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DOI: https://doi.org/10.1016/j.istruc.2022.12.017

# Assessment of Eurocode shear design provisions for cold-formed steel sections

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**Abstract**

Cold-formed steel (CFS) sections are widely used in structural systems and are particularly prone to shear failure in short span applications. Therefore, accurate shear strength estimation is essential for the design. However, while there exists an abundance of research studies investigating the shear behaviour of CFS sections, there is a clear need for the assessment of Eurocode 3 shear design provisions. Towards meeting this need, the design of CFS sections subjected to shear is studied herein. Existing experimental data on CFS lipped channels and hollow flange sections were assembled from the literature. The collected experimental data was then used to assess the accuracy of the current Eurocode 3 shear design provisions. The current Eurocode 3 design provisions are shown to provide overly conservative shear strength predictions against the experimental data. Modifications, including the slenderness of the cross-section to reflect the shear buckling coefficient were proposed. The proposed modifications to the current design provisions offer substantially improved accuracy in shear strength predictions.

**Keywords**: Cold-formed steel sections; Shear strength; Experimental data; Eurocode 3; New design equations.

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**Nomenclature**

| | |
|---|---|
| $\emptyset$ | Slope of the web reference to the flanges |
| $\lambda_w$ | Relative web slenderness |
| $\lambda_{w,new}$ | Revised relative web slenderness |
| $\gamma_{M0}$ | Partial factor |
| $a$ | Shear span |
| $b$ | Flange width |
| $COV$ | Coefficient of variation |
| $d_1$ | Clear web height |
| $E$ | Elastic modulus of the material |
| $f_{bv}$ | Shear strength considering the buckling into account |
| $f_{yb}$ | Basic yield strength of the steel |
| $h$ | Web height |
| $h_w$ | Web height between midlines of the flanges |
| $k_n$ | Level of restraint |
| $k_{sf}$ | Shear buckling coefficients of plates with simple-fixed boundary condition |
| $k_{ss}$ | Shear buckling coefficients of plates with simple-simple boundary condition |
| $k_v$ | Shear buckling coefficient |
| $s_w$ | Height of the web measured between the midpoints of the corners |
| $t$ | Thickness |
| $r$ | Corner radius |
| $V_{b,Rd}$ | Shear resistance |
| $V_{EN1993-1-3}$ | Shear strength prediction from Eurocode |
| $V_{Test}$ | Shear strength from experiments |

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# 1. Introduction

Cold-formed steel (CFS) sections are used as primary and secondary load-bearing elements. Thin-walled CFS sections are often subjected to various loading patterns and subjected to failures including shear failure. Shear strength of the CFS section is, therefore, treated as an essential part of member design. International design codes – European standards (EN1993-1-3[1]), North American Standards (AISI S100 [2]), and Australian and New Zealand standards (AS/NZS 4600 [3]) – contain shear design provisions to estimate the shear strength of CFS sections. These shear design provisions have been suggested for updates based on extensive research studies on the shear response of CFS sections.

Over the past, shear behaviour of CFS sections has been investigated through numerous experimental and numerical campaigns. Keerthan and Mahendran [4-8] investigated the shear behaviour of CFS hollow flange sections [4-6] and lipped channel sections [7, 8], and explored the level of restraint at the web-flange juncture and suggested including this effect in the design rules. They concluded that the level of restraint influences the shear strength of the section and restraint level may vary from cross-section to cross-section. In parallel, Pham and Hancock [9-11] conducted experimental and numerical studies on CFS lipped channel sections and SupaCee (CFS channels with longitudinal web stiffeners and edge stiffened lip) and proposed modifications to the design rules. The outcomes of these abovementioned studies, such as the direct strength method (DSM) approach proposed by Pham and Hancock [9, 10] has been adopted in AISI S100 [2] and AS/NZS 4600 [3], while Keerthan and Mahendran [4-6]'s modified shear buckling coefficients considering level of restraint has been included in the Appendix D3 of AS/NZS 4600 [3]. Pham et al. [12, 13] reported more recent advancements of minimising the effect of bending moments on shear capacity, including new shear test configurations. Further, the shear and flexural behaviour of optimised CFS channels and built-

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up sections were reported in [14-17], in which innovative CFS cross-sections were considered in addition to lipped channel cross-sections.

The Eurocode 3 standard shear design provision has been assessed by the previous research studies for cold-formed stainless steel channels [18] and rolled-formed aluminium channels [19]. Based on the experimental and numerical results, Dissanayake et al.[18] proposed new series of Eurocode 3 based shear design equations for stainless steel channels while Rouholamin et al.[19] proposed Eurocode 3 based shear design equations for aluminium channels taking inelastic reserve capacities into account. As described, previous investigations have focused on improving the accuracy of AISI S100 [2], and AS/NZS 4600 [3] design provisions and the development of the DSM approach. However, investigations into the assessment of EN 1993-1-3 [1] shear design provisions remain scarce.

Therefore, an investigation into the shear behaviour of CFS sections is performed aiming to assess the suitability of current EN 1993-1-3 [1] design provisions. Experimental results available from the literature were collected and a shear strength data pool was developed. The data pool included various cross-sections of CFS sections (lipped channels and hollow flange channels) and covered a wide dimensional range. The current shear design provisions were assessed against the shear strength data pool. Finally, modified shear design provisions for CFS sections under shear loading are proposed.

## 2. Experimental database

### *2.1. General*

It is essential to have a database of compiled results for the calibration of EN1993-1-3 [1] shear design provisions for CFS sections. Therefore, experimental results were collected from literature which investigated the shear behaviour of CFS sections. This section covers a brief overview of shear experiments followed by a summary of collected database.

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### *2.2. Overview of experimental studies*

Research studies by Keerthan and Mahendran [5], Keerthan and Mahendran [8], Pham and Hancock [11], Chen et al. [20] and Pham et al. [12, 13] investigated the shear behaviour of CFS sections through series of laboratory experiments. Their experimental investigations include hollow flange and lipped channel CFS cross-sections (See Fig. 1).

