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Flexural behaviour of cold-formed steel oval hollow section beams

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Abstract

This paper presents a detailed investigation into the flexural strength and behaviour of cold-formed steel oval hollow section beams. A series of four-point bending tests were conducted as part of the experimental programme. Following this, a total of 12 tested beams were simulated by a non-linear finite element analysis. Upon validation of the newly developed numerical model, a total of 36 numerical results were generated through a parametric study to assess the structural response of the oval hollow section beams over a wider range of cross-section dimensions. The experimental and numerical data were used to evaluate the applicability of five existing design methods for cold formed steel structural members, including Eurocode 3, the North American Specification, the Australian/New Zealand Standards, the direct strength method and the continuous strength method. In general, the results indicate that all the three design standards produce unconservative predictions, while the direct strength method and the continuous strength method yield relatively accurate and consistent predictions.

Keywords: Cold-formed steel; Design methods; Experimental investigation; Finite element analysis; Flexural behaviour; Oval hollow section.

1. Introduction

In the construction industry, the classic steel tubular sections include circular, square and rectangular hollow sections. In recent decades, the structural steel catalogue has seen some new additions, such as the oval hollow section (OHS) and elliptical hollow sections (EHS), which are now becoming more accessible and widely

32 recognised by engineers and architects alike [1]. The geometry and characterisation of
33 the OHS currently has collective definitions, as shown in Fig. 1, which outlines the
34 geometry of a general OHS. Departing from elliptical hollow section (EHS), OHS
35 investigated herein is composed of two flat web plates and two curved, semi-circular
36 flanges (see Fig. 1). Both EHS and OHS integrates the aesthetic appeal of circular
37 hollow sections (CHS) with the structural efficiency of the rectangular hollow section
38 (RHS) [2].

39 Since OHS is a new hollow section shape, the current knowledge and
40 understanding of the OHS is rather limited due to a lack of sufficient structural
41 performance data and design guidance, which might inhibit the uptake of OHS. To date,
42 there are limited research studies conducted on oval hollow sections. In terms of studies
43 on cold-formed steel OHS columns, it has been found two publications by Zhu and
44 Young [3,4] in literature. Sachidananda and Singh [5,6] conducted experimental and
45 numerical studies on fixed ended stub columns of lean duplex stainless steel OHS.
46 However, the majority of the investigations have been focused on EHS, with the aid of
47 equivalent depth design approach [1, 7-12]. For instance, Chen and Young [7-8,13]
48 have carried out comprehensive experimental and numerical studies on cold-formed
49 steel EHS structural members under various loading conditions and proposed accurate
50 design rules accordingly. The structural performance of hot-rolled steel [12, 14-16] and
51 stainless steel [11, 17] EHS members have also been investigated. In addition, Chen
52 and Young [18-22] have performed extensive investigations on cold-formed steel semi-
53 oval hollow section (SOHS) structural members, where the profile of SOHS is half of
54 the OHS investigated in this study. Regarding concrete-infilled OHS members, Ding et
55 al. [23], Hassanein and Patel [24] and Zhou and Young [25] carried out experimental
56 investigation on concrete-filled steel OHS stub columns. The current Eurocode 3 [26-
57 27], North American specification [28], Australia/New Zealand standard [29] for steel
58 structures do not cover the structural design of cold-formed steel OHS. Therefore,
59 certain assumptions for the design of OHS must be made in these design codes.

60 The aim of this project is to investigate the behaviour of cold-formed steel OHS
61 under bending. Further to this, the applicability of current design methods for
62 engineering design of cold-formed steel OHS beams is assessed. An experimental
63 programme on OHS beams is presented in this paper. In addition, a numerical model
64 was developed using finite element analysis to simulate the loading responses of OHS

beams under four-point bending. Upon validation against test results, the numerical model is used to conduct a parametric study, generating a total of 36 numerical results with a wider range of cross-sectional dimensions. The data pool combining experimental and numerical results is used to assess the applicability of design predictions by the existing design methods for cold formed steel structural members, including Eurocode 3 [26-27], the North American Specification [28], the Australian/New Zealand Standard [29], the direct strength method (DSM) and the continuous strength method (CSM). The reliability of these design rules is also evaluated herein.

2. Experimental investigation

A test programme has been designed to determine the flexural capacity and failure modes of cold-formed steel OHS beams using four-point bending tests. A total of three cold-formed steel OHS beams were tested. The definitions of the nominal cross-sectional geometries are summarized in Fig. 1(a), where D is the section depth, h is the flat depth, W is the section width, and t is the thickness of the section. The test specimens, O1-PB, O2-PB and O3-PB, refer to the nominal cross-sectional dimensions of $120 \times 48 \times 2.0$ mm ($D \times W \times t$), $42 \times 21 \times 2.8$ mm and $30 \times 15 \times 1.6$ mm, respectively, as shown in Table 1. A non-dimensional cross-section slenderness $\bar{\lambda}_p$, is also reported in Table 2, and the calculation method is given in Eq. (1).

$$\bar{\lambda}_p = \frac{h/t}{28.4\varepsilon\sqrt{k_\sigma}} \quad \text{Eq. (1)}$$

where ε is equal to $\sqrt{235/f_y}$, f_y is yield stress, k_σ is buckling factor equal to 4.

