]. Hence in this study, finite element models of single LCBs with a shear centre loading and simply supported boundary conditions were used to simulate the shear tests of back to back LCBs with web openings.

ABAQUS has several element types to simulate the shear behaviour of beams with web openings. But among those, shell element was selected as it has the capability to simulate the elastic buckling and nonlinear ultimate shear behaviour of thin steel beams such as LCBs. The shell element in ABAQUS called S4R5 was used to model the shear behaviour of LCBs with web openings. This element is thin, shear flexible, isometric quadrilateral shell with four nodes and five degrees of freedom per node, utilizing reduced integration and bilinear interpolation scheme.

Finite element modelling was carried out using MD PATRAN R2.1 pre-processing facilities using which the model was created and then submitted to ABAQUS for the analysis. The results were also viewed using MD PATRAN R2.1 post-processing facilities.

The two methods of analysis employed in FEA were bifurcation buckling and non-linear static analysis. Bifurcation buckling analyses were used to obtain the eigenvectors for the inclusion of geometric imperfections. Non-linear static analyses, including the effects of large deformation and material yielding, were then employed to investigate the shear behaviour and strength of LCBs with web openings up to failure.

4.2. Finite Element Mesh of LCB with web Openings

All FE models were developed using centerline dimensions. In FEA, the selection of mesh size and layout is critical. It is desirable to use as many elements as possible in the analysis. However, such an analysis will require excessive computer time. In this analysis, adequate numbers of elements were chosen for both flanges and web based on detailed convergence studies in order to obtain sufficient accuracy of results without excessive use of computing time. In order to get accurate results, Paver Mesh was applied around the web openings. Convergence studies showed that in general, element sizes of approximately 5 mm x 5 mm provided an accurate representation of shear buckling and yielding deformations and good accuracy of results for all the sections. The geometry and finite element mesh of a typical LCB with web openings is shown in Figure 3.

4.3. Material Model and Properties of LCBs

The ABAQUS classical metal plasticity model was used in all the analyses. This model implements the von Mises yield surface to define isotropic yielding, associated plastic flow theory, and either perfect plasticity or isotropic hardening behaviour. A perfect plasticity model was adopted in all the finite element models with measured yield stresses. When the measured strain hardening in the web element was used in FEA, the shear capacity improvement was less than 1%. Hence it was not considered in the analyses. The elastic modulus and Poisson’s ratio were taken as 200,000 MPa and 0.3, respectively.

4.4. Loads and Boundary Conditions of LCBs

Simply supported boundary conditions were implemented in the finite element models of LCBs with web openings.

Left and right supports:	$u_x = 0$	$\theta_x = 1$	Mid-span loading point:	$u_x = 1$	$\theta_x = 1$
	$u_y = 1$	$\theta_y = 0$		$u_y = 0$	$\theta_y = 0$
	$u_z = 1$	$\theta_z = 0$		$u_z = 1$	$\theta_z = 0$

Experimental studies [15] showed that the failure of straps did not occur when they were used in the first 32 tests. Considering this observation, the straps were not explicitly modelled. Instead they were simulated using suitable boundary conditions as follows.

Strap Location:	$u_x = 0$	$\theta_x = 1$
	$u_y = 0$	$\theta_y = 0$
	$u_z = 1$	$\theta_z = 0$

Note: u_x, u_y and u_z are translations and θ_x, θ_y and θ_z are rotations in the x, y and z directions, respectively. 0 denotes free and 1 denotes restrained.

The vertical translation was not restrained at the loading point. Figure 4 shows the applied loads and boundary conditions of the model. Single point constraints and concentrated nodal forces were used in the finite element models to simulate the experimental boundary conditions. In order to prevent twisting, the applied point load and simply supported boundary conditions were applied at the shear centre using rigid body reference node. Shear test specimens included a 75 mm wide full height web side plate at each support to prevent lateral movement and twisting of the cross-section. These full height web side plates were modelled as rigid bodies using R3D4 elements. In ABAQUS [16] a rigid body is a collection of nodes and elements whose motion is governed by the motion of a single node, known as the rigid body reference node. The motion of the rigid body can be prescribed by applying boundary conditions at the rigid body reference node. Hence simply supported boundary conditions were applied to the node at the shear centre in order to provide an ideal pinned support.