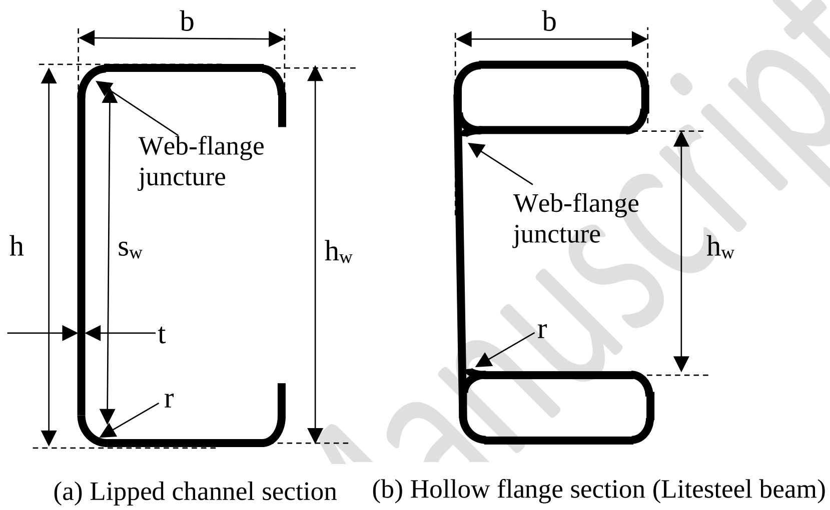


Fig. 1: Cold-formed steel cross-sections and notations

Shear experiments were conducted with a 3-point loading setup with simply supported boundary conditions. Thick web side plates were attached at loading and end-support locations to overcome the failure of the web due to the concentrated applied load and reactions. In the experiments, the specimen lengths were chosen to be shorter to minimise the possibilities of bending failure and to allow the sections to predominantly fail under shear. Hence, an aspect ratio (ratio between shear span and clear web height) of 1.0 was used while other higher aspect ratios (1.5, 1.6 and 2.0) were also considered. Further, the experimental set-up included angle straps attached to the flanges. This is to eliminate the flange distortion that can occur as a result of unbalanced shear flow in open sections. Experimental set-ups used in previous research studies are depicted in Fig. 2.

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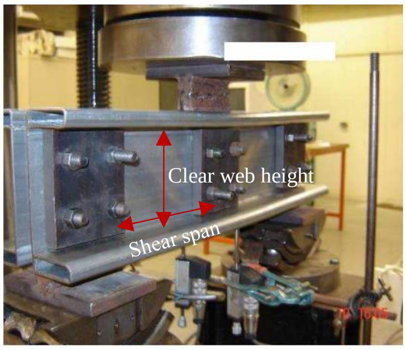
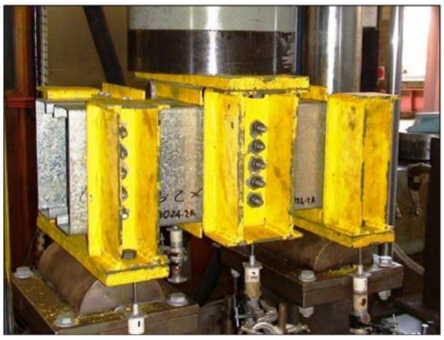



(a) Shear test set-up for hollow flange sections [21]



(b) Shear test set-up for lipped channel sections [22]

Fig. 2: Shear test set-up used in previous research studies [21, 22]

### *2.3. Data collected from the literature*

A total of 67 shear strength experimental data of CFS sections were collected from 6 studies and presented in Table 1. The collected database includes CFS lipped channel sections and hollow flange sections. For CFS hollow flange sections, 23 results were collected from Keerthan and Mahendran's [5] experimental study. It should be noted that Keerthan and Mahendran [5] reported 25 experimental results and among them, 2 results were excluded in this study as those 2 specimens were not tested to failure due to the limited capacity of the testing machine. The tested aspect ratios were 1.0, 1.5, and 1.6.

CFS lipped channel sections' experimental data were assembled from five research studies (Keerthan and Mahendran [8], Pham and Hancock [11], Chen et al. [20], and Pham et al. [12, 13] ). Shear experiments of lipped channel sections by Keerthan and Mahendran [8] include a total of 15 results and the experiments were performed under aspect ratios of 1.0 and 1.5. In total, 18 experimental results of lipped channel sections were reported in Pham and Hancock's [11] investigation. All shear experiments were conducted with an aspect ratio of 1.0. Using new test configurations to minimise the influence of bending on shear capacity, Pham et al. [12, 13] performed 7 experiments in total for lipped channels with aspect ratios of 1.0 and 2.0. It should be noted that these experimental results include repeated tests. Pham and Hancock

[11] and Pham et al. [12, 13] also conducted experiments on SupaCee channels which contain longitudinal stiffeners in the CFS channel web and the edge stiffened lip. Those results were not considered in the database as the shear response of CFS channels with longitudinal web stiffeners is not within the focus of this investigation. Recently, Chen et al. [20] conducted experiments on CFS lipped channel sections subjected to shear and 4 results from that study were added to the database. Chen et al. [20]'s study used aspect ratios of 1.0 and 1.5 in their experiments.

Table 1: Summary of the collected database

| Source | Family | No. of data | Aspect ratio | Properties range | | |
|---|---|---|---|---|---|---|
| | | | | Web height (mm) | Thickness (mm) | Yield strength (MPa) |
| Keerthan and Mahendran [5] | Hollow flange | 23 | 1.0 – 1.6 | 125 - 300 | 1.58 – 2.51 | 422.6 – 459.7 |
| Keerthan and Mahendran [8] | Lipped Channel | 15 | 1.0 – 1.5 | 120 - 250 | 1.49 - 1.95 | 271 - 537 |
| Pham and Hancock [11] | Lipped Channel | 18* | 1.0 | 153.15 – 204.71 | 1.5 – 2.4 | 483.49 – 541.13 |
| Chen et al. [20] | Lipped Channel | 4 | 1.0 - 1.5 | 240 - 290 | 1.81 – 2.11 | 301.6 – 308.6 |
| Pham et al. [12] | Lipped Channel | 4* | 1.0 - 2.0 | 200.45 – 204.5 | 1.51 – 1.54 | 490 – 538.9 |
| Pham et al. [13] | Lipped Channel | 3* | 2.0 | 202.3 – 203.7 | 1.51 – 1.54 | 532.5 |

*Included repeated tests

Table 1 summarises the assembled data from past research studies including the dimensional and mechanical properties range. It can be observed that the assembled data covers almost a full range of practically used dimensional and yield strength parameters in CFS construction. For example, the web height of the lipped channel sections ranges from 120 mm to 290 mm while the depth of the hollow flange sections ranges from 125 mm to 300 mm. Overall, the thickness range is 1.49 mm – 2.51 mm. Besides, the assembled data covers steel

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grades of nominal yield strength between 271 MPa and 541.13 MPa. Therefore, the assembled dataset was appropriate to assess and calibrate the EN 1993-1-3 [1] shear design provisions.

## 3. Shear design of CFS channels to Eurocode 3

### *3.1. Existing shear design provisions*

According to Eurocode 3 – Part 1.3 (EN1993-1-3 [1]), the design shear resistance of a CFS member $\left(V_{b,Rd}\right)$ may be estimated from:

$$V_{b,Rd} = \frac{(h_w/\sin\emptyset)tf_{bv}}{\gamma_{M0}} \tag{1}$$

where $h_w$ and $\emptyset$ are the web height between midlines of the flanges and slope of the web reference to the flanges, respectively. The thickness of the web is represented by $t$. The term $f_{bv}$ is the shear strength considering the buckling into account (shear buckling strength). $\gamma_{M0}$ is the partial factor for the ultimate limit state.

