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[Poologanathan, Keerthan & Mahendran, Mahen](#) (2012) Shear behaviour and strength of LiteSteel Beams with web openings. *Advances in Structural Engineering*, 15(2), pp. 197-210.

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<http://dx.doi.org/10.1260/1369-4332.15.2.171>

Shear Behaviour and Strength of LiteSteel Beams with Web Openings

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Abstract: The LiteSteel Beam (LSB) is a new cold-formed steel hollow flange channel beam recently developed in Australia. It is commonly used as a floor joist or bearer in buildings. Current practice in flooring systems is to include openings in the web element of floor joists or bearers so that building services can be located within them. Shear behaviour of LSBs with web openings is more complicated while their shear strengths are considerably reduced by the presence of web openings. However, no research has been undertaken on the shear behaviour and strength of LSBs with web openings. Therefore a detailed experimental study involving 26 shear tests was undertaken on simply supported LSB test specimens with web openings and an aspect ratio of 1.5. This paper presents the details of this experimental study and the results of their shear capacities and behavioural characteristics. Experimental results showed that the current design rules in cold-formed steel structures design codes are very conservative for the shear design of LSBs with web openings. Improved design equations have been proposed for the shear strength of LSBs with web openings based on the experimental results from this study.

Keywords: *LiteSteel beam, Shear strength, Cold-formed steel structures, Web openings, Slender web and Hollow flanges.*

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1. Introduction

The use of cold-formed steel members in building construction has increased significantly in recent times. There are many benefits associated with the use of lightweight cold-formed steel sections in residential, industrial and commercial buildings. Thinner cold-formed steel sections with varying geometry are continuously developed to suit changing requirements including higher flexural capacities. LiteSteel Beam (LSB) is a new cold-formed steel hollow flange channel beam produced by OneSteel Australian Tube Mills (LST, 2009). It is manufactured from a single strip of high strength steel through the use of a combined cold-forming and dual electric resistance welding process. The effective distribution of steel in LSBs with two rectangular hollow flanges results in a thin and lightweight section with good flexural capacity (Figure 1(a)). The LSB has many applications but, in particular, has become a very popular choice in the flooring systems as shown in Figure 1(b) (LST, 2009). Table 1 provides the currently available LSB sections and their dimensions.

Current practice in flooring systems is to include openings in the web of floor joists or bearers so that building services can be located within them. Without web openings, services have to be located under the joists leading to increased floor height. This is not an effective use of space and an undesirable result for users. Pokharel and Mahendran (2006) recommended the use of circular web openings in LSBs based on an investigation using finite element analyses. Figure 2 shows an LSB joist with circular web openings. Three standard opening sizes of 60, 102 and 127 mm are used with currently available LSBs (OATM, 2008).

The use of web openings in a beam section significantly reduces its shear capacity due to the reduced web area. The shear flow in 125x45x2.0 LSB shown in Figure 3 demonstrates that approximately 88% of the shear force is supported by the main web element of LSB. The reduction in the primary shear resisting area will lead to a significant reduction in shear capacity of LSBs. However, the effect of web openings on the flexural capacity is negligible as the web openings are normally located at the centre of web.

There are many variables that affect the shear capacity of members containing web openings. They include the shape, size and location of 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 LSB sections using a detailed experimental study.

This paper presents the details of a series of shear tests of LSBs with circular web openings, and the results. Experimental shear capacities are compared with the predicted shear capacities using the current design rules in Australian/New Zealand Standards AS/NZS 4600 (SA, 2005) and the North American Specification (AISI, 2007). This paper also includes a brief review into the shear capacities of steel beams with web openings, and available shear design rules.

2. Review of Shear Capacities of Steel Beams with Web Openings

2.1. Overview

Research into the effects of web openings on the shear capacity of steel beams was conducted first in the early 1960s. However, most of the researches undertaken before the 1970s concentrated primarily on the effect of openings in moderately thick elements (Hoglund, 1971). The investigation of cold-formed steel sections containing web openings was undertaken in the last decade (Baranda et al., 1978). However, only limited research has been undertaken on the effect of web openings in relation to LSBs. Design capacity tables produced by OATM do not consider the shear capacity of LSBs containing web openings (OATM, 2008).

Hoglund (1971) researched the effects of rectangular and circular holes on the shear performance of plate girders with thin webs subject to static loading. He found that when holes were placed in the areas of high shear force, the reduction in shear capacity was high. Baranda et al., (1978) stated that the flexural capacity of sections with web openings was reduced by approximately 2 to 5%. The minimal reduction in flexural capacity is a result of the section mid-depth being a region of reduced bending stresses. Redwood and Shrivastava (1980) stated that openings less than 30% of the section height will not cause a significant reduction to the flexural capacity.

Narayanan (1982) tested 20 thin web plate girders containing centrally located perforations, which were subject to shear loading. It was found that their shear capacity was reduced linearly with increasing depth of web opening. If the depth of web opening covers the full web depth, then the failure is caused by the Vierendeel mechanism with hinges formed centrally at the top and bottom of the flanges.

Shan et al. (1997) conducted detailed experimental studies into the shear capacity of thin cold-formed steel sections. They recommended that the nominal shear capacity of cold-formed lipped channel beams with web openings can be calculated using a reduction factor applied to the solid web strength of the shear element. Eiler (1997) extended Shan et al.'s (1997) work to include the behaviour of web elements with openings subjected to linearly varying shear force. Based on 85 tests of beam type members, Eiler (1997) proposed suitable design equations for the shear strength of cold-formed steel beams with web openings, which are given in the following sections.

McMahon et al. (2008) conducted ten shear tests of back to back LSB sections with web openings using a three-point loading method. Their tests included three different LSB sections with three circular web opening sizes of 60, 102 and 127 mm. Details of their test specimens are given in Table 2. All three sections were also tested without web openings for calibration and comparative purposes. In order to reduce the effect of bending moment on the shear capacity, extra plates were welded to the top and bottom flanges of the beams at the centre to increase the bending capacity and thus avoid the bending failure at the centre. Table 2 shows the induced shear force and moment by the ultimate load (McMahon et al., 2008). As recommended by Shan et al. (1997), McMahon et al. (2008) also developed suitable equations for the shear capacity reduction factor in terms of the depth of web openings (d_{wh}), which are given in the following sections.

Chung et al. (2001) investigated the Vierendeel action associated with web openings. Vierendeel action is a result of four plastic hinges forming at critical locations around the web openings. This formation results in a Vierendeel mechanism as the shear forces are transferred laterally across the openings. This, therefore, demonstrates that the effect of Vierendeel forces is a function of the length of openings. Chung et al. (2001) stated that when a large web opening is present the shear capacity is significantly reduced and consequently the flange shear capacity becomes important.

Hagen et al. (2009) showed that the reduction in shear capacity due to web openings could depend on a range of parameters relating to the removal of material from the web. The primary parameters that govern the shear capacity are the web slenderness (d_1/t_w), the opening size (d_{wh}/d_1), the yield stress of steel f_{yw} (specifically the ratio $\sqrt{f_{yw}/E}$) and the spacing between transverse web stiffeners. The secondary parameters refer to the shape, location and

spacing of openings. The tertiary parameters are related to the flange cross sectional area, bending and torsional stiffness.

A significant effect of web openings is web crippling under loading. This research is not concerned with web crippling and therefore experiments were conducted in such a manner to prevent web crippling. A practical solution is to stiffen the web opening. This can be achieved by providing steel area to the web which will increase the shear capacity. However, web stiffening often can not be a practical solution due to the associated high cost.

