Thin-Walled Structures 187 (2023) 110660

Contents lists available at ScienceDirect

Thin-Walled Structures

journal homepage: www.elsevier.com/locate/tws

Full length article

Shear capacity of the cold-formed steel beam to column welded moment connection using clip-angle and flange-cleat

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ARTICLE INFO

Keywords:
Welded moment connection
Welded shear connection
CFS beam-to-column connection
Welded clip-angle connector
Reliability analysis

ABSTRACT

The current research endeavours to evaluate the shear performance of the cold-formed steel (CFS) welded moment connection between beam-to-column with 36 laboratory tests. The web portions of the beam and column were connected by CFS welded clip-angle to form a CFS welded shear connection. Subsequently, it is converted into a welded moment connection by including a flange cleat between the flange portions. The shear capacity of the welded shear connection increases by an average of 67% after the inclusion of flange cleats, which is quantified using a performance ratio variable. This research presents two shear equations for the CFS welded moment connection (i) a new empirical shear equation; (ii) a new shear equation representing the shear strength of the moment connection as a function of the shear strength of the shear connection. The variability of the shear performance of welded moment and welded shear connections is expressed with force versus displacement plots and failure modes of the clip angles. The failure modes observed in the clip-angle in both welded moment and welded shear connections are (i) Local buckling and (ii) Distortional buckling. The shift in failure modes of some of the clip-angle in the WM connection (Local buckling) and the WS connection (Distortional buckling) indicates the effectiveness of flange-cleat in resisting due to free twisting of the beam because of load offset from the shear center. The design factors were also determined for the LRFD, LSD, and ASD methods by performing reliability studies.

1. Introduction

The cold-formed steel (CFS) sections are widely accepted as a construction material due to their cost-effectiveness, high strength-to-weight ratio, assured quality, ease of fabrication, transportation, and erection. Due to their thin-walled behavior, the existing design standards and methods for the traditional thick-walled hot-rolled steel (HRS) sections are not applicable to CFS sections. Unlike the limited HRS section shapes, numerous sections are possible with CFS due to the ease of fabrication and controlling the profile quality at room temperature. The CFS sections' profile shapes most commonly used are plain C channels, lipped C channels, and Z-shaped cross-sections.

The CFS sections will be subjected to additional torsional effect along with transverse loading as the transverse load acts away from the shear center (Fig. 1). CFS sections are typically thin open cross-sections with less torsional stiffness compared to HRS members or members with closed sections. The torsional effects associated with the thin-walled behavior of the CFS section may significantly affect the load-carrying capacity of the connections, specifically the beam-to-column connection. Hence, this free twisting of CFS members must be restrained to improve the strength of the CFS members and reduce the adverse effects on the CFS connections between the members.

Research into the CFS member's connection behavior and design were more focused in the last two decades as the connection failures are typically catastrophic. Lennon et al. [1] conducted laboratory tests to understand the shear behavior of the thin gauge steel connections using press joining, self-piercing rivets, pop rivets, and self-tapping screws. The ductility of the self-tapping screw connections was high compared to other connections in Lennon et al. study [1]. The behavior and design of the apex joints in the CFS structural system were experimentally investigated by Lim and Nethercot[a] [2], and the concept of the moment capacity of the connection is less than the member moment capacity was explained and addressed. In the companion technical note, Lim and Nethercot[b] [3] predicted the initial bolt-hole elongation stiffness to determine the overall stiffness of the bolted moment connection. The failure of the CFS members adjoining the connection was observed in a direct CFS member connection through the fasteners [3]. The shear capacity of the CFS web cleats in beam-to-beam, and beam-to-column configurations were studied by Chung and Lawson [4] with different positioning of web cleats. This study suggested replacing the CFS web cleats over typical HRS angles in a steel building as the considerable shear resistance found in the CFS web cleat. The experimental investigation of the CFS floor assembly