These OHS specimens were subjected to bending about the minor (x-x) axis. The beam specimens in this study had the same batch of column specimens conducted by Zhu and Young [3]. For each OHS, coupon specimens were extracted from the flat and curved portions of the sections, and the coupon test results are reported in Ref. [3]. The grade of the steel materials is S355, with the nominal yield stress of 355 MPa. For better comparison, the material properties of each OHS specimen were determined using the tensile coupon test method following the Australian Standard AS1391 [30]. The Young's modulus (E), yield stress (f_y), ultimate tensile stress (f_u) and strain at fracture

(ε_f) for flat and curved parts are shown in Table 1. Fig. 2 presents typical experimental, static and true curves of the stress-strain relationship of the flat part extracted from specimen O1-PB [3].

Four-point bending tests were conducted to determine the bending moment capacities (M_{exp}), the curvature at ultimate (k_{exp}), the full moment-curvature curves and the failure mode of the cold-formed OHS beams. The OHS specimens were loaded symmetrically at two points through a spreader beam, using a half round welded to bearing plate and roller on bearing plate, as shown in Fig. 3. The load was applied via a servo-controlled hydraulic testing machine. Steel plates were welded to the specimens at loading points and supports. Loads were applied to these welded plates to avoid localize failure of the specimens due to the concentrated loads. The length between the support and the applied load, is known as the shear length (shear span) of 700 mm for specimen O1-PB, and 400 mm for specimens O2-PB and O3-PB. The length between the two loading points is known as the bending length (moment span), which is 500 mm for all three specimens. Three linear variable displacement transducers (LVDTs) were positioned along the bending length of each specimen, with one LVDT positioned at the mid-span of the beam. The vertical deflections measured by LVDTs were used to calculate the curvature of the pure bending region. The tests were conducted by displacement control at a constant speed of 0.1 mm/min.

Fig. 4 depicts the full moment-curvature curves for the three specimens. Table 2 presents the results of bending moment capacities, the curvatures at ultimate and the failure mode. O1-PB was failed by a local buckle after yielding denoted as “L”, while O2-PB and O3-PB were failed by in-plane bending, denoted as “F” in Table 2. Shear failure was not observed for all beams. A typical failure mode of tested specimen O3-PB is shown in Fig. 5(a).

3. Numerical Modelling

In parallel with the experimental investigation, finite element (FE) models were established using ABAQUS version 6.13 [31] to simulate the four-point bending tests, and further to perform a parametric study. Test results of three specimens bending about minor axis generated in this study together with results of nine specimens subjected to the same four-point bending but about either minor axis or major axis reported by Chan and Gardner [12] were used to validate FE models.

3.1 Development of FE models

The type of elements selected for the FE models was S4R reduced integration shell elements, which are four-noded, iso-parametric and suitable for thin shell applications. Convergence studies, on similar elliptical hollow section specimens have been performed previously by Silvestre and Gardner [32] and McCann and Gardner [33]. In both studies, it was concluded that a characteristic mesh size of 10×10 mm can accurately simulate the behaviour of thin-walled EHS specimens, whilst optimizing computation time.

The measured geometries and material properties of the OHS specimens were used in each FE model. In the linear elastic analysis stage of simulation, the material properties were defined by the Young's modulus (E), the yield stress (f_y) and the Poisson's ratio ($\nu = 0.3$ for steel). In the non-linear plastic analysis stage of the simulation, material non-linearity was introduced by the true stress (f_{true}) and true plastic strain ($\varepsilon_{pl,ln}$) given by Eqs. (2) – (3). The engineering stress-strain curve from the coupon tests, the converted static stress-strain curve and the true stress-strain curve for the specimen O1-PB are shown in Fig. 2.

$$f_{true} = f(1 + \varepsilon) \quad \text{Eq. (2)}$$

$$\varepsilon_{pl,ln} = \ln(1 + \varepsilon) - \frac{f_{true}}{E} \quad \text{Eq. (3)}$$

where f and ε are stress and strain in engineering stress-strain curve.

To simulate the simple supported end conditions of the experimental tests, the FE modelled beam was restrained at the vertical and out-of-plane directions at the end supports, but allowing for in-plane and out-of-plane rotations. An additional restraint was added at the mid-span of the beam in the longitudinal direction. The developed FE model is demonstrated in Fig. 6. The non-linear Riks method was applied to simulate the non-linear response of the beams beyond the ultimate load.