2.2. Shear Capacity Equations for LSBs without Web Openings

Keerthan and Mahendran (2010) proposed new shear strength formulae (Eqs. 1 to 6) for LSBs based on the current design equations for shear strength given in AISI (2007) using finite element analysis and experimental results. The increased shear buckling coefficient given by Equation 7 (k_{LSB}) is included to allow for the additional fixity in the web-flange juncture. Equations 1 to 3 present the proposed shear strength (τ_v) equations which also include the post-buckling strength present in LSBs. The shear capacity in kN can be obtained by multiplying the shear strength by its web area of $d_1 t_w$.

$$\tau_v = \tau_{yw} \quad \text{for} \quad \frac{d_1}{t_w} \leq \sqrt{\frac{Ek_{LSB}}{f_{yw}}} \quad (\text{Shear yielding}) \quad (1)$$

$$\tau_v = \tau_i + 0.2(\tau_{yw} - \tau_i) \quad \text{for} \quad \sqrt{\frac{Ek_{LSB}}{f_{yw}}} < \frac{d_1}{t_w} \leq 1.508 \sqrt{\frac{Ek_{LSB}}{f_{yw}}} \quad (\text{Inelastic shear buckling}) \quad (2)$$

$$\tau_v = \tau_e + 0.2(\tau_{yw} - \tau_e) \quad \text{for} \quad \frac{d_1}{t_w} > 1.508 \sqrt{\frac{Ek_{LSB}}{f_{yw}}} \quad (\text{Elastic shear buckling}) \quad (3)$$

where

$$\tau_{yw} = 0.6 f_{yw} \quad (4)$$

$$\tau_i = \frac{0.6 \sqrt{Ek_{LSB} f_{yw}}}{\left[\frac{d_1}{t_w} \right]} \quad (5)$$

$$\tau_e = \frac{0.905Ek_{LSB}}{\left[\frac{d_1}{t_w}\right]^2} \quad (6)$$

$$\text{For LSBs } k_{LSB} = k_{ss} + 0.87(k_{sf} - k_{ss}) \quad (7)$$

where k_{ss} , k_{sf} = shear buckling coefficients of plates with all four edges simply supported, and two opposite edges simply supported and fixed, respectively.

The direct strength method provides simple design procedures for cold-formed steel members. Proposed design equations (1 to 3) are also therefore recast in the new direct strength method format and are given as Equations 8 to 10 (AISI, 2006).

$$\frac{\tau_v}{\tau_{yw}} = 1 \quad \lambda \leq 0.815 \quad (8)$$

$$\frac{\tau_v}{\tau_{yw}} = \frac{0.815}{\lambda} + 0.2\left(1 - \frac{0.815}{\lambda}\right) \quad 0.815 < \lambda \leq 1.23 \quad (9)$$

$$\frac{\tau_v}{\tau_{yw}} = \frac{1}{\lambda^2} + 0.2\left(1 - \frac{1}{\lambda^2}\right) \quad \lambda > 1.23 \quad (10)$$

where

$$\tau_{yw} = 0.6f_{yw} \quad (11)$$

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

$$\lambda = \sqrt{\left(\frac{\tau_{yw}}{\tau_{cr}}\right)} = 0.815 \left(\frac{d_1}{t_w}\right) \sqrt{\left(\frac{f_{yw}}{Ek_{LSB}}\right)} \quad (13)$$

A capacity reduction factor of 0.95 is recommended in the calculation of design shear capacities of LSBs without web openings from the above equations (AISI, 2007).

2.3. Shear Capacity Reduction Factor for Cold-formed Steel Members with Web Openings

Shan et al. (1997) recommended that the nominal shear capacity of cold-formed lipped channel beams with web openings (V_{nl}) can be calculated using a reduction factor q_s applied to the solid web strength of the shear element (V_v) (Eq. 14). The nominal shear capacity of lipped channel beams with web openings is defined as follows. Equations 15 and 16 are based on a linear relationship whereas Equation 17 is a non-linear strength reduction relationship.

$$V_{nl} = q_s V_v \quad (14)$$

$$q_s = -3.66 \left(\frac{d_{wh}}{d_1} \right) + 1.71 \quad \text{for } \frac{d_{wh}}{d_1} \leq 0.38 \quad (15)$$

$$q_s = -0.38 \left(\frac{d_{wh}}{d_1} \right) + 0.46 \quad \text{for } 0.38 < \frac{d_{wh}}{d_1} \leq 1.0 \quad (16)$$

$$q_s = 1.51 \times 10^{[-1.33(d_{wh}/d_1)]} \quad \text{for } 0 \leq \frac{d_{wh}}{d_1} \leq 1.0 \text{ and } q_s \leq 1 \quad (17)$$

where d_{wh} is the depth of web openings, d_1 is the depth of the flat portion of web and V_v is the nominal shear strength of the solid web element. Their research included non-circular web openings and the developed equations are considered applicable to both circular and non-circular web openings.

McMahon et al. (2008) recommended that the shear capacity of LSBs with web openings can be calculated using a reduction factor (q_s) applied to the solid web strength of the shear element. The reduction factor is defined as follows based on a simple linear relationship.

$$q_s = 1 - \frac{d_{wh}}{2d_1} \quad \text{for } 0.4 \leq \frac{d_{wh}}{d_1} \leq 0.8 \quad (18)$$

where

d_1 = depth of the flat portion of web

The shear capacity reduction factor is a function of many parameters. It includes the type of applied stress, the shape, size, location and the number of openings. Using the relevant parameters in a dimensionless format, a general method is recommended in AS/NZS 4600

(SA, 2005). The main parameter is the ratio of the depth of web opening to the clear height of web, d_{wh} / d_1 .

AS/NZS 4600 (SA, 2005) and NAS (AISI, 2007) require that there is a minimum clear distance of 450 mm between web openings. The ratio of the depth of web opening to the thickness must also be less than or equal to 200, i.e. $d_{wh} / t_w \leq 200$. AS/NZS 4600 restricts the ratio of depth of web openings to flat portion of clear height of web (d_{wh}/d_1) to a value of 0.7. The web openings are considered to be centred at mid-depth of the web. Circular hole diameters should be less than or equal to 150 mm. AS/NZS 4600 and NAS design equations for the shear capacity reduction factor (q_s) are given next. Unlike in Shan et al. (1997), different equations are given for circular and non-circular web openings.

$$q_s = 1 \quad \frac{c}{t} > 54 \quad (19)$$

$$q_s = \frac{c}{54t} \quad 5 \leq \frac{c}{t} < 54 \quad (20)$$

$$c = \frac{d_1}{2} - \frac{d_{wh}}{2.83} \quad \text{for circular web openings} \quad (21)$$

$$c = \frac{d_1}{2} - \frac{d_{wh}}{2} \quad \text{for non-circular web openings} \quad (22)$$

$$\frac{d_{wh}}{d_1} < 0.7 \quad \frac{d_{wh}}{t_w} \leq 200 \quad d_o > 15 \text{ mm}$$

where

d_o = outside diameter of circular web openings

d_{wh} = depth of web openings

d_1 = depth of the flat portion of web

2.4. Guidelines for LSBs with Web Openings

OneSteel Australian Tube Mills (2008) provides the maximum allowable dimensions and minimum spacing requirements for the web openings in LSB floor systems. They are used for the web openings greater than 15 mm in diameter, where services are required to pass through, and must be centred at the mid-height of web. Table 3 gives the allowable dimensions of these parameters.

3. Shear Tests of LSBs with Web Openings

It is vital that key parameters are chosen carefully in the design of a test program. In order to fully understand the shear behaviour of LSB sections with web openings several important issues were considered when deciding these parameters such as the ratios of the depth of web openings to clear height of web (d_{wh}/d_1) and the depth of web openings to thickness of web (d_{wh}/t_w). Test specimens of LSBs were designed to fail in shear prior to reaching other section capacities.

3.1. Test Specimens and Test Set-up

Experimental studies were carried out to investigate the shear behaviour of LSBs with web openings using a series of primarily shear tests of simply supported LSBs subjected to a mid-span load (see Figure 4). Two LSB sections were bolted back to back using three T-shaped stiffeners and three 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 LSB sections (see Figure 4) to allow the test beams to behave independently while remaining together to resist torsional effects. In order to simulate a primarily shear condition, relatively short test beams were selected based on an aspect ratio (shear span a / clear web height d_1) of 1.5. Three opening sizes of 60, 102 and 127 mm were chosen based on the standard sizes given in OATM (2008) for seven of LSB sections, giving a total of 26 shear tests. Table 4 presents the details of the shear test specimens. It includes the measured thicknesses and yield stresses of the web elements of tested LSBs.

For LSBs d_1 is defined as the clear height of web instead of the depth of the flat portion of web measured along the plane of the web as defined in AS/NZS 4600 for cold-formed channel sections. The reasons for this are as follows.

- LSB has two rectangular hollow flanges, which are likely to increase the shear yield capacity by framing action.
- Outside of the corners are filled with weld material unlike in cold-formed channel sections.
- Buckling occurs within the clear height of web.