Keerthan and Mahendran's [12] shear test results for LSBs without web openings showed that some practical support conditions were not sufficient to provide simply supported conditions while the use of full height web side plates on both sides as used in the shear tests of this study simulated the required simply supported conditions (not fixed conditions), while also eliminating the web crippling failure. In this study the same web side plate arrangement was used and hence simulated the required simply supported conditions. Other researchers also used the web side plates in cold-formed steel beams for the same reasons [4,9].

The web side plates at the supports were connected using high strength steel bolts (M16 8.8S) to avoid bolt failures during testing. Our shear tests [15] confirmed that there were no bolt or plate failures. Therefore these web side plates were modelled as rigid bodies using R3D4 elements.

4.5. Initial Geometric Imperfections and Residual Stresses of LCBs

The magnitude of local imperfections was taken as $0.006d_1$ for all LCB sections [17]. The critical imperfection shape was introduced using the *IMPERFECTION option in ABAQUS. Preliminary finite element analyses showed that the effect of residual stresses on the shear capacity of LSBs without openings is less than 1% [12]. Therefore the effect of residual stresses on the shear capacity of LCBs with web openings is also likely to be very small. It was thus decided to neglect the residual stresses in the FEA of LCBs with web openings.

4.6. Validation of Finite Element Models

It is essential to validate the developed finite element models for non-linear analyses of LCBs with web openings subjected to shear. For this purpose finite element analysis results of the first 32 shear tests conducted with angle straps were compared with those from testing, with particular attention given to the ultimate loads, load-deflection curves and failure mechanisms. They were chosen to exclude tests with any flange distortion failures. Table 2 presents a summary of the FEA results of ultimate loads of LCBs with openings in shear (V_{nl}) and a comparison of these results with the corresponding shear test results. The mean and COV of the ratio of test to FEA ultimate loads are 1.02 and 0.036, respectively. This indicates that the finite element model developed in this study is able to predict the ultimate shear capacities of LCBs with very good accuracy.

Figure 5 shows the FEA results in the form of applied load versus deflection for 160x65x15x1.9 LCB with 60 mm web openings (Test Specimen 9) and compares them with corresponding experimental results. Figure 6 shows the shear failure modes of 200x75x15x1.9 LCB with 125 mm web openings (Test Specimen 32) while Figure 7 shows the shear failure modes of 160x65x15x1.9 LCB with 60 mm web openings (Test Specimen 9). These figures demonstrate a good agreement between the results from FEA and experiments and confirm the adequacy of the developed finite element models in predicting the ultimate loads, deflections and failure modes of LCBs with web openings subjected to shear.

Figure 5(a) shows that initially the web began to deflect out of plane at 36.25 kN (applied load of 145kN/4) and reached the ultimate shear capacity of 49.5 kN (applied load of 198 kN/4). This confirms that LCBs with web openings also have considerable post-buckling strength.

Figures 8 (a) and (b) illustrate the shear behaviour of LCB with web openings by including the deformed LCB sections from the point of buckling to post-ultimate stage. Both experimental and finite element analysis results show that slender LCBs with web openings do not collapse when elastic buckling stress is reached and still have considerable post-buckling strength (Figure 8 (a)).

Figures 9 (a) to (c) show the effect of web openings (d_{wh}) and web thickness (t_w) on the post-buckling strength of LCBs. Figures 9 (a) and (b) show that 200x75x15x1.9 LCB with larger web openings (100 mm) has a higher post-buckling strength than that with smaller web openings (60 mm) where as Figures 9 (b) and (c) show that thinner LCB (200x75x15x1.9) has a higher post-buckling strength than the thicker LCB (200x75x15x2.5) with 100 mm web openings.

5. Parametric Study of LCBs with Web Openings

Following the validation of the developed finite element models, a detailed parametric study was undertaken based on the validated model to develop an extensive shear strength data base and then to use them to develop improved design equations for LCBs with web openings. In this study an aspect ratio of 1.0 was used with five LCB sections, 120x50x18x1.95 LCB, 160x65x15x1.9 LCB, 200x75x15x1.0 LCB, 200x75x15x1.5 LCB and 200x75x15x1.9 LCB. Nominal dimensions (t_w and d_1) and yield stresses (250, 400 and 500 MPa) were used in the analyses. The ultimate shear capacities of LCBs with web openings (V_{nl}) and the corresponding shear capacity reduction factors for varying ratios of d_{wh}/d_1 obtained from this study are given in Table 3. The ultimate shear capacities of the selected LCBs without web openings (V_v) were also obtained from FEA. The shear capacity reduction factor q_s was calculated as the ratio of V_{nl} to V_v. This table shows that the ultimate shear capacities decrease with increasing depth of web openings. In order to investigate the effect of web yield stress (f_{yw}) and clear height of web to web thickness ratio (d_1/t_w) on the ultimate shear capacity of LCBs with web openings, the same finite element model was used with varying web thickness (t_w) and web yield stress (f_{yw}) values. The clear web height to thickness ratio (d_1/t_w) was varied by simply changing the web thickness. Table 3 shows that the effect of web yield stress (f_{yw}) and clear height of web to web thickness ratio (d_1/t_w) on the ultimate shear capacity of LCBs with web openings is small.