3.2 Model validation

The FE models developed for all test specimens were validated against the experimental results. The moment-curvature relationships predicted by the FE analyses are plotted against the experimental results; a comparison for specimens bent about

minor axis and major axis are shown in Fig. 7. Please note that for EHS beams, the failure mode is dominated by material yielding; elastic buckling will not dominate unless for those very slender sections, as reported in Ref [12]. According to the proposed classification limits in Ref [12], the cross-section of specimen $500 \times 250 \times 8.0$ -B2 belongs to Class 1. Thus, the moment-curvature curves look more ductile. Comparison between the ultimate moment (M_u) predicted by the FE modelling and tests are presented in Table 3. Fig. 5 shows the failure modes of a tested specimen O3-PB and its FE model, which were both failed by global flexural buckling. Overall, satisfactory agreement between the test results and numerical models was achieved, with a mean ratio of FE to test ultimate moment of 1.00 and coefficient of variation (CoV) of 0.042.

3.3 Parametric Study

The developed FE model closely predicted the structural responses of the cold-formed steel beams. Thereafter, the validated FE model was used for the parametric study, in order to generate a data pool of numerical results. In parametric study, the width and thickness of the section were varied to achieve a wide range of section slenderness, so that the effects of section slenderness on flexural behaviour of OHS beams were investigated. A total of 36 specimens were included in the parametric study. The results from parametric study are outlined in Table 4. The specimens have been labelled with respect to the cross-sectional width (W), and the specimen thickness (t). For example, the specimen label “W75T10.0”, refers to a specimen with a cross sectional width of 75 mm, and a thickness of 10 mm. All specimens in the parametric study had the same depth (D) of 300 mm. The material properties of the specimens in this study are the same as those of specimen O1-PB in the experimental study. The length of the specimens is taken as 1900 mm, with the corresponding shear length of 700 mm and the bending length of 500 mm. These dimensions are consistent throughout the parametric study. All specimens in this study have been analysed in minor axis bending. The magnitude of local imperfection used throughout the study was chosen as 10% of the thickness of the specimens. A mesh density of 10×10 mm (length by width) was also used.

4. Design methods

4.1 Eurocode 3

European code for structural steel design (EC3) places steel sections into behavioural classes based upon the cross-sectional slenderness [12]. The classification of different cross-sections is defined in Section 5.5 of EN1993-1-1 [26]. Class 1 sections are defined by being able to achieve and maintain their full plastic moment (M_{pl}), in bending. Class 2 sections are also able to achieve their full plastic moment in bending with a lower rotational capacity than Class 1 sections. Class 3 sections do not exhibit full plastic moment and are limited to achieving the elastic (yield) moment (M_{el}). Class 4 sections do not achieve the elastic (yield) moment and exhibit local buckling in the elastic range. Design resistance for bending about one principal axis of a cross-section is defined under Section 6.2.5 of EN1993-1-1 [26] and Section 4.4 of EN1993-1-5 [27] based on the aforementioned cross-sectional classification rules. Equations for calculating bending moment capacity (M_{EC}) are listed in Eqs. (4) – (6) respectively.

$$M_{EC} = M_{pl} = W_{pl} f_y \quad \text{for Class 1 or 2 cross sections} \quad \text{Eq. (4)}$$

$$M_{EC} = M_{el} = W_{el} f_y \quad \text{for Class 3 cross sections} \quad \text{Eq. (5)}$$

$$M_{EC} = W_{eff} f_y \quad \text{for Class 4 cross sections} \quad \text{Eq. (6)}$$

where W_{pl} is plastic section modulus, W_{el} is elastic section modulus, W_{eff} is effective section modulus. The EHS calculation of equivalent depth is detailed in Gardner [1], while the effective depth of slender OHS (h_{eff}) is calculated based on Eqs. (7) – (9),

$$h_{eff} = \rho h \quad \text{Eq. (7)}$$

$$\text{For } \bar{\lambda}_p \leq 0.5 + \sqrt{0.085 - 0.055\psi} \quad \rho = 1 \quad \text{Eq. (8)}$$

$$\text{For } \bar{\lambda}_p > 0.5 + \sqrt{0.085 - 0.055\psi} \quad \rho = \frac{\bar{\lambda}_p - 0.055(1 + \psi)}{\bar{\lambda}_p^2} \quad \text{Eq. (9)}$$

where ρ is depth reduction factor; ψ is stress ratio and equal to 1.