Further, EN1993-1-3 [1] defines that the shear buckling strength $f_{bv}$ may be estimated based on the relative web slenderness $(\lambda_w)$ and basic yield strength of the steel $\left(f_{yb}\right)$. For webs without longitudinal stiffeners which is within the focus of this investigation, relative web slenderness $(\lambda_w)$ maybe determined from the following equation.

$$\lambda_w = 0.346\left(\frac{s_w}{t}\right)\sqrt{\frac{f_{yb}}{E}} \tag{2}$$

Here, E is the elastic modulus of the material and $s_w$ is the height of the web measured between the midpoints of the corners (median points of the corners). EN1993-1-3 [1] indicates that shear buckling strength $f_{bv}$ is influenced by the stiffening conditions: (i) web with stiffening at the support and (ii) web without stiffening at the support. Here, stiffening at the support is referred to attachment of cleat plates or web side plates to prevent the web distortion (web crippling) and safely resisting the support reaction. The experiments reported in the literature have been conducted attaching the rigid web side plates to the web at loading and end support locations. EN1993-1-3 [1] recommends the following equations (Eq. (3)) to estimate the shear buckling strength $f_{bv}$ for the condition web with stiffening at the support.

$$f_{bv} = 0.58 f_{yb} \qquad \text{for} \quad \lambda_w \leq 0.83 \tag{3a}$$

$$f_{bv} = \frac{0.48 f_{yb}}{\lambda_w} \qquad \text{for} \quad \lambda_w > 0.83 \tag{3b}$$

Fig. 3 depicts the graphical demonstration of the shear design provisions recommended in EN1993-1-3 [1], denoted as EC3 in the subsequent figures. The shear design provision for the web without stiffening at the support condition has also been plotted in the same figure in order to demonstrate the different design approaches provided in EN1993-1-3 [1].

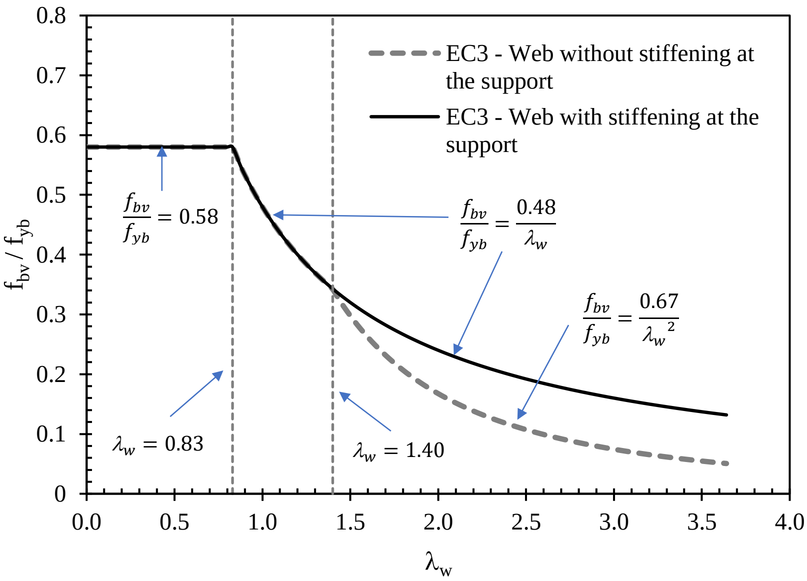


Fig. 3: Current EN1993-1-3 [1] shear design provisions.

### *3.2. Assessment of existing shear design provisions against experimental results*

The accuracy of the shear design provisions available in EN1993-1-3 [1] has been assessed against the collected database of experimental results from the literature. For the comparison, shear buckling strength $f_{bv}$ values for the corresponding experimental results were calculated from Eq. (1) with the known shear strength results and dimensional parameters of the cross-section. The relative web slenderness $\lambda_w$ for the corresponding experiments was determined using Eq. (2) substituting dimensional and mechanical properties of the cross-section. The partial factor $\gamma_{M0}$ was set to unity for the comparison. Since all the experiments have been performed with attached web side plates at loading and end support locations, web with stiffening at the support criteria in EN1993-1-3 [1] was focused.

The shear design curve from the EN1993-1-3 [1] is plotted in Fig. 4, along with the experimental data of hollow flange sections. The shear strength comparison between the experimental results and the shear strength predictions from EN1993-1-3 [1] is depicted in Fig. 5 while the statistical comparison is presented in Table 2. It can be observed that the current shear design provisions in EN1993-1-3 [1] provide overly conservative shear strength predictions as the average of $V_{Test}/V_{EN\ 1993\text{-}1\text{-}3}$ equals 1.47 with coefficient of variation (COV) equals 0.141. The overly conservative prediction can be explained by the level of restraint for the hollow flange sections at the web-flange juncture and aspect ratios [4-6] as the experimental results included aspect ratios of 1.0 – 1.6 and the failure to recognise those effects.

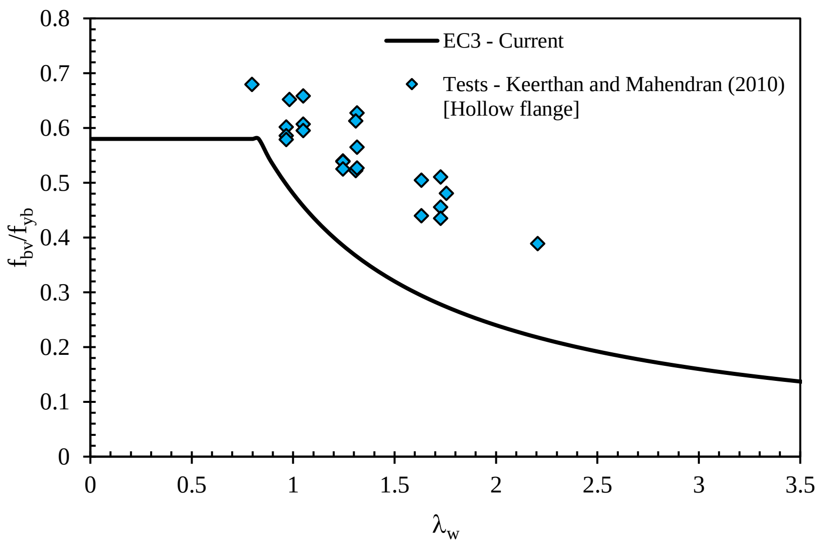


Fig. 4: CFS hollow flange section experimental results [5] (Series A) plotted with EN1993-1-3 [1] shear design curve

DOI: https://doi.org/10.1016/j.istruc.2022.12.017

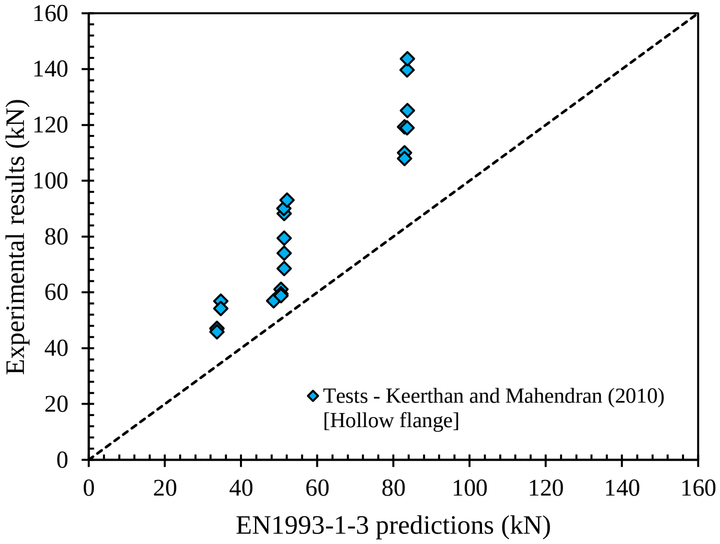


Fig. 5: Comparison of CFS hollow flange section experimental results [5] (Series A) with predictions from current EN1993-1-3 [1].