AS/NZS 4600 [7] and AISI [6] design rules gave conservative predictions of the shear capacity of LCBs as they assumed simply supported conditions along the web to flange juncture in cold-formed steel beams and did not include post-buckling strength. Hence Keerthan and Mahendran [12] developed Equation 7 to predict the increased shear buckling coefficients of LCBs. Similar research findings were also observed by for LiteSteel beams in shear. When LCBs included web openings, their elastic buckling capacities were also enhanced by the presence of higher level of fixity along the web to flange juncture.

Keerthan and Mahendran [12] found that simply supported conditions along the edges of a web panel are sufficient to develop post-buckling strength due to the tension filed action for LiteSteel beams, in particular for those with slender webs. Finite element analyses and shear tests conducted in this study for LCBs with web openings also showed the existence of post-buckling strength despite the presence of web openings (see Figures 5a, 8a and 9). Figure 10 shows that the tension field action observed in the shear tests was predicted accurately by finite element analyses. It also shows the mechanics of post-buckling behaviour of LCBs with web openings in shear.

6. Comparison of Shear Capacities with Current Design Rules

In this section, the ultimate shear capacities obtained from tests and FEA are compared with the predictions from the currently available design equations based on suitable shear capacity reduction factors. For this purpose, the shear capacities from 32 tests conducted with straps and the corresponding FEA results were considered as shown in Table 4. The comparison of shear capacity reduction factors (q_s) from tests, FEA and various design equations shows that the shear capacities predicted by Shan et al.'s [4] design equations are very conservative for LCBs with web openings in most cases. AS/NZS 4600 [7] design equations are also conservative for LCB sections with small web openings, however, they are unconservative for LCB sections with large openings.

The ultimate shear capacities from FEA and tests are also compared in Table 4 with the predictions from Keerthan and Mahendran's [13] and McMahon et al.'s [14] design equations developed for LSBs. This comparison shows that the shear capacities predicted by Keerthan and Mahendran's [13] design equations are unconservative for LCBs with large web openings while McMahon et al.'s [14] design equations are unconservative in all the cases of LCBs with web openings. Since both these design equations were developed for LiteSteel beams (LSBs) with rectangular hollow flanges, new shear capacity reduction factor equations should be developed for the open lipped channel beams (LCBs) with web openings. Details of the proposed shear capacity reduction factor equations are given next. It must be noted that the FEA and experimental shear capacities of LCBs without web openings obtained from this study and reported in Table 5 were used as the reference values to determine the shear capacity reduction factor q_s in all cases. Comparison of shear capacities was undertaken to investigate the accuracy of only the shear capacity reduction factor equations.

AS/NZS 4600 [7] restricts the ratio of depth of web opening to clear height of web (d_{wh}/d_1) to a value of 0.7 while McMahon et al. [14] restricts it to a value between 0.4 and 0.8. Some shear tests conducted in this research exceeded the above limits (see Table 4) and hence their equations were not used in predicting the shear capacities in such cases.

Since the currently available shear capacity reduction factor equations are either conservative or unsafe, new equations are proposed to predict the shear capacity of LCBs with web openings based on FEA and experimental results. It is proposed that the shear capacity of

LCBs with web openings (V_{nl}) can be calculated using a reduction factor q_s applied to the shear capacity of LCBs without web openings (V_v). The use of a shear capacity reduction factor (q_s) to the shear capacities of LCBs without web openings is considered adequate as a simple design method. Section 2 presents the shear capacity equations for LCBs without web openings (V_V) as Eqs. (4) to (6). Equations 20 to 23 show the proposed design equations for the shear capacity of LCBs with web openings (V_{nl}).