- Although the use of d_1 as the depth of the flat portion of web is conservative in estimating the shear yield capacity, it is not safe in the case of elastic buckling.
- Shear behaviour of LSBs is very similar to that of plate girders (web-flange boundary condition and welding process), for which, clear web height is used in shear capacity calculations.

The LSB sections were loaded through the central T-shaped stiffener that was attached to the back to back test beam and the two web side plates with 4 M16 bolts at the mid-span loading point. These T-shaped stiffeners were important as they avoided bearing failure of the flanges. This method of loading has the added advantage of loading through the shear centre thus avoiding eccentric loading and web crippling. Similar T-shaped stiffeners were also located at the supports, and were bolted to the two LSBs and the two web side plates on either side. Figure 4 shows the experimental set-up used in the shear tests of this research.

The support system was designed to ensure that the test beam acted as a simply supported beam with pinned supports at each end. The test beam was supported on round sections using ball bearings. All contact surfaces within the system were machine ground and polished to a very smooth surface. These surfaces were lubricated to aid in the sliding of the support as shown in Figure 4. The ends of the test beams were free to rotate and it was therefore seen that simply supported conditions were simulated accurately at the end supports.

The applied load at the mid-span of the test beam is the important parameter. The measuring system was set-up to record the applied load and associated test beam displacements. Two displacement transducers were located on the test beam under the loading points (see Figure 4).

As mentioned earlier, relatively short test beams of span based on an aspect ratio of 1.5 was selected. It is practically impossible to set up a loading scheme in shear tests where a test panel is subjected to pure shear. A bending moment will be present too. But it can be assumed that the shear capacity is not affected provided the ratio of applied moment M^* to the section moment capacity M_s is less than 0.70. This is based on the design rules of AS 4100 for combined shear and bending (SA, 1998). This requirement was met by all the test specimens chosen in this study (Table 4) and hence it is considered that the shear capacities from these tests will be accurate.

The expected shear capacities calculated from AS/NZS 4600 are given in Table 5 for each test beam. For the thicker LSB sections, 300x75x2.5 LSB and 300x75x3.0 LSB, the predicted shear capacities are more than 75 kN. In order to test them to failure, a testing machine with a capacity of 600 kN (four times the shear capacity) is needed. Since the available testing machine's capacity is 300 kN, these thicker LSB sections were tested using a simplified experimental arrangement based on single LSB (see Keerthan and Mahendran, 2010). In this arrangement using the same three-point loading method, single LSBs were loaded as close as possible to the shear centre at mid-span to eliminate any torsional effects. The LSB test specimen was also laterally restrained at the supports and the loading point. As used with the back to back LSB shear tests, T-shaped stiffeners were used at the supports and the loading point to avoid web crippling and flange bearing failures. They also provided lateral restraint to the specimens and allowed the load application to be closer to the shear centre.

3.2. Test Procedure

Two LSBs with web openings were cut to the required length, and their sizes, in particular, the clear web height (d_1) and web thickness (t_w) were measured (Table 4). Test specimens were cut 50 mm longer than their required span in order to allow 25 mm overhang at either end of the test beam. Holes were then inserted at the loading and support positions to allow for the effective connections at these points.

Shear span (a) was taken as the distance between the centre of inner bolts on the web side plates and the required test span was calculated based on the aspect ratios (see Figure 4 (a)). For example, in the case of 150x45x2.0 LSB with $d_1 = 120$ mm, shear span was 180 mm corresponding to an aspect ratio of 1.5. Hence the specimen length was 545 mm based on the spacing of bolts in the web side plates of 45 mm and the edge distance of outer bolts of 25 mm. The two LSBs were then assembled as back to back LSBs. The assembled pair of LSB sections with web openings was positioned accurately in the test rig to ensure that the three point loading method was achieved.

Two displacement transducers were positioned and connected to the data acquisition system. Each channel was individually checked to ensure correct operation. A small load was applied first to allow the loading and support systems to settle on bearings evenly. The measuring

system was then initialised with zero values and the loading was commenced. The cross-head of the testing machine was moved at a constant rate of 1 mm/minute until the test beam failed.

3.3. Test Results

The purpose of conducting full-scale tests is to experimentally establish the shear capacities of LSB sections with web openings. These experimental results are important as they provide a point of comparison with which to gauge the performance of the shear design rules as well as presenting some data with which to verify finite element models of LSB sections with web openings. As seen from Figure 4, the shear force induced in each LSB section is equal to the applied load (P) divided by 4 in the back to back LSB test arrangement. It is $P/2$ in the case of simplified test arrangement based on single LSBs.

For the purpose of planning and designing the test arrangement, the shear capacities of test specimens with web openings were first predicted using AS/NZS 4600 (SA, 2005) and NAS (AISI, 2007). In these calculations the measured specimen sizes (clear height d_1 and web thickness t_w) were used. Table 5 presents the predicted shear capacities of single LSB sections with web openings.

Figure 5 shows the load-deflection curve for the shear test of 200x60x2.0 LSB section with 60 mm web openings. At Point 1, the web began to distort out of plane and the beam reached the ultimate shear capacity of 58.3 kN (applied load of 233.2 kN/4) at Point 2. Similarly, Figure 6 shows the load - deflection curve for the shear test of 200x45x1.6 LSB section with 102 mm web openings. At Point 1, the beam started to show some signs of lateral deflection within the web, indicating the beginning of buckling and the beam reached the ultimate shear capacity of 29.1 kN (applied load of 116.4 kN/4) at Point 2. Figures 7 and 8 show the failure modes of 150x45x2.0 LSB and 200x45x1.6 LSB with web openings, respectively.

3.4. Shear Behaviour of LSBs with Web Openings

Keerthan and Mahendran's (2010) 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.

Keerthan and Mahendran's (2010) shear test results also showed that the elastic buckling capacity of LSBs without web openings was considerably increased due to the presence of increased fixity along the web to flange juncture. They showed that the assumption of simply supported conditions along the web to flange juncture in AS/NZS 4600 and NAS design rules gave conservative predictions. Equation 7 was therefore developed to predict the increased shear buckling coefficients of LSBs. Similar research outcomes were also produced by Lee et al. (1995) for steel plate girders in shear. When LSBs included web openings, their elastic buckling capacity was again enhanced by the presence of higher level of fixity along the web to flange juncture. This was evident from the shear test results in this study. For example, the elastic shear buckling load of 200x45x1.6 LSB with 102 mm web openings can be estimated as 22.75 kN at Point 1 (applied load of 91 kN) from Figure 6 in comparison to the predicted buckling load of 17.9 kN based on simply supported web-flange juncture (Keerthan and Mahendran, 2009).

Figure 6 also shows that the test beam reached the ultimate shear capacity of 29.1 kN (applied load of 116.4 kN/4) at Point 2. This confirms that LSBs with web openings also have post-buckling strength due to the tension field action. Yoo et al. (2006) showed that simply supported conditions along the edges of a rectangular panel are sufficient to develop a practically meaningful post-buckling strength due to the tension field action for plate girders. The same observation was made in Keerthan and Mahendran's (2010) experimental studies for LSBs without web openings. They showed that considerable post-buckling strength was present for LSBs due to the tension field action, in particular for LSBs with slender webs. Shear tests conducted in this study for LSBs with web openings have also shown the existence of post-buckling strength despite the presence of web openings.

Shanmugam et al. (2002) investigated the shear behaviour of plate girders with web openings using finite element analyses and showed the presence of post-buckling strength due to tension field action. In addition to the observations made in the shear tests, non-linear finite element analyses of Keerthan and Mahendran (2009) also showed that slender LSBs with web openings do not collapse when elastic buckling stress is reached, but has considerable post-buckling strength. Figure 9 shows that the tension field action observed in the shear tests was

predicted accurately by finite element analyses. It was found that the diagonal tension field shown in Figure 9 developed after the elastic buckling of LSBs. This observation is similar to Shanmugam et al.'s (2002) findings. Figure 9 also shows the mechanics of post-buckling behaviour of LSBs with web openings in shear and confirmed that simply supported conditions are sufficient to develop post-buckling strength of LSBs with web openings due to the tension field action. Keerthan and Mahendran (2009) found that significant reserve strength beyond elastic buckling is present for slender LSBs with web openings. Therefore post-buckling shear strength can be taken into account in the design of slender LSBs with web openings.