4.2 North American Specification & Australian/ New Zealand Standards

The design rule specifications provided by North American Specification (AISI S100-2016) [28] and Australian / New Zealand Standards (AS/NZS 4600:2018) [29] are identical in calculating the design strengths of cold-formed steel members acting in

bending. Design methods detailed in Section F2.3 and F3.1 of the AISI S100-2016 and Section 3.6 of the AS/NZS 4600:2018 are adopted for capacity calculations of cold-formed steel OHS beams. In this method, three flexural strengths are considered: yielding strength or lateral-torsional buckling strength (M_{ne}), local buckling strength (M_{bl}) and distortional buckling strength (M_{bd}). Since there is no distortional buckling observed in OHS beams, M_{NAS} or $M_{AS/NZS}$ is obtained as shown in Eq. (10).

$$M_{NAS} \text{ or } M_{AS/NZS} = \min(M_{ne}, M_{bl}) \quad \text{Eq. (10)}$$

The yielding strength or lateral-torsional buckling strength M_{ne} is given by Eqs. (11) – (13).

$$\text{For } f_{cre} \geq 2.78f_y : \quad M_{ne} = W_{el}f_y \quad \text{Eq. (11)}$$

$$\text{For } 0.56f_y < f_{cre} < 2.78f_y : \quad M_{ne} = \frac{10}{9}W_{el}f_y \left(1 - \frac{10f_y}{36f_{cre}}\right) \quad \text{Eq. (12)}$$

$$\text{For } f_{cre} \leq 0.56f_y : \quad M_{ne} = W_{el}f_{cre} \quad \text{Eq. (13)}$$

where f_{cre} is critical elastic lateral-torsional buckling stress, which can be derived by CUFSM programme [34].

Local buckling strength (M_{bl}) is checked as reduction in nominal flexural strength for its interaction with yielding or lateral-torsional buckling. This reduction is considered through either effective width method as given in Eq. (14) or direct strength method discussed in Section 4.3 of this paper.

$$M_{bl} = W_{eff} \frac{M_{ne}}{W_{el}} \quad \text{Eq. (14)}$$

4.3 Direct strength method

The direct strength method (DSM) for members subject to bending is detailed in Section F3.2 of the AISI S100-2016 [28] and clause 7.2.2 of the AS/NZS 4600 [29]. This method also examines three flexural strengths as mentioned in Section 4.2 of this paper. The DSM local buckling moment capacity is taken as M_{bl-DSM} . Without occurrence of distortional buckling, M_{DSM} of the OHS beams is given by Eq. (15),

$$M_{DSM} = \min(M_{ne}, M_{bl-DSM}) \quad \text{Eq. (15)}$$

M_{ne} is determined as Eqs. (11) – (13), M_{bl-DSM} is calculated by Eqs. (16) – (17),

$$\text{For } \lambda_l \leq 0.776: \quad M_{bl-DSM} = M_{ne} \quad \text{Eq. (16)}$$

$$\text{For } \lambda_l > 0.776: \quad M_{bl-DSM} = \left[1 - 0.15 \left(\frac{M_{crl}}{M_{ne}} \right)^{0.4} \right] \left(\frac{M_{crl}}{M_{ne}} \right)^{0.4} M_{ne} \quad \text{Eq. (17)}$$

where $\lambda_l = \sqrt{M_{ne} / M_{crl}}$ is non-dimensional slenderness for local buckling, and $M_{crl} = W_{el} f_{ol}$ is the critical elastic local buckling moment, f_{ol} is the critical elastic local buckling stress which is derived by CUFSM programme [34] and values are listed in Table 4.

4.4 Continuous strength method

The Continuous Strength Method (CSM) is a recently developed design method for tubular structures made of various metallic materials, including cold-formed and high strength structural steel [35], austenitic and duplex stainless steel [36–41], ferritic stainless steel [36–42] and aluminium alloys [43]. Regarding for the cross-section shapes, the CSM is applicable to square and rectangular hollow sections (SHS/RHS) [35–38, 42, 43], I-sections [36–38], channel sections [36–38, 41] and circular hollow sections (CHS) [39, 40]. Based on CSM discussed in Afshan and Gardner [36] and Zhao et al. [37] for RHS, plus the discussion by Buchanan et al. [39] for CHS, there are two key components in CSM method: (1) a bi-linear (elastic, linear hardening) material model, and (2) a base curve covering both non-slender and slender sections. Please note that there is no existing CSM approach for OHS beams at the moment, the CSM approach for RHS and CHS beams have been employed for comparison purpose due to the special geometrics of OHS, i.e. OHS is composed of two flat web plates and two curved, semi-circular flanges, which could be treated as a combination of RHS and CHS. The existing CSM approaches were used in this study.

The material model for cold-formed steel material, as given in Eqs. (18) – (19). The strain hardening modulus E_{sh} is defined by the ultimate tensile stress f_u , the yield strength f_y , the strain at ultimate ε_u and the yield strain ε_y , thus,

$$E_{sh} = (f_u - f_y) / (0.45 \varepsilon_u - \varepsilon_y) \quad \text{Eq. (18)}$$

$$\varepsilon_u = 0.6(1 - f_y / f_u) \quad \text{Eq. (19)}$$

The CSM base curve for RHS is shown as Eq. (20), comprising two parts - for non-slender sections (slenderness $\bar{\lambda}_P \leq 0.68$) and slender sections (slenderness $\bar{\lambda}_P > 0.68$), respectively, where ε_{csm} is the limiting strain for cross-section, $\varepsilon_{csm} / \varepsilon_y$ is strain ratio defined as deformation capacity of cross-section. Similarly, the base curve for CHS is shown in Eq. (21), and the boundary limit between non-slender and slender sections is equal to 0.3.