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https://doi.org/10.1016/j.tws.2023.110660
Received 29 October 2022; Received in revised form 5 February 2023; Accepted 24 February 2023
Available online 27 April 2023
0263-8231/

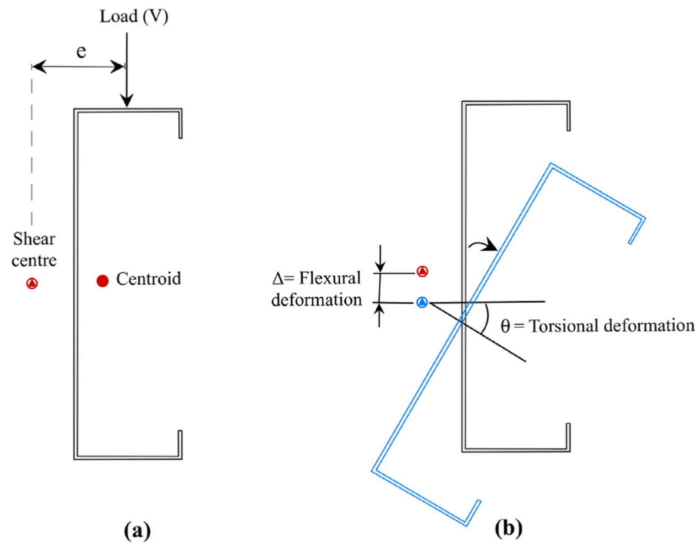

Fig. 1. Twisting of CFS members: (a) Load offset; (b) Torsional deformation.

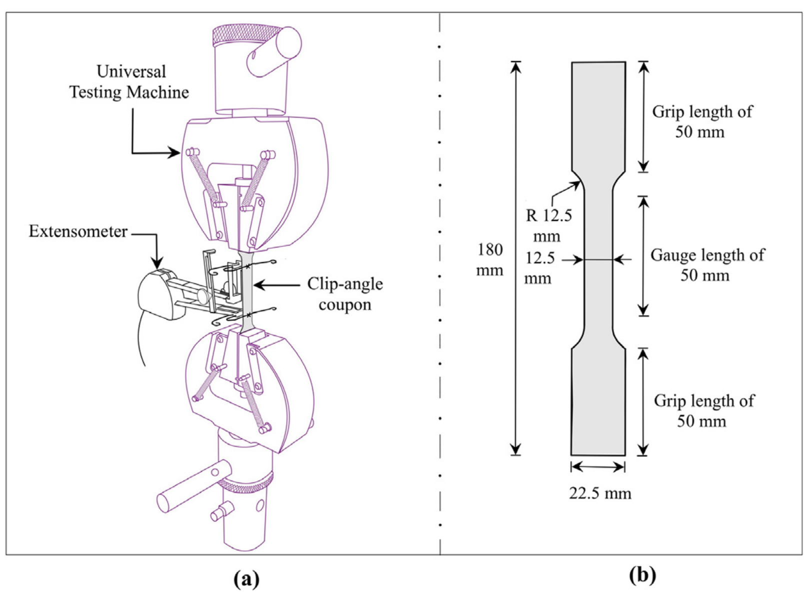

Fig. 2. Material test: (a) Test set-up on UTM; (b) Geometric layout of the coupon.