When the depth of web openings (d_{wh}) is increased considerably (ie. d_{wh} approaches clear web height d_1) the LSBs are expected to retain considerable amount of shear capacity (24 to 37%). Since the LSBs have rigid hollow flanges, this becomes possible due to the Vierendeel action. Although shear tests did not demonstrate this clearly, finite element analyses of Keerthan and Mahendran (2009) were able to show this. These observations are similar to Narayanan's (1982) test observations for thin web plate girders with perforations.

3.5. Comparison of Shear Capacities with Current Design Rules

Experimental ultimate shear capacities are given in Table 5 for the 26 shear tests conducted in this study, and compared with the predictions from the design equations in AS/NZS 4600 (SA, 2005) and Shan et al. (1997) based on suitable shear capacity reduction factors. The purpose of this comparison is to investigate the accuracy of the available design equations for the shear capacity reduction factors. Hence the experimental shear capacity of LSB without web openings was used as the reference capacity in all the cases. The comparisons in Table 5 show that the shear capacities predicted by the current design rules in AS/NZS 4600 and Shan et al.'s (1997) design equations are very conservative for LSBs with web openings in most cases.

The ultimate shear capacities from tests are also compared with the predictions from McMahan et al.'s (2008) design equations in Table 5. It shows that the shear capacities predicted by McMahan et al.'s (2008) design equations are unconservative for LSBs with web openings. McMahan et al. (2008) derived their design equations based on experimental shear capacities obtained from shear tests of relatively longer beams (1.4 m). A higher bending

moment was present in their test specimens in comparison to that used in this study. This appeared to have caused about 30% reduction in the shear capacity of LSB without web openings reported in McMahon et al. (2008) and hence resulted in the unconservative nature of their design equations.

It must be noted that experimental shear capacities of LSBs without web openings was used as the reference value in all cases. Comparison of shear capacities was undertaken to investigate the accuracy of only the shear capacity reduction factor equations.

It can be seen that the 250x75x2.5 LSB section with 102 mm web openings could not be tested to failure using the Tinius Olsen testing machine (Capacity of 300 kN). The 300x75x2.5 LSB with 127 mm web openings also posed similar problems. This is why a simplified experimental arrangement based on single LSB sections with web openings was used in some tests as mentioned in Table 4.

AS/NZS 4600 (SA, 2005) restricts the depth of web opening to clear height of web ratio (d_{wh}/d_1) to a value of 0.7 while McMahon et al. (2008) restricts it to a value between 0.4 and 0.8. Some shear tests carried out in this research exceeded the above limits of AS/NZS 4600 or McMahon et al.'s (2008) design equations (see Table 5) and hence their equations were not used in predicting the shear capacities in such cases.

4. Proposed Equations for the Shear Capacity of LSBs with Web Openings

Since the currently available shear capacity equations are either conservative or unsafe, new equations are proposed for the shear capacity of LSBs with web openings based on experimental results. It is proposed that the shear capacity of LSB with web openings (V_{nl}) can be calculated using a reduction factor q_s applied to the shear capacity of LSBs without web openings (V_v). This approach is similar to that used in the current cold-formed steel design codes (SA, 2005, AISI, 2007) and by other researchers (Shan et al., 1997, McMahon et al., 2008). As described in the previous section, the shear behavioural characteristics of LSBs with and without web openings are similar in terms of their elastic shear buckling and post-buckling behaviour and capacities. As for LSBs without web openings, the elastic shear buckling behaviour of LSBs with web openings was enhanced by the increased web-flange

fixity while they also developed post-buckling strength due to tension field action. However, the presence of web openings lead to reduced shear capacities for LSBs. Therefore the use of a shear capacity reduction factor (q_s) to the shear capacities of LSBs without web openings is considered adequate as a simple design method.

Extensive experimental and finite element studies were used to develop accurate shear capacity equations for LSBs without web openings (V_v), which are presented as Eqs.1 to 13 in Section 2. Equations 23 and 24 show the proposed design equations for the shear capacity of LSBs with web openings (V_{nl}).

$$V_{nl} = q_s V_v \quad \text{for } 0.24 < \frac{d_{wh}}{d_1} < 0.85 \quad (23)$$

$$q_s = 1 - \frac{d_{wh}}{1.45d_1} \quad (24)$$

where

d_{wh} = depth of web openings

d_1 = clear height of web

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

Table 6 shows the shear capacity reduction factor ($q_s = V_{nl}/V_v$) and the depth of web openings to clear height of web ratio (d_{wh}/d_1) while Figure 10 shows 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 LSBs with web openings (Eq. 24), their predictions are compared with the experimental shear capacity reduction factors in Table 6. It shows that the shear capacity reduction factors predicted by Equation 24 agree well with the experimental shear capacity reduction factors. The mean value of test to predicted shear capacity reduction factor ratio is 1.00 while the corresponding coefficient of variation (COV) is 0.073. However, it was found that the proposed design equation (Equation 24) is slightly unconservative when the LSBs have large web openings (see Figure 10 and Table 6). Further research using detailed finite element analyses is needed to improve the proposed design equations for the shear capacity of LSBs with web openings in this region.

Figure 11 shows the shear capacities of 200x60x2.0 LSBs as a function of depth of web opening. It shows that McMahon et al.'s (2008) design equation is unconservative for the shear capacity of LSBs with web openings while those in AS/NZS 4600 and Shan et al. (1997) are very conservative for the shear capacity of LSBs with web openings. However, the proposed design equation was able to predict the experimental shear capacities.

5. Conclusions

This paper has presented the details of an experimental investigation into the shear behaviour of a new cold-formed hollow flange channel beam known as LiteSteel Beams with web openings. Twenty six shear tests were undertaken using a three point loading arrangement. Comparison of ultimate shear capacities from tests showed that AS/NZS 4600 (SA, 2005) and NAS (AISI, 2007) design equations are conservative for the shear design of LSBs with web openings. It was found that McMahon et al.'s (2008) design equation is unconservative while Shan et al.'s (1997) design equations are too conservative for the shear capacity of LSBs with web openings. Appropriate improvements have been proposed in the form of modified shear capacity reduction factors to determine the shear capacity of LSBs with web openings based on experimental results.

Acknowledgements

The authors would like to thank Australian Research Council and OneSteel Australian Tube Mills for their financial support, and the Queensland University of Technology for providing the necessary facilities and support to conduct this research project. They would also like to thank Mr Ross Dempsey, Manager - Research and Testing, OneSteel Australian Tube Mills for his technical contributions, and his overall support to the many different phases of this research project.

References

- American Iron and Steel Institute (AISI) (2006). *Specification for the Design of Cold-formed Steel Structural Members*, AISI, Washington, DC, USA.
- American Iron and Steel Institute (AISI) (2007). *North American Specification (NAS) for the Design of Cold-formed Steel Structural Members*, AISI, Washington, DC, USA.

- Baranda, H., Daly, M. J. and Redwood, R. G. (1978). "Tests of Thin-webbed Beams with Unreinforced Holes", *ASCE Journal of the Structural Division*, Vol. 104, pp. 577 – 595.
- Chung, K.F., Ko, A.C.H. and Liu, T.C.H. (2001) "Investigation on Vierendeel Mechanism in Steel Beams with Circular Web Openings", *Journal of Constructional Steel Research*, Vol. 57, pp. 467 – 490.
- Eiler, M.R. (1997). *Behaviour of Web Elements with Openings Subjected to Linearly Varying Shear*, Masters Thesis, University of Missouri-Rolla, Missouri-Rolla, USA.
- Hagen, N.C., Larsen P.K. and Aalberg, A. (2009). "Shear Capacity of Steel Plate Girders with Large Web Openings Part I: Modelling and Simulations", *Journal of Constructional Steel Research*, Vol. 65, pp. 65142–150.
- Hoglund, T. (1971). "Shear Buckling of Steel and Aluminium Plate Girders", *Journal of Structural Engineering*, The Royal Institute of Technology, Stockholm, Sweden.
- Keerthan, P. and Mahendran, M. (2009). *Nonlinear Finite Element Analyses of the Shear Behaviour of LiteSteel Beams with Web Openings*, Research Report, Faculty of Built Environment and Engineering, Queensland University of Technology, Brisbane, Australia.
- Keerthan, P. and Mahendran, M. (2010). "Experimental Studies on the Shear Behaviour and Strength of LiteSteel Beams", *Engineering Structures*, Vol 32, pp. 3235-3247.
- Lee, S.C., Davidson, J.S. and Yoo, C. (1995). "Shear Buckling Coefficients of Plate Girder Web Panels", *Computers and Structures*, Vol. 59, pp. 789-795.
- LiteSteel Technologies (LST), *LiteSteel Beam Products*, Brisbane, Queensland, Viewed 10 March, 2009, <www.litesteelbeam.com.au>.