Table 2: Summary of the collected database

| Source | Family | Evaluation parameter | $V_{Test}/V_{EN1993\text{-}1\text{-}3}$ | $V_{Test}/V_{Modified\ EN1993\text{-}1\text{-}3}$ |
|---|---|---|---|---|
| Keerthan and Mahendran [5] | Hollow flange (Series A) | Mean | 1.47 | 1.01 |
| | | COV | 0.141 | 0.065 |
| Keerthan and Mahendran [8] | Lipped Channel (Series B) | Mean | 1.38 | 0.98 |
| | | COV | 0.142 | 0.090 |
| Pham and Hancock [11] | Lipped Channel (Series C) | Mean | 1.49 | 1.03 |
| | | COV | 0.130 | 0.102 |
| Chen et al. [20] | Lipped Channel (Series D) | Mean | 1.27 | 0.90 |
| | | COV | 0.219 | 0.154 |
| Pham et al. [12] | Lipped Channel (Series E) | Mean | 1.50 | 1.09 |
| | | COV | 0.107 | 0.013 |
| Pham et al. [13] | Lipped Channel (Series F) | Mean | 1.39 | 1.10 |
| | | COV | 0.046 | 0.046 |
| Overall | | Mean | 1.44 | 1.00 |
| | | COV | 0.141 | 0.092 |

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The lipped channel sections' experimental data from the previous research studies [8, 11-13, 20] is plotted along with the shear design curve from the EN1993-1-3 [1], shown in Fig. 6, Fig. 8, Fig. 10, and Fig. 12. The shear strength predictions from EN1993-1-3 [1] and experimental results are compared in Fig. 7, Fig. 9, Fig. 11, and Fig. 13. The statistical evaluation for the variation in shear strength prediction is given in Table 2. As can be seen from this comparison, all experiment-to- EN1993-1-3 [1] prediction ratios are higher than unity, demonstrating that the current EN1993-1-3 [1] shear design provisions provide extremely conservative predictions for CFS lipped channel sections. Hence, it is clear that account of the aspect ratio and level of restraint at the web-flange juncture [7, 8] should be taken into account to achieve accurate shear strength predictions using the EN1993-1-3 [1].

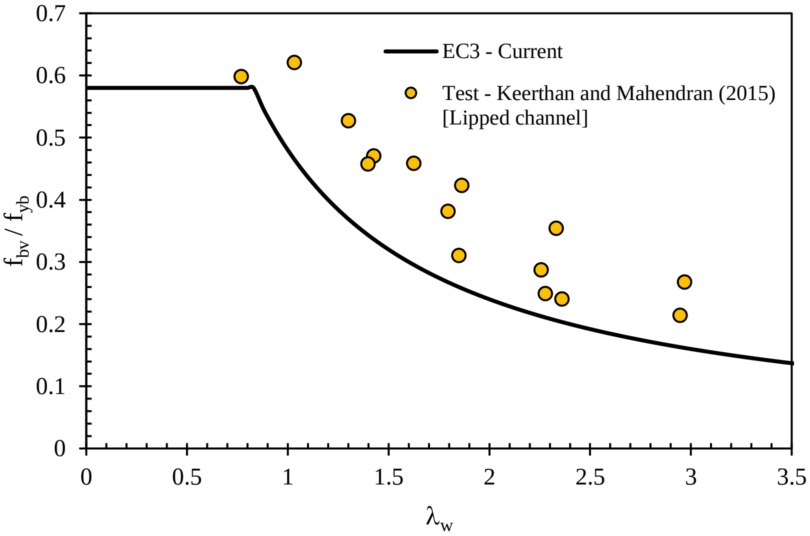


Fig. 6: CFS lipped channel section experimental results [8] (Series B) plotted with EN1993-1-3 [1] shear design curve

DOI: https://doi.org/10.1016/j.istruc.2022.12.017

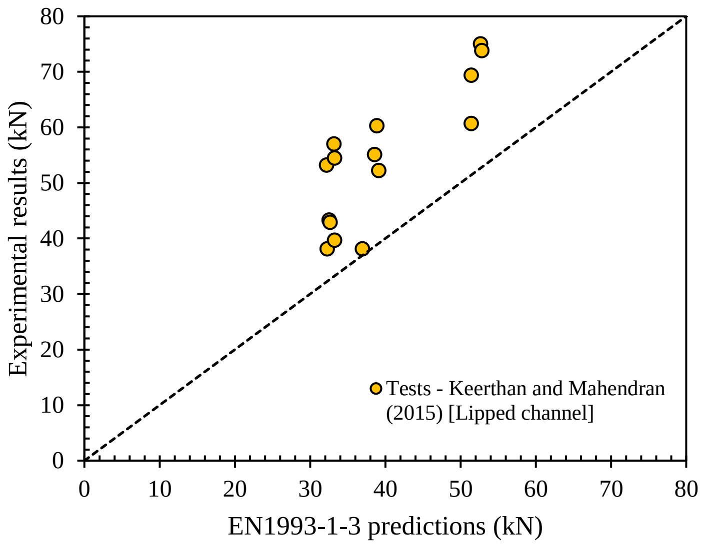


Fig. 7: Comparison of CFS lipped channel section experimental results [8] (Series B) with predictions from current EN1993-1-3 [1].