using a screwed clip-angle as a bearing stiffener was conducted by S. R. Fox [5]. The shear performance of the CFS clip-angle was found to be satisfactory [4,5]. The failure mechanism of an HRS clip-angle welded to an HRS coped beam was analyzed by Yam et al. [6,7] through experimental investigation and numerical validation, respectively. Considerable research into the HRS connections was done by Gong [8–11]. The experimental outcomes of Gong's [8] research work addressed the fabrication issues with the double-angle shear connection in hollow steel columns and suggested recommendations. Gong's [9] research further explored the shear strength of the welded shear tab to a flexible column. In addition to the shear strength determination, Gong [10,11] studied the effect of the secondary moment on the shear capacity and application of moment equations for shear connections, respectively. A large amount of research on the CFS clip-angle with screw fasteners was conducted by Yu et al. [12,13] for tension, compression, and shear loads by Yu et al. [14] as the clip-angle is primarily subjected to these forces. The research works that are exclusively focused on the behavior and performance of the CFS clip-angle against shear load are Yu et al. [14], Natesan and Madhavan [15,16], and Mallepogu and Madhavan [17]. The shear resistance of the screwed clip-angle was studied by Yu et al. [14], and a design method was proposed to evaluate the same. The shear capacity of the clip-angle with the 2-bolted and 3-bolted configurations in a beam-to-column configuration was studied by Natesan and Madhavan [15] and Natesan et al. [16], respectively. Recently, Mallepogu and Madhavan [17] investigated the shear behavior and suggested design recommendations for a welded clip-angle shear connection (WS) between the CFS beam and column. The above list of papers investigated the capacity of the shear connection with the clip-angle that connected the web portions of the CFS members. In the present study, the authors endeavor to improve the shear capacity of the CFS beam-to-column connection by strengthening it using a flange cleat (connects the flange portions) along with a clip-angle (connects the web portions). In addition to the shear strength increment, the flange-cleat will resist the free twisting of the CFS beam member due to the load offset (e), as shown in Fig. 1.

The clip-angle in the welded moment connection (WM) assembly failed in a mechanism not predicted by a welded shear connection model developed by Mallepogu and Madhavan [17]. Also, the existing design equation for the welded shear connection [17] was inefficient for the present welded moment connection. Therefore, the authors

Table 1
Mechanical properties of the clip-angle and flange-cleat.

Component	Thickness (mm)	Sample number	Yield strength f_y (N/mm^2)	Ultimate tensile strength f_u (N/mm^2)	Elongation (%)	Young's Modulus, E (N/mm^2)
Clip-angle	1.5	1	404.21	474.07	49.3	200483
		2	402.85	477.29	45.5	201925
		3	393.08	441.24	40.5	199062
		Mean	400.05	464.20	45.10	200490
	2	1	274.11	339.71	34	200701
		2	276.77	343.68	33.5	200664
		3	275.15	338.1	29.01	203413
		Mean	275.34	340.50	32.17	201593
	2.5	1	433.61	457.71	38.07	200991
		2	432.67	475.82	36.51	199367
		3	444.32	475.73	39.9	200768
		Mean	436.87	469.75	38.16	200375
Flange cleat	2	1	242.18	402.37	32.01	200000
		2	237.48	395.37	29.07	200000
		3	233.46	388.93	31.01	200000
		Mean	237.71	395.56	30.70	200000