- McMahon, B., Peach, T.L., Spear, N. and Wilkinson, T. (2008). “Shear Capacity of LiteSteel Beams with Circular Web Openings”, *Proc. of the Fifth International Conference on Thin-Walled Structures*, Brisbane, Australia, pp. 419-424.
- Narayanan, R. (1982). “Shear Capacity of Thin web Plate Girders with Perforations”, *Proc. of the International Bridge Conference*, Pennsylvania, USA pp. 445–462.
- OneSteel Australian Tube Mills, (OATM) (2008). *Design of LiteSteel Beams*, Brisbane, Australia.
- Pokharel, N. and Mahendran, M. (2006). *Preliminary Investigation into the Structural Behaviour of LSB Floor Joists Containing Web Openings*, Research Report, Queensland University of Technology, Brisbane, Australia.
- Redwood, R.G. and Shrivastava, S.C. (1980). “Design Recommendations for Steel Beams with Web Holes”, *Canadian Journal of Civil Engineering*, Vol. 7, pp. 642 – 650.
- Shan, M.Y., LaBoube, R.A., Langan, J.E. and Yu, W.W. (1997). “Cold-formed Steel Webs with Openings: Summary report”, *Thin-Walled Structures*, Vol. 27, pp. 79 – 84.
- Shanmugam, N.E, Lian V.T. and Thevendran, V. (2002). “Finite Element Modelling of Plate Girders with Web Openings”, *Thin-Walled Structures*, Vol. 40, pp. 443–464.
- Standards Australia/Standards New Zealand (SA) (1998). *Australian Standard AS 4100 Steel Structures*, Sydney, Australia.
- Standards Australia/Standards New Zealand (SA) (2005). *Australia/New Zealand Standard AS/NZS4600 Cold-Formed Steel Structures*, Sydney, Australia.
- Yoo, C.H., Lee S.C. and M. (2006). “Mechanics of Web Panel Post-buckling Behaviour in Shear”, *ASCE Journal of Structural Engineering*, Vol. 132, pp. 1580-1589.