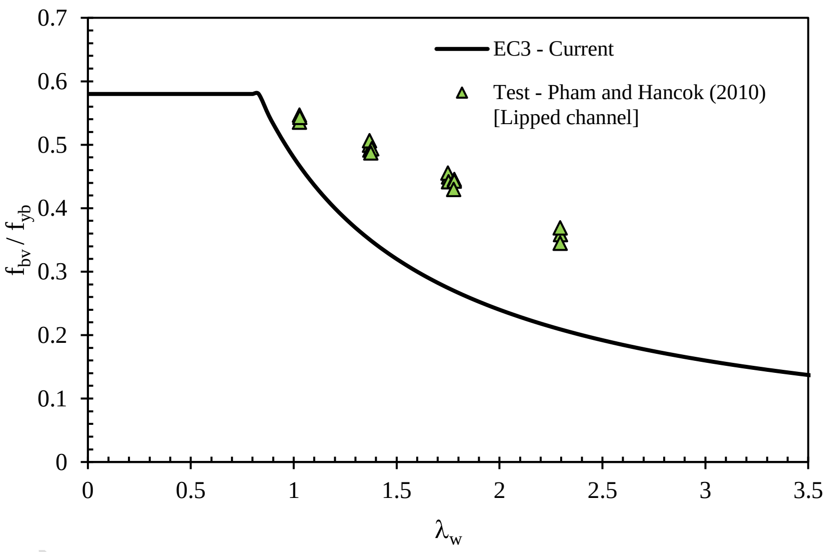


Fig. 8: CFS lipped channel section experimental results [11] (Series C) plotted with EN1993-1-3 [1] shear design curve

DOI: https://doi.org/10.1016/j.istruc.2022.12.017

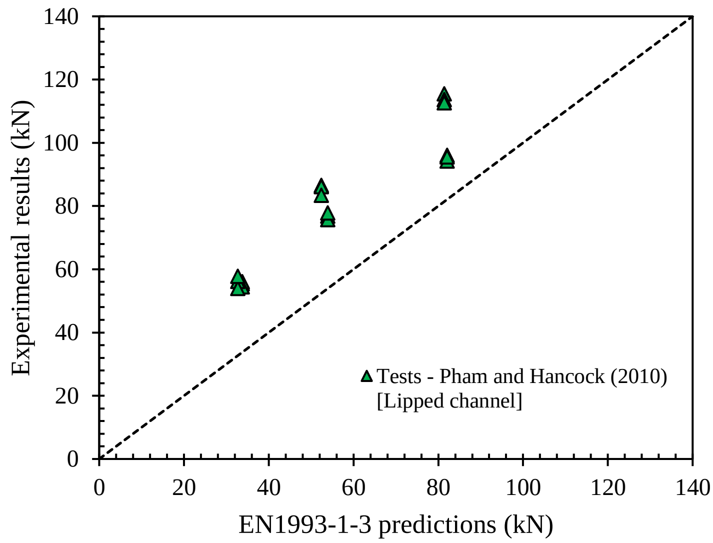


Fig. 9: Comparison of CFS lipped channel section experimental results [11] (Series C) with predictions from current EN1993-1-3 [1].

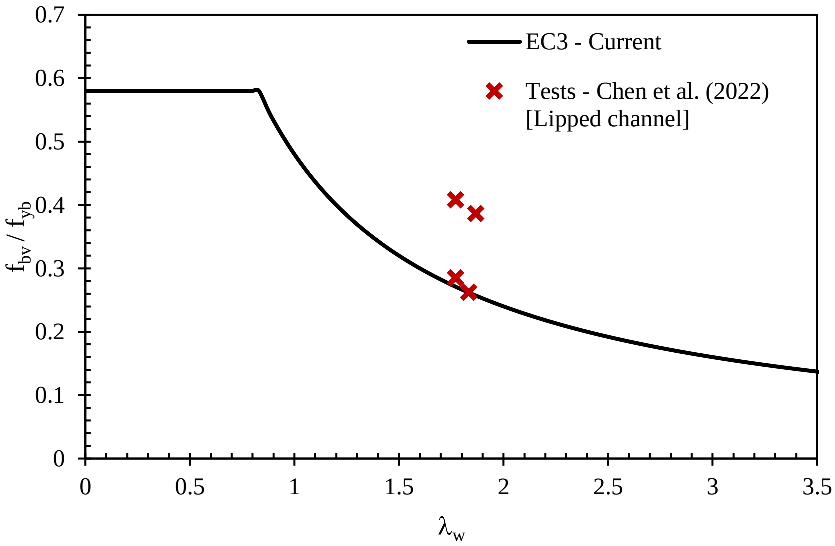


Fig. 10: CFS lipped channel section experimental results [20] (Series D) plotted with EN1993-1-3 [1] shear design curve

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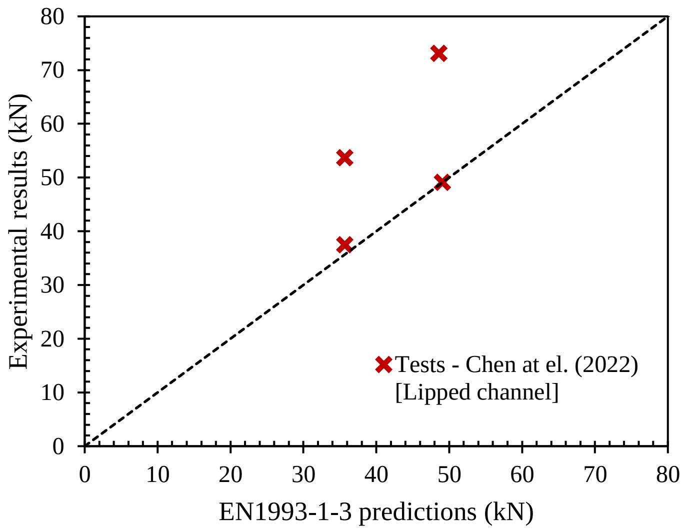


Fig. 11: Comparison of CFS lipped channel section experimental results [20] (Series D) with predictions from current EN1993-1-3 [1].

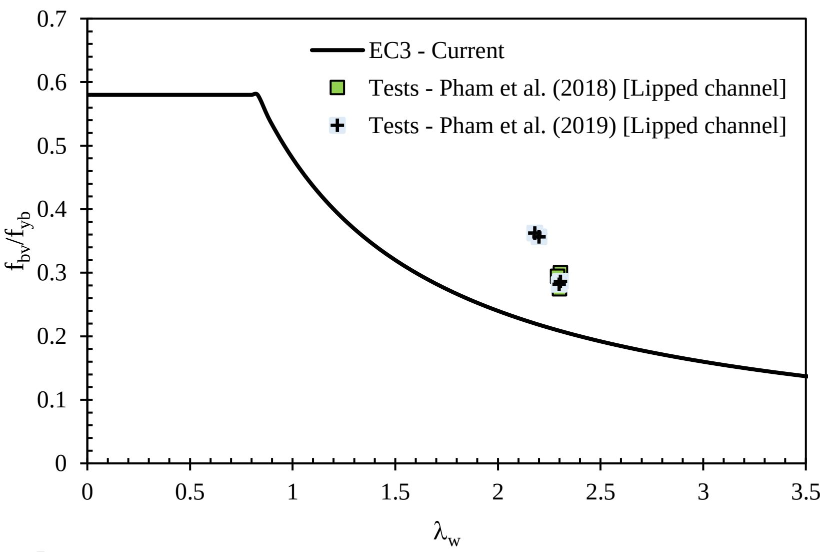


Fig. 12: CFS lipped channel section experimental results [12, 13] (Series E and Series F) plotted with EN1993-1-3 [1] shear design curve