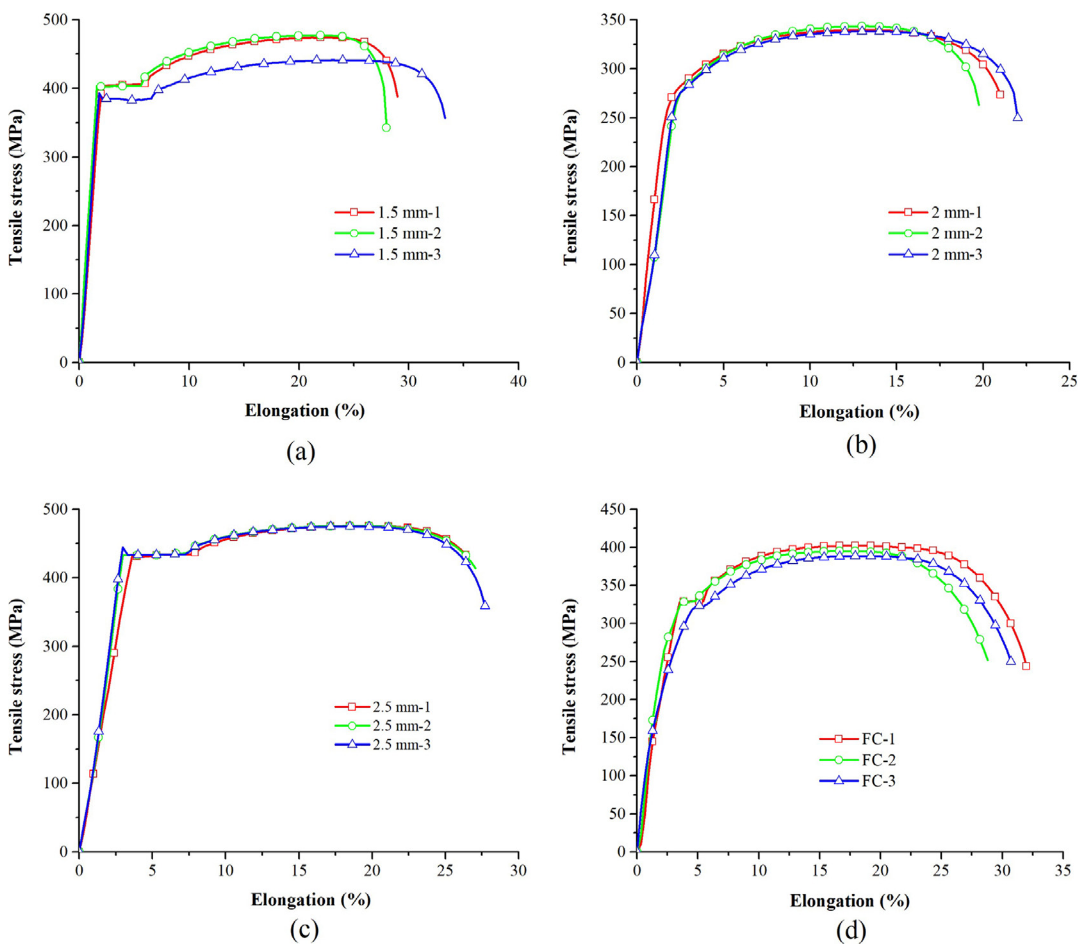

Fig. 3. Coupon test data: (a) 1.5 mm clip-angle; (b) 2 mm clip-angle ; (c) 2.5 mm clip-angle; (d) 2 mm flange-cleat.

developed design models for the shear behavior of the CFS welded moment connection because no behavioral and design models currently exist in the literature. Along with a new empirical shear equation explicitly suggested for the present welded moment connection, a new shear equation was also developed by improvising the existing design equation [17] of the welded shear connection. This improvised shear equation is applicable for both welded shear (WS) connection [17] and the test results from welded moment (WM) connection. Both of them are in good agreement with the suggested empirical equation of the welded moment connection.

Research objectives:

- To study the shear behavior of the welded moment connection (WM) connection through the laboratory tests
- To propose a design shear equation and classify the failure modes of the WM connection
- To evaluate the shear capacity increment of the WM connection over the WS connection
- To evaluate the design factors for the WM connection from the experimental data.

2. Coupon tests

The material properties of the cold-formed steel (CFS) clip-angle and CFS flange cleat were determined from the tensile coupon tests on a Universal Testing Machine (UTM) of 30 kN capacity. As per ASTM E8/E8M-13a [18] and Huang and Young [19] recommendations, the coupons were prepared and tested with a 0.01 mm/sec loading rate. The determined material properties are tabulated in Table 1, and test results are illustrated in Fig. 3. The Young's modulus of the coupons was determined from the extensometer reading instrumented over a gauge length of 50 mm, as shown in Figs. 2(a) and 2(b).

3. Experimental program

3.1. labeling of test specimens

All the test specimens were labeled as follows

t-A-D-(WM or WS)-R

Where,

t = Thickness of the clip-angle

A = Total width of outstanding leg of the clip-angle

D = Depth of the clip-angle

WM = welded moment connection. It refers to the moment connection between the beam and column with CA and FC.