(a) LSB Section



(b) LSB Floor Systems

Figure 1: LiteSteel Beams

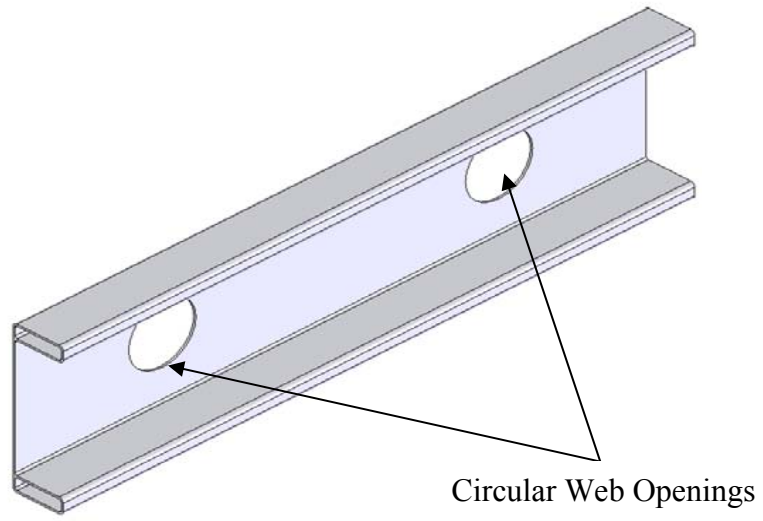


Figure 2: LiteSteel Beams with Circular Web Openings for Services

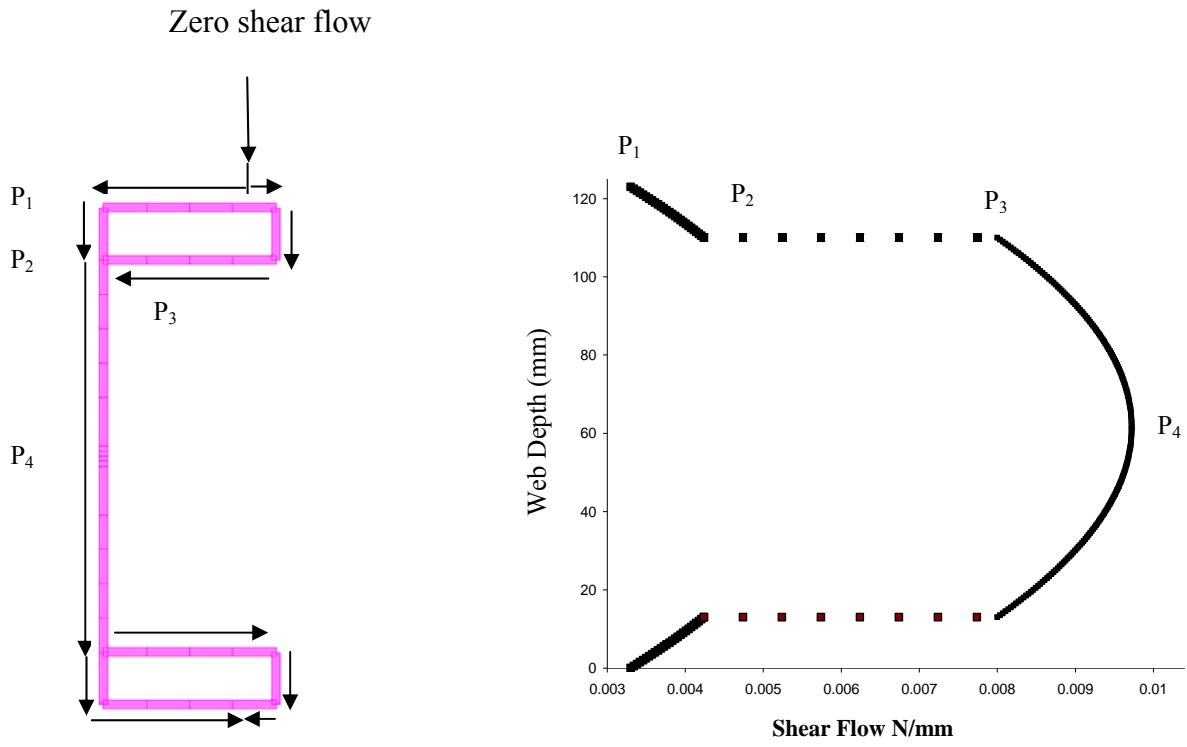


Figure 3: Shear Flow along the Web Depth in 125x45x2.0 LSB

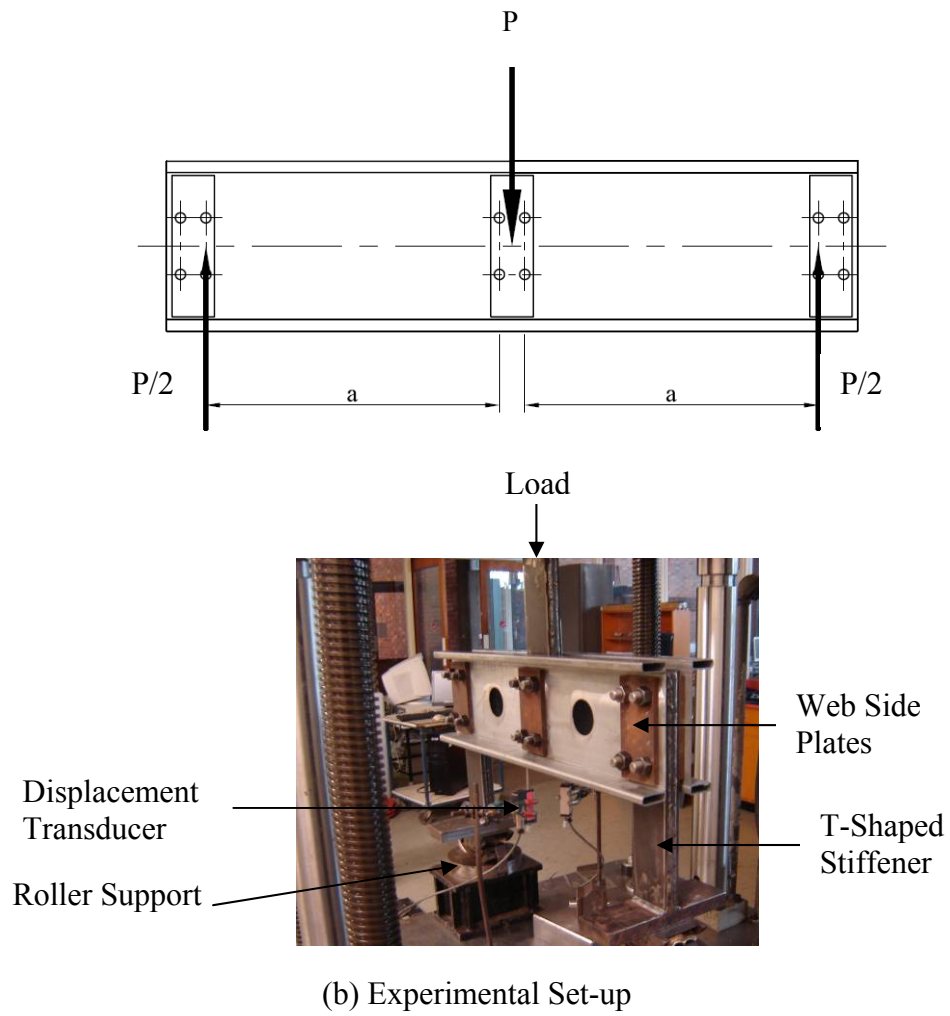


Figure 4: Experimental Set-up of Back to Back LSBs with Web Openings

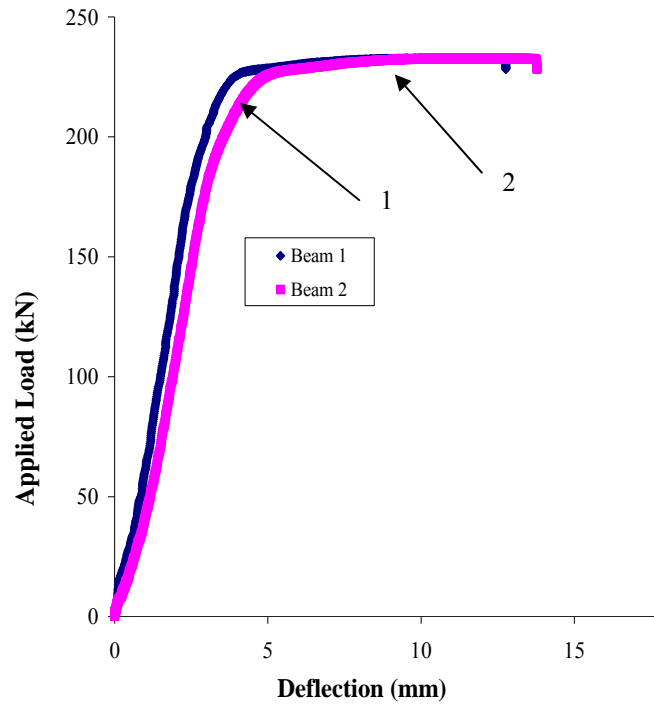


Figure 5: Applied Mid-span Load versus Mid-span Deflection for 200x60x2.0 LSB with 60 mm Web Openings (Aspect Ratio = 1.5)

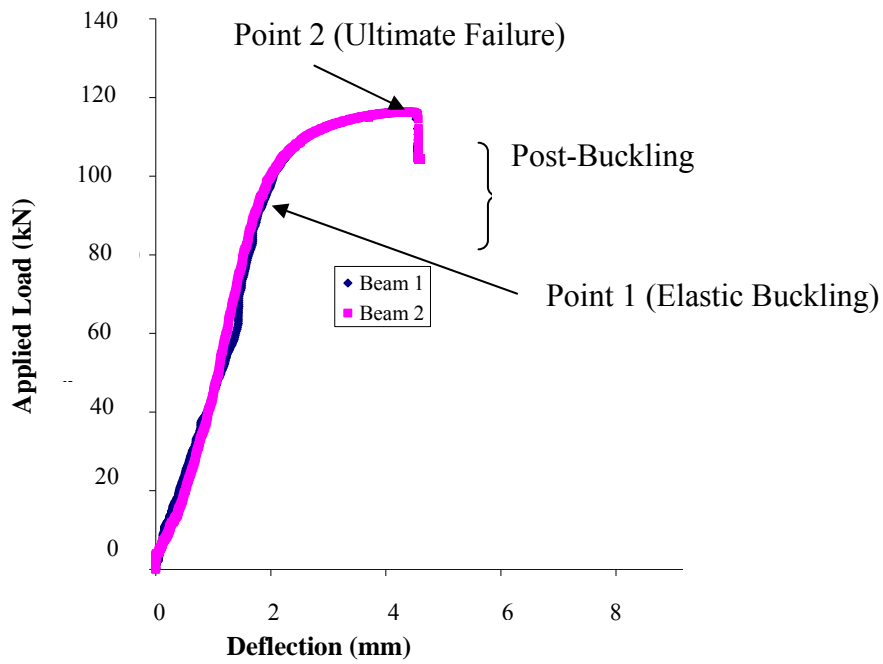
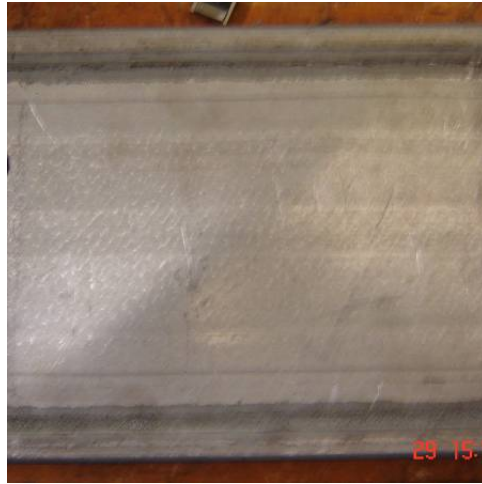


Figure 6: Applied Mid-span Load versus Mid-span Deflection for 200x45x1.6 LSB with 102 mm Web Openings



(a) No Web Openings

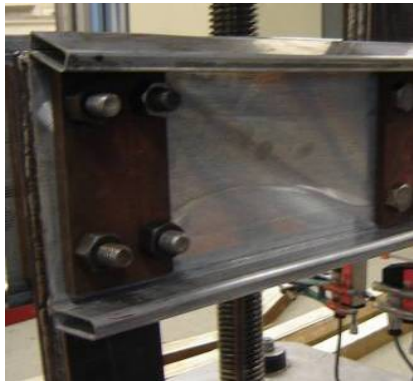


(b) Depth of Web Opening = 60 mm



(c) Depth of Web Opening = 102 mm

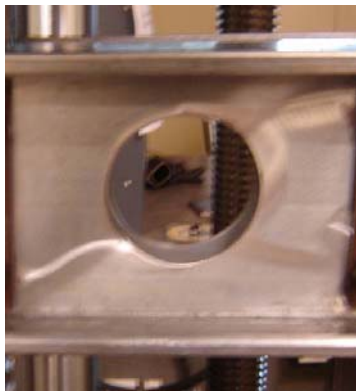
**Figure 7: Failure Mode of 150x45x2.0 LSB with Web Openings
(Aspect Ratio = 1.5)**