$$V_{nl} = q_s V_v \qquad 0 < d_{wh} / d_1 \leq 0.85 \tag{20}$$

$$q_s = 1 - 0.6\left[\frac{d_{wh}}{d_1}\right] \qquad 0 < \frac{d_{wh}}{d_1} \leq 0.30 \tag{21}$$

$$q_s = 1.215 - 1.316\left[\frac{d_{wh}}{d_1}\right] \qquad 0.30 < \frac{d_{wh}}{d_1} \leq 0.70 \tag{22}$$

$$q_s = 0.732 - 0.625\left[\frac{d_{wh}}{d_1}\right] \qquad 0.70 < \frac{d_{wh}}{d_1} \leq 0.85 \tag{23}$$

q_s = shear capacity reduction factor = V_{nl}/V_v

d_{wh} = depth of web openings

d_1 = clear height of web

Figure 11 show the non-dimensional curve of q_s versus d_{wh}/d_1. In order to assess the accuracy of the proposed design equations for the shear capacity of LCBs with web openings (Eqs. 21 to 23), Figure 11 compares their predictions with the corresponding FEA and experimental shear capacity reduction factors reported in Table 5. In this figure the shear capacity reduction factors obtained from the parametric study (Table 3) are also included. It shows that the shear capacity reduction factors predicted by Equations 21 to 23 agree well with FEA and experimental shear capacity reduction factors. The overall mean value of FEA to predicted shear capacity reduction factor ratio is 1.01 while the corresponding coefficient of variation (COV) is 0.088. The mean value of test to predicted shear capacity reduction factor ratio is 1.03 while the corresponding coefficient of variation (COV) is 0.089.

Figures 12 and 13 show the shear capacity reduction factors q_s of 160x65x15x1.9 LCB, 200x75x15x1.9 LCB, respectively (aspect ratio = 1.0) as a function of the depth of web openings to clear height of web ratio (d_{wh}/d_1) in comparison to those proposed by other researchers in the past. These figures show that Shan et al.'s [4] design equations significantly underestimate the shear capacity of LCBs with web openings when d_{wh}/d_1 is greater than 0.2 while their shear capacity reduction factor (q_s) is equal to 1.0 (no reduction in shear capacity) when d_{wh}/d_1 is less than 0.2. Hence Shan et al.'s [4] design equations are not acceptable for the shear design of LCBs with web openings.

As shown in Figures 12 and 13, AS/NZS 4600 [7] design equations are conservative for LCB sections with small web openings while they are unconservative for LCB sections with large openings. However, the proposed design equations in this paper are able to accurately predict the FEA and experimental shear capacities of LCBs with web openings.

Keerthan and Mahendran's [13] developed Equations 16 to 18 for the shear capacity reduction factor of LSBs as a function of d_{wh}/d_1. These equations are similar to the proposed equations for LCBs in this paper. As seen in Figures 12 and 13, Keerthan and Mahendran's [13] equations were able to predict the shear capacity reduction factors for LCBs with d_{wh}/d_1 ratio less than 0.4. However, their equations overestimated the shear capacities of LCBs when d_{wh}/d_1 is greater than 0.4 as they were developed for LiteSteel beams (LSBs) with rectangular hollow flanges. Figures 12 and 13 also show that McMahon et al.'s [14] design equations are unconservative in all the cases of LCBs with web openings.

7. Vierendeel Mechanism of LSBs with Web Openings

In order to investigate the effect of vierendeel mechanism of LCBs with web openings, nonlinear finite element analyses were conducted for LCBs with large web openings. Finite element analysis results showed that when the depth of web opening, d_{wh}, was equal to the clear height of web (d_1), the failure was caused by the Vierendeel mechanism with hinges forming centrally at the top and bottom of the flanges. These observations are similar to Keerthan and Mahendran's [13] observations for LiteSteel beams and Narayanan's [18] test observations for thin web plate girders with perforations. Keerthan and Mahendran [13] found that LSBs with web openings have considerable amount of shear capacity (24 to 37%) due to the rigid hollow flange even when the depth of web opening, d_{wh}, was equal to the clear height of web (d_1). However, the shear capacity of LCBs with web openings was reduced to 9% when the depth of web opening, d_{wh}, was equal to the clear height of web (d_1). Figure 14 shows the vierendeel mechanism of 200x75x15x1.9 LCB (aspect ratio = 1.0).