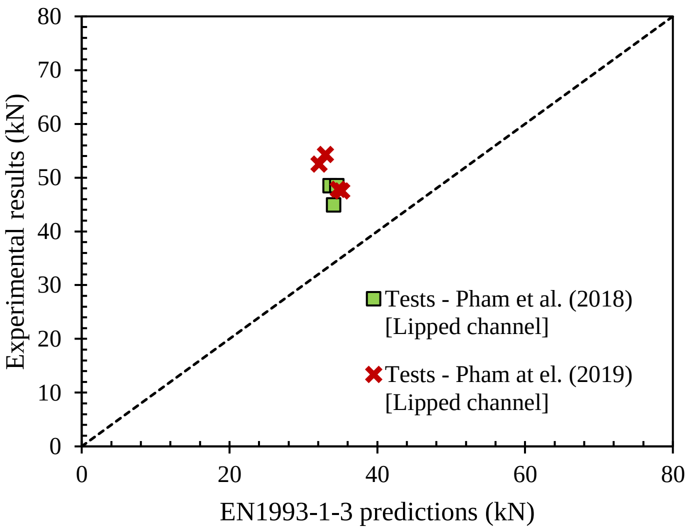


Fig. 13: Comparison of CFS lipped channel section experimental results [12, 13] (Series E and Series F) with predictions from current EN1993-1-3 [1].

Fig. 14 presents the overall comparison of the experimental results against the EN1993-1-3 [1] shear design curve and shear strength predictions while Fig. 15 explains the influence of the aspect ratios on shear strength predictions. Overall, the comparisons revealed that the current EN1993-1-3 [1] shear design provisions are unable to provide more accurate shear strength predictions for CFS sections including hollow flange sections and lipped channel sections.  This is primarily due to the fact that the influence of cross-sectional shape which is directly related to level of restraint at web-flange juncture and the influence of aspect ratio, which were not well reflected in the current EN1993-1-3 [1] shear design provisions. New shear design provisions are presented in the next section to address these shortcomings.

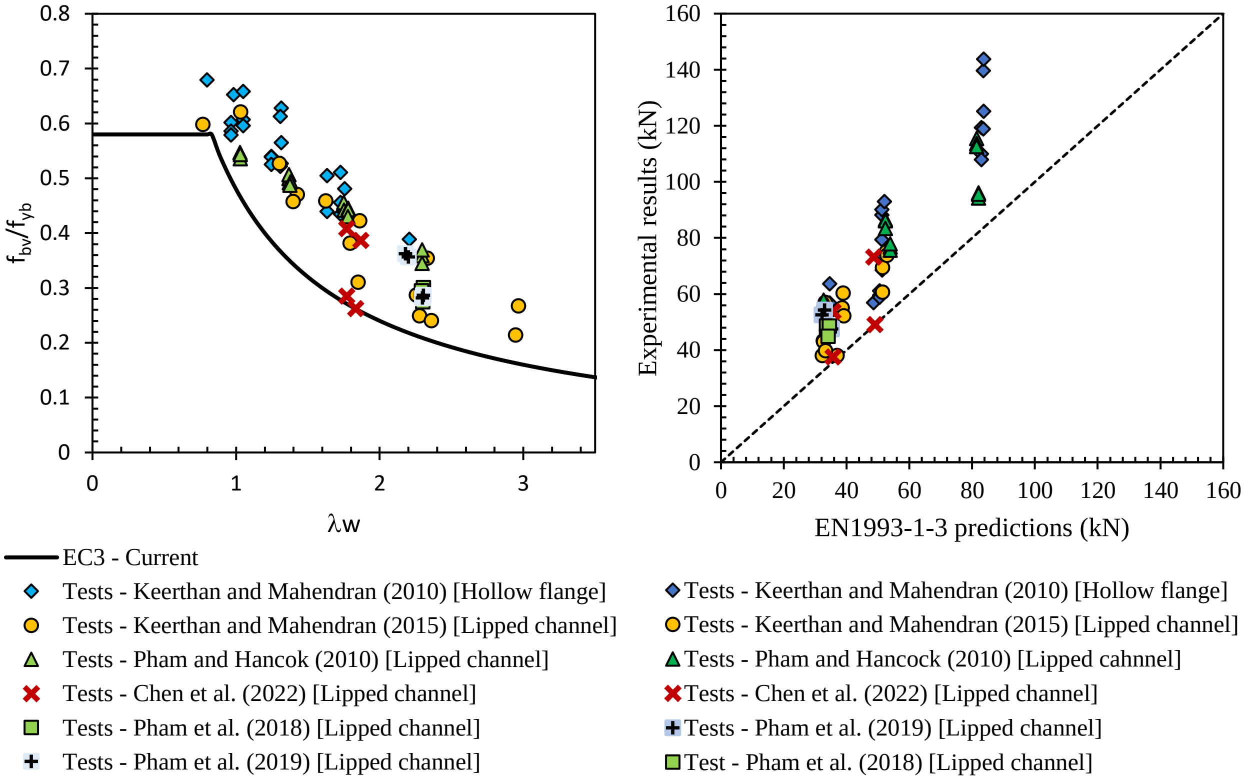


Fig. 14: Comparison of CFS sections' experimental results with current EN1993-1-3 [1]