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \begin{cases} \frac{0.25}{\bar{\lambda}_R^{3.6}} \leq \min(15, \frac{0.4\varepsilon_u}{\varepsilon_y}) & \bar{\lambda}_P \leq 0.68 \\ \frac{1}{\bar{\lambda}_R^{1.05}} (1 - \frac{0.222}{\bar{\lambda}_R^{1.05}}) & \bar{\lambda}_P > 0.68 \end{cases} \quad \text{Eq. (20)}$$

$$\frac{\varepsilon_{csm}}{\varepsilon_y} = \begin{cases} \frac{0.00444}{\bar{\lambda}_C^{4.5}} \leq \min(15, \frac{0.4\varepsilon_u}{\varepsilon_y}) & \bar{\lambda}_P \leq 0.3 \\ \frac{1}{\bar{\lambda}_C^{0.342}} (1 - \frac{0.224}{\bar{\lambda}_C^{0.342}}) & \bar{\lambda}_P > 0.3 \end{cases} \quad \text{Eq. (21)}$$

With the calculated CSM strain ratio ($\varepsilon_{csm} / \varepsilon_y$) and the strain hardening modulus ratio (E_{sh} / E), the CSM bending moment could be predicted by Eq. (22) for RHS beams and Eq. (23) for CHS beams.

$$M_{csm,RHS} = \begin{cases} W_{pl} f_y [1 + \frac{E_{sh}}{E} \frac{W_{el}}{W_{pl}} (\frac{\varepsilon_{csm}}{\varepsilon_y} - 1) - (1 - \frac{W_{el}}{W_{pl}}) / (\frac{\varepsilon_{csm}}{\varepsilon_y})^2] & \bar{\lambda}_P \leq 0.68 \\ \frac{\varepsilon_{csm}}{\varepsilon_y} W_{el} f_y & \bar{\lambda}_P > 0.68 \end{cases} \quad \text{Eq. (22)}$$

$$M_{csm,CHS} = \begin{cases} W_{pl} f_y [1 + \frac{E_{sh}}{E} \frac{W_{el}}{W_{pl}} (\frac{\varepsilon_{csm}}{\varepsilon_y} - 1) - (1 - \frac{W_{el}}{W_{pl}}) / (\frac{\varepsilon_{csm}}{\varepsilon_y})^2] & \bar{\lambda}_P \leq 0.3 \\ \frac{\varepsilon_{csm}}{\varepsilon_y} W_{el} f_y & \bar{\lambda}_P > 0.3 \end{cases} \quad \text{Eq. (23)}$$

4.5 Result comparisons

A summary of the results obtained from the design method predictions is shown in Table 5. Additionally, Figure 8 plots a comparison of experimental and numerical ultimate moment (M_u) to design predictions (M_{Design}) against the cross-sectional slenderness.

Table 5 and Fig. 8 infers that the design moments predicted by the three design codes M_{EC} , M_{NAS} and $M_{AS/NZS}$ are unconservative in comparison with the experimental and numerical results, with mean values of 0.86, 0.91 and 0.91, respectively, and the COV values of 0.155, 0.206 and 0.206, respectively. It shows that the EC3 provides the most unconservative predictions for the moment capacity of the OHS members, while predictions by NAS and AS/NZS are most scattered with biggest COV values. This trend compares well with the study performed by Zhu and Young [4] for OHS columns, in which the mean values for M_u / M_{EC} , M_u / M_{NAS} and $M_u / M_{AS/NZS}$ were 0.90, 0.89, 0.89 respectively. Similarly, studies performed by Theofanous et al. [11] for EHS section beams, suggest that the design predictions from the European code are unconservative, in comparison with the ultimate moment obtained from numerical studies. A comparison of all five design methods shows that the DSM approach is the most accurate and consistent, with a mean value of 1.04 and a COV value of 0.093. This trend can be observed in Figure 8, where it is evident that the DSM predictions lay closest to the unity line. In general, the CSM approach is slightly more conservative and scattered than the DSM, with a mean value of 1.08 and a COV of 0.128 for RHS design, and a mean value of 1.04 and a COV of 0.166 for CHS design.

It is shown in Fig.8 that predictions by NAS and AS/NZS are most scattered with largest COV value of 0.206. This is mainly attributed to unconservative predictions for slender sections, using the effective width method for flat elements; while for stocky sections, the moment capacity is limited to elastic moment, i.e. not allowing the stocky

sections reaching the plastic moment. However, for EC3, the moment capacity of Class 1 stocky sections could reach plastic moment, which yields more accurate predictions compared to the test results, as shown in Fig. 8. But the Predictions by EC3 for slender sections are identical to NAS and AS/NZS, using effective width method. Furthermore, the CSM approach for RHS yields the most accurate predictions for stocky sections with the consideration of strain hardening of steel materials. For slender sections, CSM has calculated the buckling stress based on a design base curve, which accurately predicted the continuous relationship between slenderness and buckling stress of the cross-sections. As for DSM, the local buckling stress (f_{ol}) was obtained by CUFSM based on the modelling of the whole section, instead of flat parts of the section. The gross OHS sections was built in CUFSM to obtain the elastic local buckling stresses, which considers the interaction between adjacent element in the section.