WS = welded shear connection. It refers to the shear connection between the beam and column with CA alone.

R = Repeat test.

3.2. Specimen preparation

As shown in Fig. 4, the test specimen consists of an 8 mm thick Hot-Rolled Steel (HRS) beam of 1000 mm length, 2 mm thick lipped C-section CFS columns of 400 mm height, 90° bend CFS L-shape connectors between the beam and column web portions (referred to as clip-angle) and 90° bend CFS L-shape connector between the beam and column flange portions (referred as flange-cleat).

As shown in Fig. 4 for both clip-angle (CA) and flange-cleat (FC) connectors, the leg connected to the column is referred to as the outstanding leg, and the leg connected to the beam is referred to as the loading leg. The outstanding leg of the CA is fillet welded to a 2 mm thick CFS column while the loading leg is bolted to an 8 mm thick HRS beam. M4.6 grade 12 mm diameter bolts were used in the outstanding leg of the CA following the minimum pitch and edge distance specifications of AISI [20]. A 2.5 mm (ω) fillet weld was used

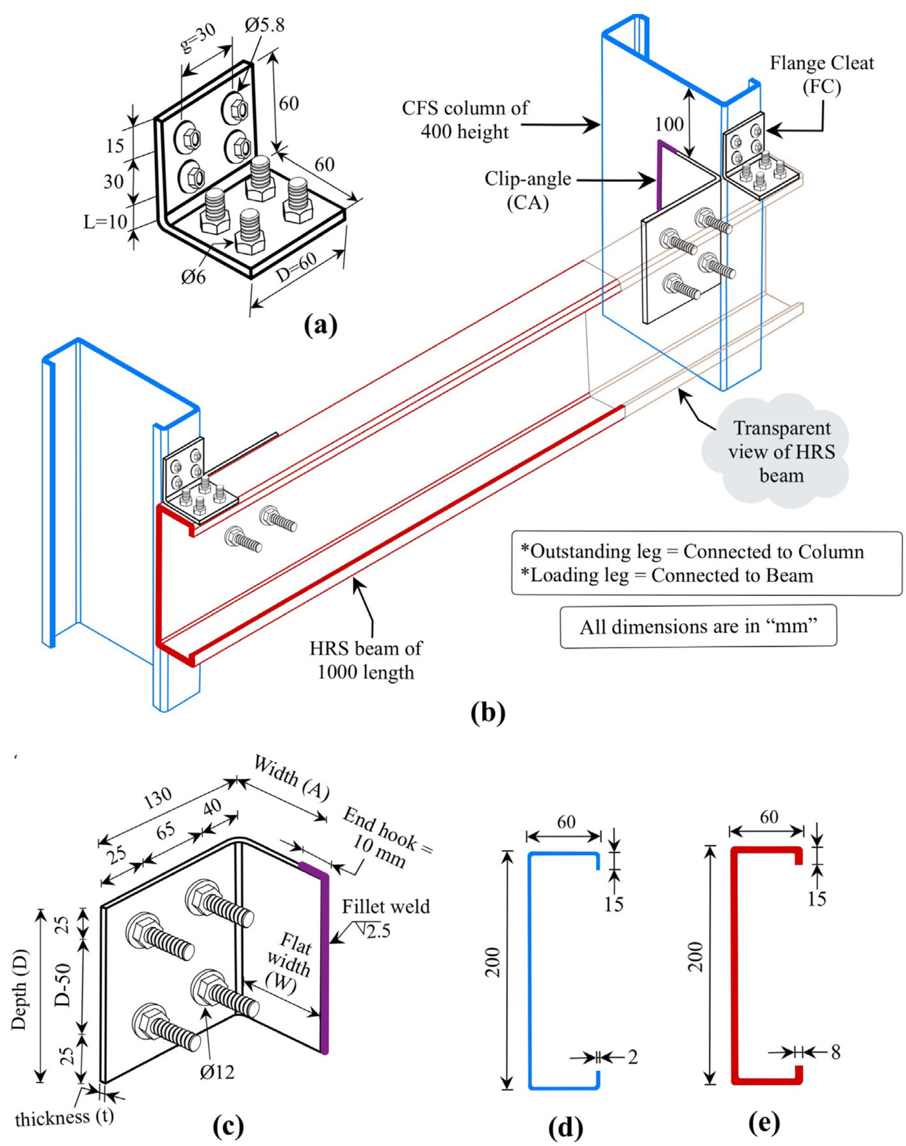

Fig. 4. Test specimen: (a) Flange-cleat; (b) Beam-to-column specimen; (c) Clip-angle; (d) Cross-section of CFS column; (e) Cross-section of HRS beam.