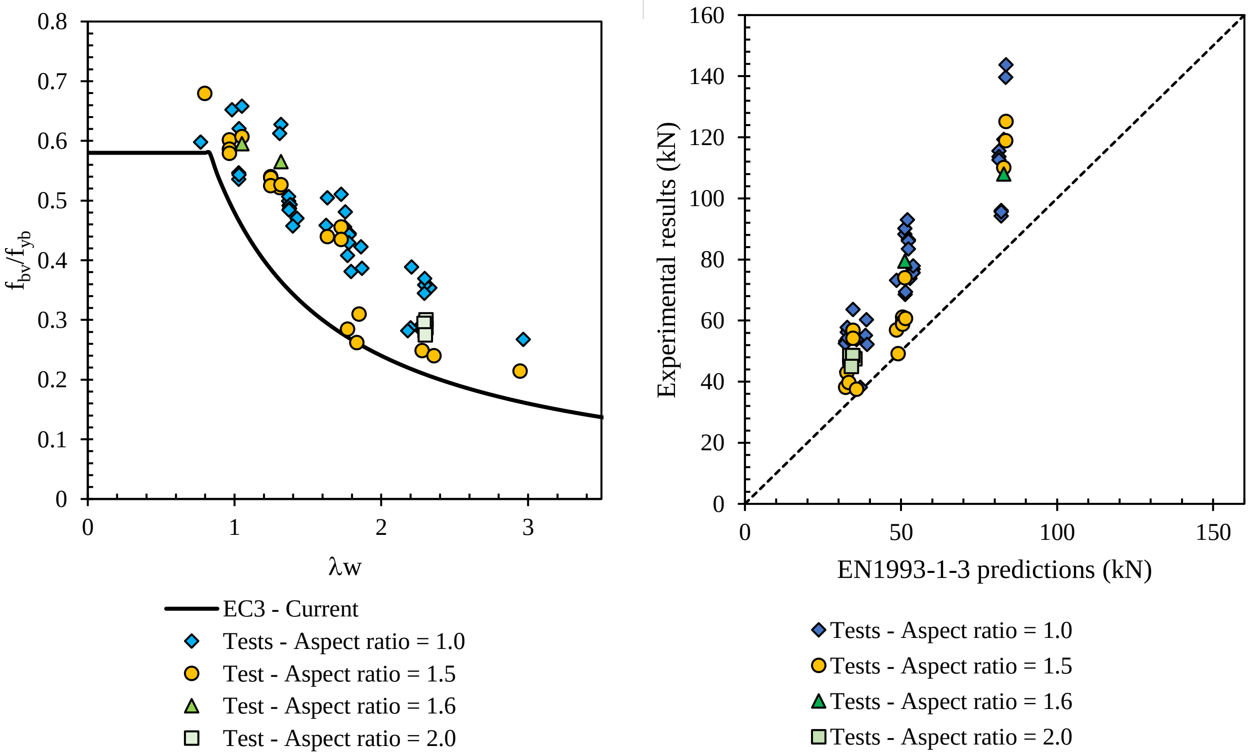


Fig. 15: Comparison of CFS sections' experimental results with current EN1993-1-3 [1] with respect to aspect ratios

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## 4. Proposed shear design provisions

### *4.1. Modified Eurocode shear design provisions*

The shear design provisions, as prescribed in EN1993-1-3 [1], do not adequately considered the level of restraint at the web-flange juncture and aspect ratio. Therefore, improved design provisions may be achieved by including those effects into the existing shear design provisions in EN1993-1-3 [1]. Keeping the same format of existing design provisions for estimating the shear strength of the sections, a revised relative web slenderness $\lambda_{w,new}$ coefficient is proposed herein in Eq. (4). As can be noted, the new proposal replaces the original coefficient 0.346 shown in Eq. (2) for 0.735 and includes a new shear buckling coefficient namely $k_v$. The coefficient 0.735 was obtained using classic genetic algorithm combined with generalised reduced gradient (GRG) non-linear solver.

$$\lambda_{w,new} = \frac{0.735}{\sqrt{k_v}} \left(\frac{s_w}{t}\right) \sqrt{\frac{f_{yb}}{E}} \tag{4}$$

The shear buckling coefficient $(k_v)$ can be determined according to Keerthan and Mahendran [23], as given in Eqs. (5)-(7) or from numerical methods (finite strip methods) proposed in [24-26]. Keerthan and Mahendran [19] proposed improved shear buckling coefficients for the various types of CFS cross-sections including hollow flange sections and lipped channel sections. They have also concluded that the level of restraint at the web-flange juncture is in between simple and fixed boundary conditions, and the level of restraint may vary from cross-section to cross-section. Hollow flange sections exhibit boundary conditions closer to fixed support at web-flange juncture while open sections such as lipped channel sections exhibit boundary conditions closer to simply support [19]. Following Lee et al. [27] approach, Keerthan and Mahendran [23] defined the shear buckling coefficient $(k_v)$ as given in Eq. (5). Here, $k_{ss}$ and $k_{sf}$ are the shear buckling coefficients of plates with simple-simple and simple-fixed boundary conditions, respectively. To elaborate, simple-fixed boundary condition means fixed boundary condition at web-flange juncture and simple supported boundary condition at other two edges. The coefficient $k_n$ is the level of restraint at web-flange juncture in a cross-section. The level of restraint at web-flange juncture for lipped channel section is 23% ($k_n = 0.23$) and hollow flange section (Litesteel beam) is 87% ($k_n = 0.87$). The shear buckling coefficients $k_{ss}$ and $k_{sf}$ can be calculated using Eq. (6) and Eq. (7), respectively. These two coefficients are sensitive to the aspect ratio which is defined as ratio between shear span $(a)$ and clear web height $(d_1)$.

$$k_v = k_{ss} + k_n\left(k_{sf} - k_{ss}\right) \tag{5}$$

$$k_{ss} = 4 + \frac{5.34}{(a/d_1)^2} \qquad \text{for } \frac{a}{d_1} < 1 \tag{6a}$$

$$k_{ss} = 5.34 + \frac{4}{(a/d_1)^2} \qquad \text{for } \frac{a}{d_1} \geq 1 \tag{6b}$$

$$k_{sf} = \frac{5.34}{(a/d_1)^2} + \frac{2.31}{(a/d_1)} - 3.44 + 8.39(a/d_1) \qquad \text{for } \frac{a}{d_1} < 1 \tag{7a}$$

$$k_{sf} = 8.98 + \frac{5.61}{(a/d_1)^2} - \frac{1.99}{(a/d_1)^3} \qquad \text{for } \frac{a}{d_1} \geq 1 \tag{7b}$$

Using this revised relative web slenderness $\lambda_{w,new}$ the shear buckling strength $f_{bv}$ for web with stiffening at the support condition may be estimated as given in Eq. 8. From the experimental database it was observed that for the small web slenderness the CFS sections experience inelastic reserve. Therefore, for $\lambda_{w,new} \leq 0.83$ the inelastic reserve has been considered as a linear trend observing the results in that region.

$$f_{bv} = (-0.22\lambda_{w,new} + 0.77)f_{yb} \qquad \text{for } \lambda_{w,new} \leq 0.83 \tag{8a}$$

$$f_{bv} = \frac{0.48 f_{yb}}{\lambda_{w,new}} \qquad \text{for } \lambda_{w,new} > 0.83 \tag{8b}$$

### *4.2. Assessment of the proposed shear design provisions*

Fig. 16 illustrates the proposed shear design curve considering the revised relative web slenderness $\lambda_{w,new}$ and the inelastic reserve. The experimental data points have also been plotted in the same figure to demonstrate the capability of the proposed shear design curve in estimating the shear strength of CFS sections. It can be observed that the proposed curve with revised relative web slenderness $\lambda_{w,new}$ shows good accuracy with the experimental results. Further, the experiment results were compared against the predictions from the modified EN1993-1-3 [1] shear design provisions, shown in Fig. 17. Statistical comparisons of the proposed shear design provisions are also given in Table 2. For the proposed EN1993-1-3 [1] shear design provisions, the statistical comparison showed a mean value of 1.00 and a coefficient of variation (COV) of 0.092 for experiments-to-prediction ratios. This reflects a greater accuracy as the statistical comparison showed the mean value of 1.44 and COV of 0.141 for current EN1993-1-3 [1] shear design provisions.