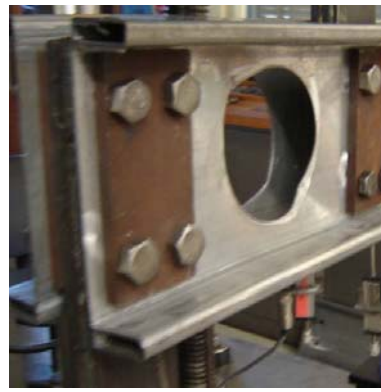
(a) No Web Opening



(b) Depth of Web Opening = 60 mm

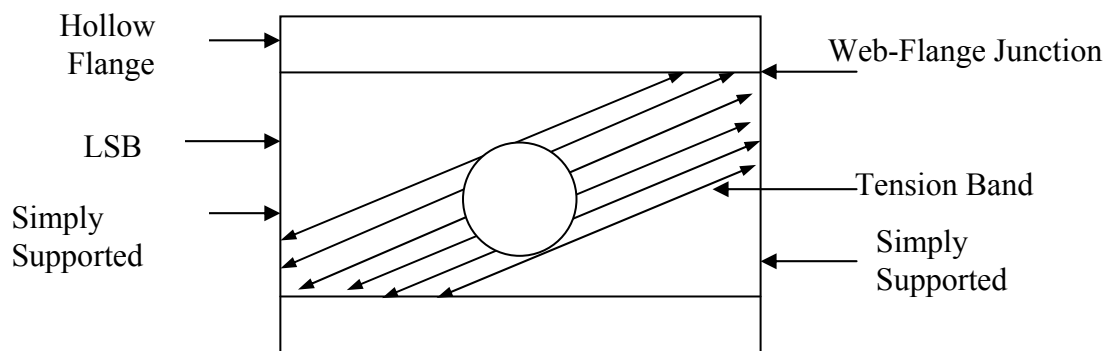
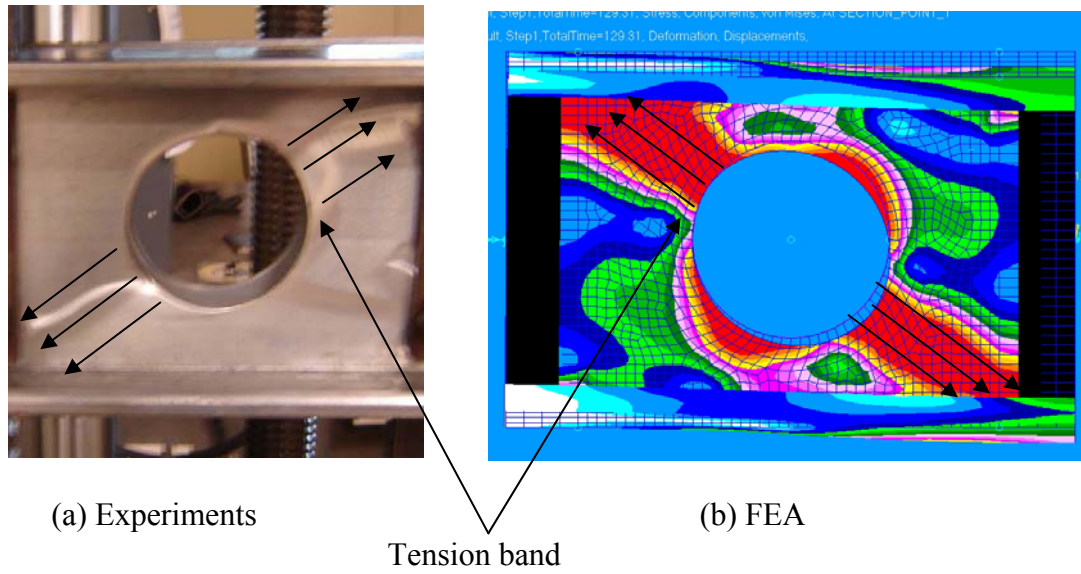


(c) Depth of Web Opening = 102 mm



(d) Depth of Web Opening = 127 mm

**Figure 8: Failure Modes of 200x45x1.6 LSB with Web Openings
(Aspect Ratio = 1.5)**



(c) Schematic Diagram

Figure 9: Mechanics of LSB with Web Openings Post-buckling Behaviour in Shear (200x45x1.6 LSB with 102 mm Web Openings)

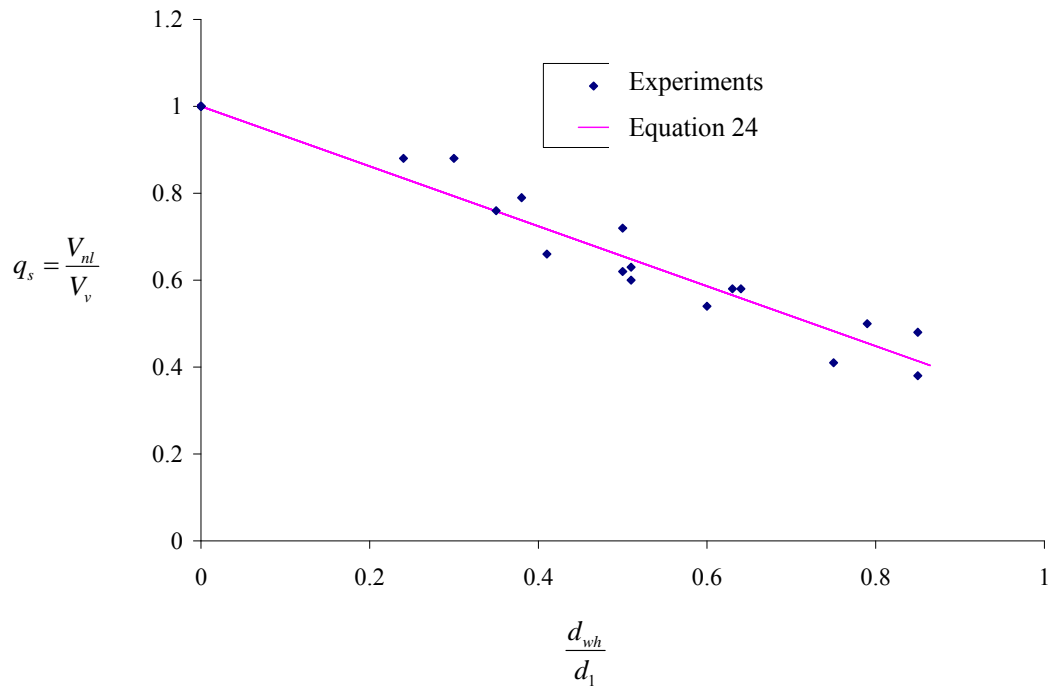
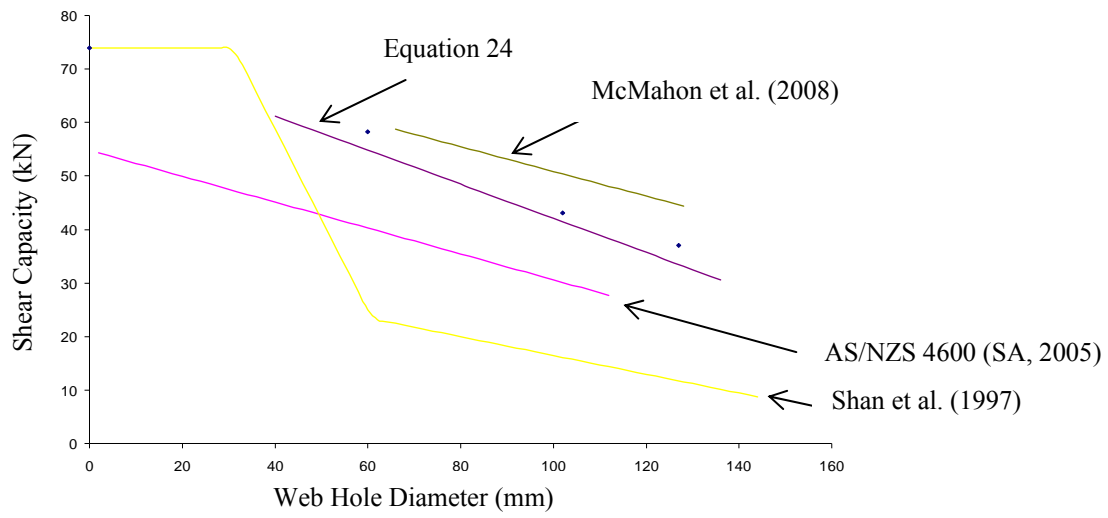


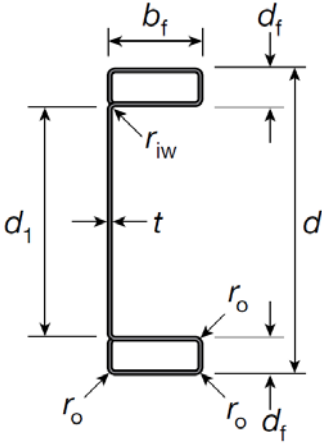
Figure 10: Shear Capacity Reduction Factor of LSBs with Web Openings