8. Combined Shear and Flange Distortion Action of LCBs with Web Openings

In some applications in the building industry, LCBs are not used with straps at the supports. In order to investigate the effect of straps on the shear behaviour of LCBs with web openings, eight shear tests (Tests 33 to 40 in Table 1) and corresponding nonlinear finite element analyses were conducted for LCBs with openings. Both FEA and tests show that relatively short span LCBs without straps (aspect ratios of 1.0 and 1.5) are subjected to a relatively new combined shear and flange distortion action when the LCBs are not used with straps at the loading point and supports (flanges are not restrained). The flange distortion occurs due to the distortional buckling or unbalanced shear flow in these sections. Figure 15 shows the failure modes of LCBs with web openings when the flanges are not restrained (without straps). This figure clearly indicates that the combined shear and flange distortion deformation occurs due to the presence of distortional buckling or unbalanced shear flow.

Table 6 shows the shear capacities of LCBs with web openings when straps are not used and compares them with the corresponding shear capacities when straps are used. Both test and FEA shear capacities are presented in this table. A comparison of test and FEA shear capacities show that the developed finite element models were also able to predict the shear capacities for LCBs without flange straps. Table 6 results show that LCB sections without flange restraints had a lower shear capacity than their restrained equivalents. They show that there is about 3 to 11% reduction in the shear capacity of LCB with web openings when straps are not attached to its flanges. Therefore it is recommended that the proposed shear design equations (Equations 21 to 23) can also be used for LCBs without straps provided a reduction factor of 0.9 is used. The reduction factor (0.9) provides a lower bound and thus ensures a safe design of LCBs without straps. Combined shear and flange distortion is likely to occur before reaching its full shear capacity due to the distortional buckling or unbalanced shear flow present in LCBs with web openings.

9. Effect of Web Side Plate Height on the Shear Behaviour and Strength of LCBs

In some applications in the building industry, LCBs are not used with full depth web side plates (WSP) at the supports. In order to investigate the effect of web side plate height on the shear behaviour and strength of LCBs, the same finite element model described in the earlier sections was used. In the earlier finite element models used in this study, full height web side plates were used and modelled as rigid bodies using R3D4 elements and the motion was controlled by the motion of a single node, known as the rigid body reference node. In the current models, the height of web side plates was changed to investigate the effect of web side plate height on the shear behaviour and strength of LCBs. Straps were not used in these finite element models to investigate the lateral movement due to the shear flow action. Table 7 shows the ultimate shear capacities of 200x75x15x1.9 LCB from FEA where s/d_1 is the ratio of WSP height to clear web height. Finite element analysis results show that shear capacities of LCBs with web openings are reduced when full height WSPs are not used at the supports.

Finite element analyses showed the tendency of LCB flanges to displace laterally as seen in Figure 16 for 200x75x15x1.9 LCB with 60 mm web openings. At mid-span, the top flange of LCB tended to displace laterally while the bottom flange displaced towards the opposite side due to the shear flow action. This occurred when the full depth of LSB web element was not supported by the web side plate, ie. the WSP height (s) was less than the clear height of web (d_1). This led to reduced restraint to the lateral movement of flanges. It was found that the shear capacity of LCB decreases with decreasing height of web side plate (see Table 7). Figure 17 also shows the effect of varying web side plate height on the shear failure modes of 200x75x15x1.9 LCB with 100 mm web openings. Our proposed shear design formulae (Equations 21 to 23) are valid when the WSPs are used to the full height of the web element at the supports (ie. no lateral movements of top and bottom flanges). Detailed experimental and finite element studies will be conducted to investigate the effect of web side plate height on the shear behaviour and strength of LCBs.

10. Conclusions

This paper has presented a detailed investigation into the shear behaviour and strength of cold-formed steel lipped channel beams (LCB) with web openings using finite element analyses. Suitable finite element models were developed and validated by comparing their results with experimental test results. The developed nonlinear finite element model was able to predict the shear capacities of LCBs with web openings and associated deformations and failure modes with very good accuracy. Numerical and experimental studies show that AS/NZS 4600 [7] design equations are conservative for LCB sections with small web openings while they are unconservative for LCB sections with large openings. It was found that Shan et al.’s [4] design equations are too conservative for the shear capacity of LCBs with web openings. Appropriately improved design equations have been proposed in the form of modified shear capacity reduction factors to determine the shear capacity of LCBs with web openings based on both numerical and experimental results.

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