Therefore, it can be concluded that the proposed shear design provisions given in Eqs. (4)-(8) improve accuracy and consistency compared to the current EN1993-1-3 [1] shear design provisions. The proposed methodology improves predictions because unlike EN1993-1-3 [1] takes into account the cross-sectional shape, aspect ratio and level of restraint at the web-flange juncture which significantly influences the shear response. The proposed shear design provisions are limited to CFS sections without longitudinal stiffeners and web with stiffening at the support condition.

DOI: https://doi.org/10.1016/j.istruc.2022.12.017

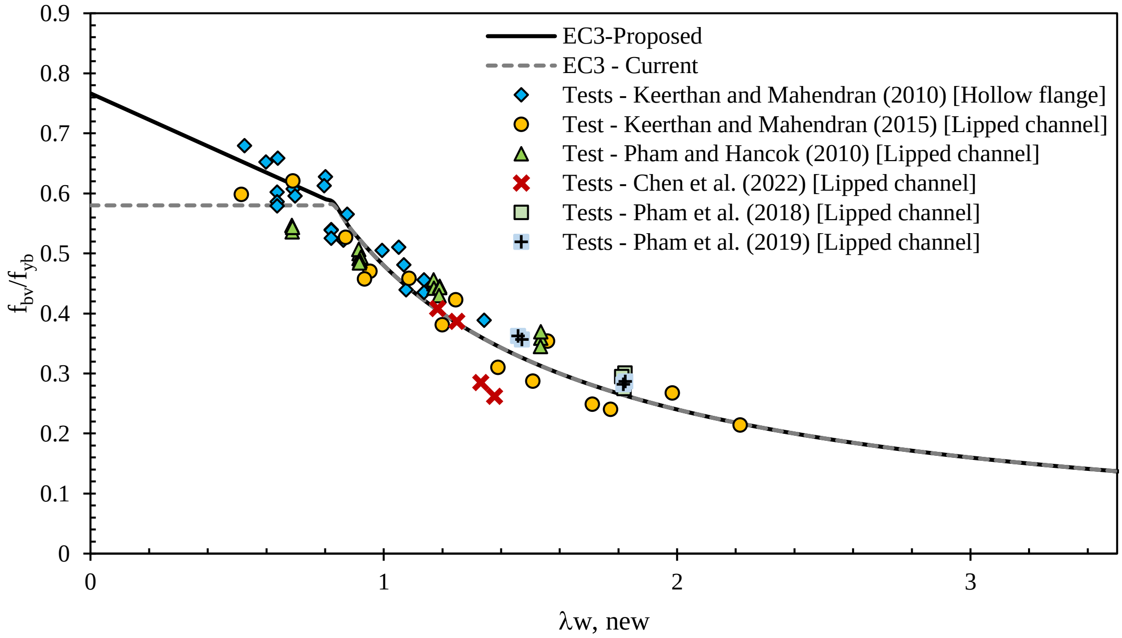


Fig. 16: Proposed shear design curve plotted with experimental results [5, 8, 11-13, 20].

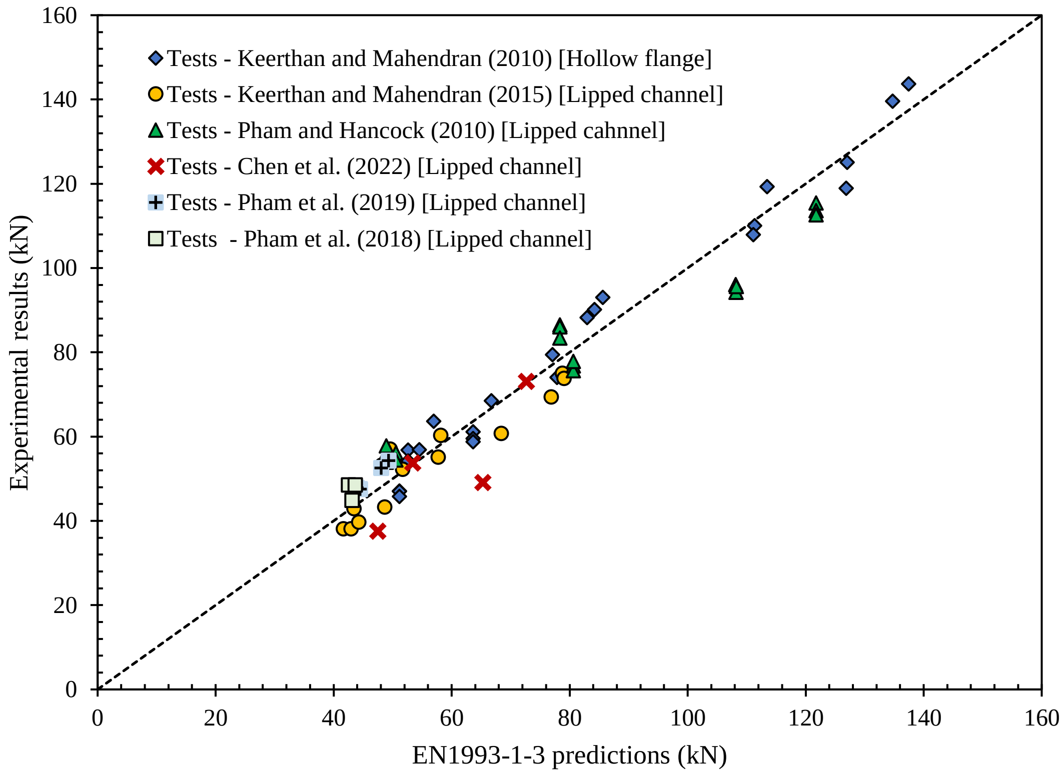


Fig. 17: Comparison of experimental results [5, 8, 11-13, 20] with predictions from the proposed shear design provisions.

DOI: https://doi.org/10.1016/j.istruc.2022.12.017

## 5. Conclusions

This paper has investigated the shear behaviour and design of cold-formed steel (CFS) sections according to Eurocode 3 (EN1993-1-3 [1]) shear design provisions. Experimental results available from the literature were assembled to form a database. The database comprised 67 shear experimental test results of CFS lipped channels and hollow flange sections. Using the assembled data, current EN1993-1-3 [1] shear design provisions were compared and the comparison showed that the current EN1993-1-3 [1] shear design provisions provide overly conservative predictions for CFS sections. It was found that the variation is due to the influence of cross-sectional shape, level of restraint at the flange-to-web junction and influence of aspect ratio. These effects were not well reflected in the current EN1993-1-3 [1] shear design provisions. Therefore, new EN1993-1-3 [1] based shear design provisions are proposed. The proposal included revised relative web slenderness ($\lambda_{w,new}$) and equation to take account of inelastic reserve for small relative web slenderness values below 0.83. The proposed shear design methodology in Eqs (4)-(8) showed a greater accuracy with a mean value of 1.00 and coefficient of variation (COV) of 0.092. Overall, the new shear design provisions lead to more accurate and reliable shear strength predictions for CFS sections.

**Acknowledgements**

The authors would like to thank Teesside University for technical support and research facilities.

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