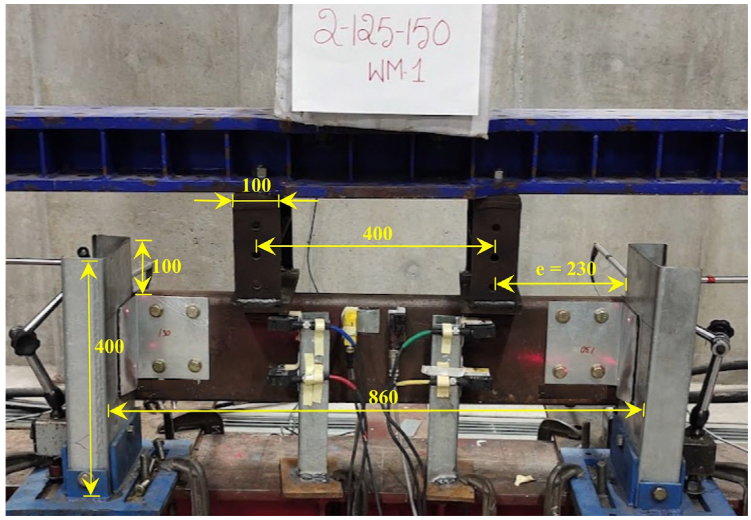

Fig. 5. The experimental setup of the test specimen.

Table 2
Experimental results.

S. No.	Clip-angle configuration: t-A-D-wm	Ultimate strength of clip-angle, $V_{ultimate}$ (kN)	Failure mode	Aspect ratio (W/D)
1	1.5-65-100-wm	21.44	LB	0.61
2	1.5-95-100-wm	14.80	LB	0.91
3	1.5-125-100-wm	12.44	LB	1.21
4	1.5-65-150-wm	33.89	LB	0.41
5	1.5-95-150-wm	21.05	LB	0.61
6	1.5-125-150-wm	15.94	LB	0.81
7	1.5-65-180-wm	45.50	DB	0.34
8	1.5-95-180-wm	27.45	LB	0.51
9	1.5-125-180-wm	17.25	LB	0.67
10	2-65-100-wm	25.59	LB	0.60
11	2-95-100-wm	17.94	LB	0.90
12	2-125-100-wm	13.54	LB	1.20
13	2-65-150-wm	39.05	DB	0.40
14	2-95-150-wm	28.75	LB	0.60
15	2-125-150-wm	18.58	LB	0.80
16	2-65-180-wm	50.08	DB	0.33
17	2-95-180-wm	35.66	LB	0.50
18	2-125-180-wm	23.79	LB	0.67
19	2.5-65-100-wm	31.27	LB	0.59
20	2.5-95-100-wm	24.28	LB	0.89
21	2.5-125-100-wm	18.91	LB	1.19
22	2.5-65-150-wm	67.21	DB	0.39
23	2.5-95-150-wm	48.40	LB	0.59
24	2.5-125-150-wm	30.70	LB	0.79
25	2.5-65-180-wm[a]	96.99	LB	0.33
26	2.5-95-180-wm	60.05	LB	0.49
27	2.5-125-180-wm	39.54	LB	0.66
28	1.5-125-100-wm-R	11.55	LB	1.21
29	2-125-100-wm-R	15.04	LB	1.20
30	2-65-150-wm-R	43.44	DB	0.40
31	2-95-150-wm-R	29.14	LB	0.60
32	2-125-150-wm-R[a]	20.02	LB	0.80
33	2-125-180-wm-R	25.09	LB	0.67