**Figure 11: Shear Capacity versus Depth of Web Opening
(200x60x2.0 LSB, Aspect Ratio = 1.5)**

Table 1: Nominal Dimensions of LSB Sections

LSB Section	Depth	Flange		Thick-ness	Corner Radius	
		Width	Depth		r_o	r_{iw}
	d	b_f	d_f	t	r_o	r_{iw}
300x75x3.0LSB	300	75	25.0	3.00	4.50	3.00
300x75x2.5LSB	300	75	25.0	2.50	3.75	3.00
300x60x2.0LSB	300	60	20.0	2.00	3.00	3.00
250x75x3.0LSB	250	75	25.0	3.00	4.50	3.00
250x75x2.5LSB	250	75	25.0	2.50	3.75	3.00
250x60x2.0LSB	250	60	20.0	2.00	3.00	3.00
200x60x2.5LSB	200	60	20.0	2.50	3.75	3.00
200x60x2.0LSB	200	60	20.0	2.00	3.00	3.00
200x45x1.6LSB	200	45	15.0	1.60	2.40	3.00
150x45x2.0LSB	150	45	15.0	2.00	3.00	3.00
150x45x1.6LSB	150	45	15.0	1.60	2.40	3.00
125x45x2.0LSB	125	45	15.0	2.00	3.00	3.00
125x45x1.6LSB	125	45	15.0	1.60	2.40	3.00



* d, b_f, d_f = External dimensions

Table 2: Shear Capacities of LSBs from McMahon et al. (2008)

Web Size	200x60x2.5 LSB		200x45x1.6 LSB		125x45x2.0 LSB	
	V_u	M^*	V_u	M^*	V_u	M^*
No Hole	69.1	22.5	39.5	12.8	28.7	9.3
60 mm Hole	67.7	22.0	36.2	11.8	27.3	8.9
102 mm Hole	58.7	19.1	28.4	9.2	N/A	N/A
127 mm Hole	50.1	16.3	23.8	7.8	N/A	N/A

Note: V_u - Shear capacity (kN) and M^* - associated applied moment at failure (kNm)

Table 3: Dimensional Requirements for Web Openings in LSBs (OATM, 2008)

LSB Depth (mm)	Maximum Hole Diameter (mm)	Min. Distance from Support to Edge of Holes (mm)	Minimum Clear Spacing between Holes (mm)
300	145	375	450
250	125	315	450
200	85	250	450
150	60	190	450
125	30	160	450

Table 4: Details of LSB Test Specimens with Web Openings
(Aspect Ratio = 1.5)

No.	LSB Section	f_{yw} (MPa)	t_w (mm)	d_1 (mm)	d_1/t_w	d_{wh} (mm)	d_{wh}/d_1
1	150x45x1.6	454.2	1.58	120.0	75.9	0	0.00
2		454.2	1.58	120.0	75.9	60	0.50
3		454.2	1.58	120.0	75.9	102	0.85
4	150x45x2.0	422.6	1.97	120.0	60.9	0	0.00
5		422.6	1.97	120.0	60.9	60	0.50
6		422.6	1.97	120.0	60.9	102	0.85
7	200x45x1.6	452.1	1.61	169.6	105.3	0	0.00
8		452.1	1.61	169.6	105.3	60	0.35
9		452.1	1.61	169.6	105.3	102	0.60
10		452.1	1.61	169.6	105.3	127	0.75
11	200x60x2.0	440.4	1.97	160.0	81.2	0	0.00
12		440.4	1.97	160.0	81.2	60	0.38
13		440.4	1.97	160.0	81.2	102	0.64
14		440.4	1.97	160.0	81.2	127	0.79
15	250x75x2.5	446.0	2.51	201.0	80.1	0	0.00
16		446.0	2.51	201.0	80.1	60	0.30
17		446.0	2.51	201.0	80.1	102	0.51
18		446.0	2.51	201.0	80.1	127	0.63
19	300x75x2.5	449.1	2.51	250.0	99.6	0	0.00
20		449.1	2.51	250.0	99.6	60	0.24
21		449.1	2.51	250.0	99.6	102	0.41
22		449.1	2.51	250.0	99.6	127	0.51
23	300x75x3.0	440.1	2.86	250.0	87.4	0	0.00
24		440.1	2.86	250.0	87.4	60	0.24
25		440.1	2.86	250.0	87.4	102	0.41
26		440.1	2.86	250.0	87.4	127	0.51

Table 5: Ultimate Shear Capacities of LSBs with Web Openings

No.	LSB Section	d_{wh} (mm)	Exp. Shear Capacity(kN)	Ult. Shear Capacity (kN)		
				McMahon et al. (2008)	AS/NZS 4600 (2005)	Shan et al. (1997)
1	150x45x1.6	0	47.5	47.5	47.5	47.5
2		60	29.4	35.6	21.6	12.8
3		102	18.1	NA*	NA*	6.5
4	150x45x2.0	0	59.5	59.5	59.5	59.5
5		60	42.6	44.6	21.7	16.1
6		102	28.4	NA*	NA*	8.2
7	200x45x1.6	0	54.2	54.2	54.2	54.2
8		60	41.4	NA	39.6	23.3
9		102	29.1	37.9	30.4	12.6
10		127	22.2	33.9	NA*	9.5
11	200x60x2.0	0	74.0	74.0	74.0	74.0
12		60	58.3	NA*	40.9	23.6
13		102	43.1	50.4	30.6	16.0
14		127	37.0	44.6	NA*	11.8
15*	250x75x2.5	0	118.9	118.9	118.9	118.9
16*		60	104.2	NA*	69.6	72.8
17		102	>75.0	88.7	56.5	31.7
18		127	69.1	81.3	48.8	26.2
19*	300x75x2.5	0	125.1	125.1	125.1	125.1
20*		60	109.8	NA*	95.8	104.0
21*		102	82.4	99.6	82.1	38.1
22		127	>75.0	93.3	74.0	33.3
23	300x75x3.0	0	Not Available	NA	NA	NA
24*		60	>120	NA	NA	NA
25*		102	112.1	NA	NA	NA
26*		127	92.0	NA	NA	NA

* Test results from the simplified experimental arrangement based on single LSBs.

NA*: d_{wh}/d_1 ratio exceeds the limits of design equations in AS/NZS 4600 (SA, 2005) and McMahon et al. (2008).

NA: Not Applicable

Not Available: Test could not be completed due to limited capacity of machine (300kN).

Table 6: Shear Capacity Reduction Factor for LSBs with Web Openings

No.	LSB Section	d_{wh} (mm)	Exp. Shear Capacity (kN)	q_s (Exp.)	q_s (Eq. 24)	$\frac{q_s(Exp.)}{q_s(Eq.24)}$
1	150x45x1.6	0	47.5	1.00	1.00	1.00
2		60	29.4	0.62	0.66	0.95
3		102	18.1	0.38	0.41	0.92
4	150x45x2.0	0	59.5	1.00	1.00	1.00
5		60	42.6	0.72	0.66	1.10
6		102	28.4	0.48	0.41	1.16
7	200x45x1.6	0	54.2	1.00	1.00	1.00
8		60	41.4	0.76	0.76	1.00
9		102	29.1	0.54	0.59	0.92
10		127	22.2	0.41	0.48	0.85
11	200x60x2.0	0	74.0	1.00	1.00	1.00
12		60	58.3	0.79	0.74	1.07
13		102	43.1	0.58	0.56	1.04
14		127	37.0	0.50	0.46	1.10
15	250x75x2.5	0	118.9	1.00	1.00	1.00
16		60	104.2	0.88	0.79	1.11
17		102	>75.0	>0.63	>0.65	>0.97
18		127	69.1	0.58	0.57	1.03
19	300x75x2.5	0	125.1	1.00	1.00	1.00
20		60	109.8	0.88	0.83	1.05
21		102	82.4	0.66	0.72	0.92
22		127	>75.0	>0.60	>0.65	>0.93
23	300x75x3.0	0	Not Available	1.00	1.00	1.00
24		60	>120.0	NA	0.83	NA
25		102	112.1	NA	0.72	NA
26		127	92.0	NA	0.65	NA