The experimental and numerical results are normalized by the plastic moment, as plotted in Fig. 9. It shows that there is a trend between an increasing cross-sectional slenderness (i.e. a more slender section), and a decreasing moment resistance. This is generally the expected trend and agrees with other studies from Gardner [1] and Chen and Young [22]. The cross-section classification limits proposed by the European design code can generally be considered acceptable, as the results show an increasing moment capacity with decreasing slenderness.

5. Reliability analysis

The reliability of the six considered design methods for cold-formed steel OHS beams were assessed by reliability analysis. The method for the reliability analysis of cold-formed steel structures is detailed in the commentary section B3.2.2 of the AISI S100-2016 [28]. Reliability is quantified by a reliability index β . The reliability index of 2.5 is considered as a lower limit, and thus, a reliability index greater than 2.5 is considered reliable. The resistance factor (ϕ) is dependent on different structural scenarios and design codes under consideration, the calculation methods are given in Eqs. (24) – (26). Load combinations of Dead Loads (DL) and Live Loads (LL) are

employed for analysis. The load combination of 1.35DL + 1.50LL is adopted for EC3 [26], 1.2DL + 1.6LL for NAS [27], 1.2DL + 1.5LL for AS/NZS [28], and 1.2DL + 1.6LL for DSM and CSM, respectively. Thus, ϕ is taken as 1.00 for EC3 design method, 0.90 for NAS and 0.95 for AS/NZS, 0.90 for DSM and CSM methods in terms of beam design. The mean value of material factor M_m equals to 1.10 since the member behaviour is governed by f_y in this study. Detailed calculation method for M_m can be found in Su et al. [43].

$$\phi = (1.609P_m) \exp(-2.5\sqrt{0.0555 + C_p V_p^2}) \quad \text{for EC3} \quad \text{Eq. (24)}$$

$$\phi = (1.582P_m) \exp(-2.5\sqrt{0.0555 + C_p V_p^2}) \quad \text{for AS/NZS} \quad \text{Eq. (25)}$$

$$\phi = (1.673P_m) \exp(-2.5\sqrt{0.0555 + C_p V_p^2}) \quad \text{for NAS, DSM and CSM} \quad \text{Eq. (26)}$$

where P_m is mean value of the ratio of test and FE results to predictions, V_p is coefficient of variation of mean value of test and FE results to predictions, $C_p = (n^2 - 1) / (n^2 - 3n)$ is correction factor, n is the specimens' number.

The values of other parameters are adopted as follows: $M_m = 1.10$ is mean value of the ratio of actual to nominal material properties, $V_M = 0.10$ is coefficient of variation of the ratio of actual to nominal material properties, $F_m = 1.00$ is mean value of the ratio of the actual to the nominal cross-sectional dimensions, $V_F = 0.05$ is coefficient of variation of the ratio of the actual to the nominal cross-sectional dimensions, $V_Q = 0.21$ is coefficient of variation of load effect.

The reliability indices are found to be 1.14, 1.67, 1.32, 2.58, 2.58 and 2.25 for the EC3, NAS, AS/NZS, DSM, CSM for RHS and CSM for CHS design rules, respectively, see Table 5. The reliability indices for the three existing international design codes and the CSM for CHS are lower than the target value of 2.5, thus suggesting that these design methods are not suitable for the flexural design of cold-formed steel OHS beams. In comparison, both DSM and CSM for RHS methods yields a reliability index exceeding the lower limit of 2.5, and therefore which suggests that both DSM and CSM for RHS approaches are reliable methods for cold-formed steel OHS beams, though they are not specifically calibrated and proposed for OHS beams.

6. Conclusions

This paper provides an experimental and numerical investigation into the strength and behaviour of cold-formed steel oval hollow sections under in-plane bending. In the tests, beam specimens were bent about the minor axis subjected to four-point bending. The specimens failed by flexural buckling. To supplement the experimental programme, a non-linear finite element model was developed and validated against 12 tested specimens. Upon validation, a parametric study was performed using the finite element model with varying cross-sectional geometries, generated additional data on cold-formed steel oval hollow section beams. Thereafter, the newly generated data was used to evaluate the applicability and reliability of five design methods - European code (EC3), the North American Specification (AISI S100-2016), the Australian/New Zealand Standards (AS/NZS 4600:2018), the direct strength method and the continuous strength method. Results show that the nominal moments predicted by the first three existing design codes are unconservative and the reliability indices of these methods are failed to satisfy the target value due to the scattered predictions; whereas, the DSM and the CSM approaches provide more accurate and consistent predictions.