[a]Specimens with the outstanding leg of flange-cleat bolted to a column, while all other specimens are screwed to the column. LB = Local buckling, DB = Distortional buckling.

in all the specimens as per AWS [21] to preclude weld failure. E6013 electrodes were used with an end-hook length of 10 mm (≤4ω) as per AWS [21]. The loading leg of the FC is bolted to the HRS beam with 6 mm diameter bolts while the outstanding leg is attached to the CFS column with 5.8 mm diameter screws. In the present study, the test set-up ensured the failure would occur in the outstanding leg of the CA while precluding the failure of the fastener (M12 bolts and 2.5 mm fillet weld). Also, the flange-cleat is kept constant in all the test specimens in terms of geometric variables (dimensions of FC, number of bolts and screws, diameter of bolts and nuts) and material variables (grade of FC, grade of bolts and screws).

3.3. Laboratory testing

As illustrated in Fig. 5, a four-point bending test was conducted on all the test specimens with a 400 mm center-to-center distance

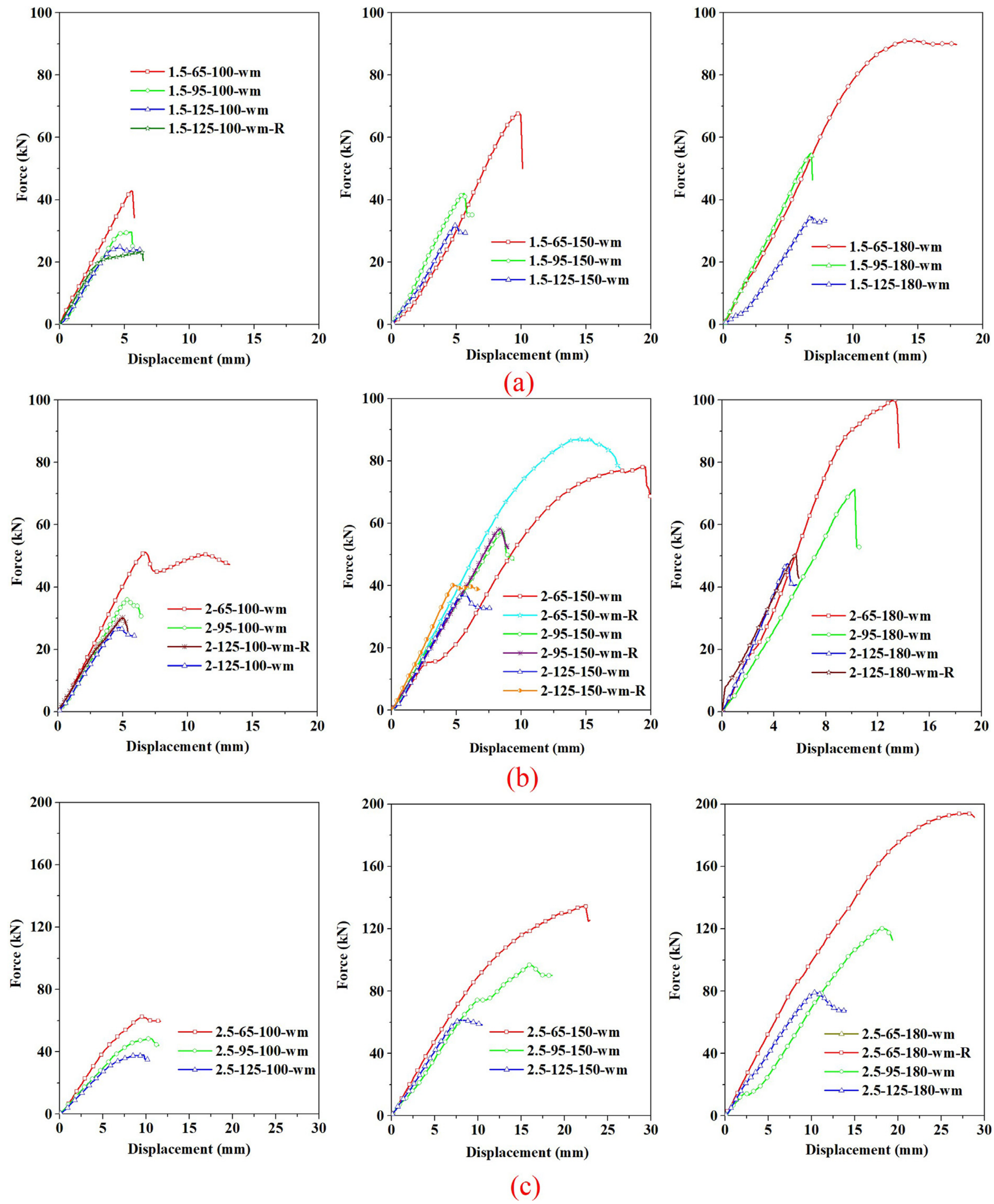

Fig. 6. Force vs displacement plots: (a) 1.5 mm CA; (b) 2 mm CA; (c) 2.5 mm CA.