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Figure 1. Geometry of oval hollow section (Zhu and Young, 2011)

Figure 2. Stress-strain curves for the flat part of specimen O1-PB [3]

Figure 3. Set-up of four-point bending test

Figure 4. Moment-curvature curves obtained from tests

a) Test

b) FEA

Figure 5. Comparison of failure modes for specimen O3-PB obtained from experiment and FE modelling

Figure 6. FE model for OHS beams developed in this study

(a) Specimen O1-PB

(b) Specimen O2-PB

(c) Specimen O3-PB

(d) Specimen 500 × 250 × 8.0-B2 (Ref [12])

Figure 7. Comparison of moment-curvature curves obtained from tests and FE modelling

Figure 8. Comparison of design predictions with experimental and numerical results

Figure 9. Normalised bending moment resistance against cross-sectional slenderness

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Table 1. Measured geometries and material properties [3] of specimens

Specimens	D (mm)	h (mm)	W (mm)	t (mm)	E_{flat} (GPa)	E_{curved} (GPa)	$f_{y,flat}$ (MPa)	$f_{y,curved}$ (MPa)	$f_{u,flat}$ (MPa)	$f_{u,curved}$ (MPa)	$\varepsilon_{f,flat}$ (%)	$\varepsilon_{f,curved}$ (%)
O1-PB	120	72	48	1.95	201.9	206.4	358.6	379.2	402.8	415.4	17.5	19.9
O2-PB	42	21	21	2.80	200.2	203.3	430.2	466.4	455.9	485.6	20.0	5.60
O3-PB	30	15	15	1.64	199.1	203.2	432.1	490.9	453.6	518.2	20.9	6.10

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Table 2. Pure bending test results

Specimens	$\bar{\lambda}_p$	M_{exp} (kNmm)	k_{exp} (mm ⁻¹)	Failure mode
O1-PB	0.81	3180	1.30E-05	L
O2-PB	0.18	830	3.79E-04	F
O3-PB	0.22	265	0.000429	F

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Note: L means local buckling failure, F means flexural failure.

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Table 3. Summary of FE validation results of test specimens

Specimens	$\bar{\lambda}_p$	M_{exp} (kNm)	M_{FE} (kNm)	$\frac{M_{FE}}{M_{exp}}$
O1-PB	0.81	3.18	3.05	0.96
O2-PB	0.18	0.83	0.83	1.00
O3-PB	0.22	0.26	0.26	1.00
400 × 200 × 8.0-B1*	0.30	186	180.21	0.97
400 × 200 × 10.0-B1*	0.45	232	225.49	0.97
400 × 200 × 12.5-B1*	0.37	288	264.20	0.92
400 × 200 × 14.0-B1*	0.31	343	336.82	0.98
400 × 200 × 16.0-B1a*	0.29	331	359.52	1.09
500 × 250 × 8.0-B1*	0.71	291	293.06	1.01
400 × 200 × 12.5-B2*	0.38	548	547.56	1.00
400 × 200 × 14.0-B2*	0.32	659	688.96	1.05
500 × 250 × 8.0-B2*	0.70	497	513.40	1.03
Mean				1.00
COV				0.042

Note: * Four-point bending tests conducted by Chan and Gardner [12].

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Table 4. Numerical results generated from parametric study

Specimens	D (mm)	W (mm)	t (mm)	$\bar{\lambda}_p$	Section class	f_{ol} (MPa)	M_{FE} (kNm)
W60T2.0	300	60	2.0	2.61	4	70	5.82
W60T2.4	300	60	2.4	2.18	4	97	7.48
W60T3.0	300	60	3.0	1.74	4	147	10.47
W60T4.0	300	60	4.0	1.31	4	250	16.39
W60T5.0	300	60	5.0	1.04	4	381	23.79
W60T6.0	300	60	6.0	0.87	4	539	31.48
W60T7.0	300	60	7.0	0.75	4	724	39.74
W60T8.0	300	60	8.0	0.65	2	935	47.76
W60T9.0	300	60	9.0	0.58	1	1174	55.43
W60T10.0	300	60	10.0	0.52	1	1438	61.10
W75T1.9	300	75	1.9	2.58	4	71	7.04
W75T2.2	300	75	2.2	2.22	4	94	8.58
W75T2.8	300	75	2.8	1.75	4	145	11.94
W75T3.3	300	75	3.3	1.48	4	195	15.70
W75T3.8	300	75	3.8	1.29	4	250	19.43
W75T4.2	300	75	4.2	1.17	4	299	21.16
W75T5.0	300	75	5.0	0.98	4	409	29.79
W75T6.0	300	75	6.0	0.82	4	570	41.49
W75T7.0	300	75	7.0	0.70	3	756	51.45
W75T10.0	300	75	10.0	0.49	1	1468	77.31
W100T1.7	300	100	1.7	2.56	4	68	7.91
W100T2.0	300	100	2.0	2.18	4	92	10.59
W100T2.5	300	100	2.5	1.74	4	140	14.66
W100T3.3	300	100	3.3	1.32	4	235	21.96
W100T10	300	100	10.0	0.44	1	1614	104.45
W150T1.3	300	150	1.3	2.51	4	63	10.34
W150T1.5	300	150	1.5	2.18	4	82	12.80
W150T1.8	300	150	1.8	1.81	4	114	16.07
W150T2.5	300	150	2.5	1.31	4	207	25.33
W150T4.0	300	150	4.0	0.82	4	480	49.02
W150T5.0	300	150	5.0	0.65	2	709	66.65
W150T6.0	300	150	6.0	0.54	1	975	88.28
W150T7.0	300	150	7.0	0.47	1	1277	107.41
W150T8.0	300	150	8.0	0.41	1	1613	125.92
W150T9.0	300	150	9.0	0.36	1	1954	144.88
W150T10.0	300	150	10.0	0.33	1	2294	158.41

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Table 5. Comparison of predicted design moment resistance with experimental and numerical results

Specimens	$\frac{M_u}{M_{EC}}$	$\frac{M_u}{M_{NAS}}$	$\frac{M_u}{M_{AS/NZS}}$	$\frac{M_u}{M_{DSM}}$	$\frac{M_u}{M_{CSM,RHS}}$	$\frac{M_u}{M_{CSM,CHS}}$
O1-PB	0.98	0.98	0.98	0.97	1.05	1.17
O2-PB	0.95	1.20	1.20	1.20	0.95	0.96
O3-PB	0.98	1.22	1.22	1.21	0.96	1.06
W60T2.0	0.66	0.67	0.67	1.02	1.27	0.77
W60T2.4	0.69	0.69	0.69	0.97	1.17	0.80
W60T3.0	0.74	0.75	0.75	0.95	1.10	0.85
W60T4.0	0.83	0.83	0.83	0.94	1.04	0.95
W60T5.0	0.92	0.92	0.92	0.96	1.05	1.06
W60T6.0	0.98	0.99	0.99	0.96	1.04	1.14
W60T7.0	1.04	1.05	1.05	1.02	1.04	1.21
W60T8.0	1.06	1.09	1.09	1.09	1.00	1.25
W60T9.0	1.03	1.13	1.13	1.13	0.98	1.28
W60T10.0	0.96	1.14	1.14	1.14	0.95	1.26
W75T1.9	0.66	0.67	0.67	1.03	1.29	0.79
W75T2.2	0.68	0.69	0.69	0.98	1.19	0.80
W75T2.8	0.72	0.72	0.72	0.93	1.08	0.84
W75T3.3	0.78	0.78	0.78	0.94	1.07	0.90
W75T3.8	0.81	0.82	0.82	0.93	1.04	0.95
W75T4.2	0.79	0.79	0.79	0.87	0.96	0.92
W75T5.0	0.90	0.91	0.91	0.94	1.02	1.05
W75T6.0	1.01	1.02	1.02	0.99	1.07	1.19
W75T7.0	1.06	1.06	1.06	1.05	1.06	1.24
W75T10.0	0.96	1.14	1.14	1.14	0.96	1.25
W100T1.7	0.61	0.62	0.62	1.00	1.25	0.76
W100T2.0	0.68	0.68	0.68	1.02	1.24	0.83
W100T2.5	0.73	0.73	0.73	0.98	1.14	0.87
W100T3.3	0.79	0.79	0.79	0.93	1.04	0.93
W100T10	0.97	1.14	1.14	1.14	0.96	1.25
W150T1.3	0.68	0.70	0.70	1.21	1.53	0.90
W150T1.5	0.71	0.72	0.72	1.18	1.45	0.93
W150T1.8	0.73	0.73	0.73	1.10	1.30	0.94
W150T2.5	0.79	0.80	0.80	1.01	1.15	0.99
W150T4.0	0.90	0.90	0.90	0.93	1.01	1.09
W150T5.0	0.94	0.96	0.96	0.96	0.98	1.14
W150T6.0	0.94	1.06	1.06	1.06	0.97	1.22
W150T7.0	0.95	1.11	1.11	1.11	0.96	1.24
W150T8.0	0.97	1.15	1.15	1.15	0.97	1.25
W150T9.0	1.00	1.18	1.18	1.18	0.97	1.26
W150T10.0	0.98	1.17	1.17	1.17	0.95	1.23
Mean, P_m	0.86	0.91	0.91	1.04	1.08	1.04
COV, V_P	0.155	0.206	0.206	0.093	0.128	0.166
Resistance factor, ϕ	1.00	0.90	0.95	0.90	0.90	0.90
Reliability index, β	1.14	1.67	1.32	2.58	2.58	2.25

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