between the loading points. An eccentricity of 230 mm was maintained between the center of the loading point and the column face. A 100 mm wide HRS fixture was used to apply the patch load on the specimens with a loading rate of 0.01 mm/sec with a servo-hydraulic controlled actuator of 250 kN capacity. The repeat tests were performed to check the consistency of the test results. Also, these repeat tests were found to have good agreement with initial tests with <10% variation in the ultimate strength (refer to Table 2). Hence these test data were also considered in the development of a new design shear equation. The experimental outcomes are tabulated in Table 2 and illustrated in Fig. 6.

In Table 2, except 2.5-65-180-wm specimen, all the remaining specimens consist of a 2 mm CFS column, and the outstanding leg of the FC screwed to the CFS column. In the 2.5-65-180-wm specimen, the

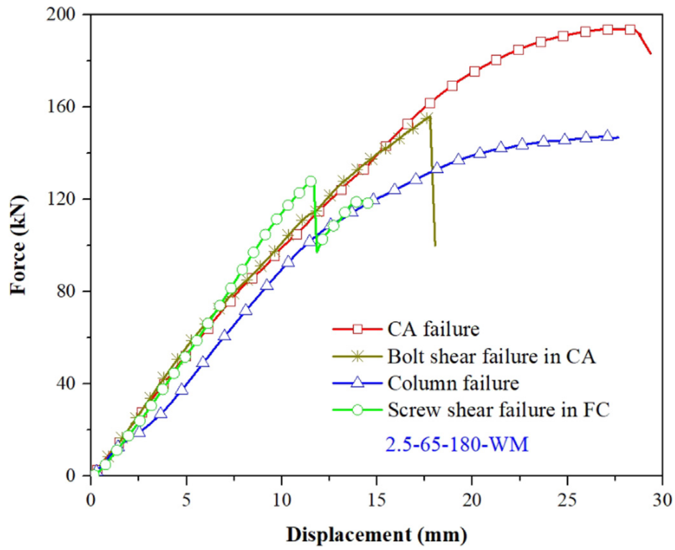

Fig. 7. Ancillary tests for 2.5-65-180-WM specimen.

2 mm CFS column was replaced with the 8 mm HRS column, and the 5.8 mm screws were replaced with the 6 mm bolts, as the former failed due to column twisting and screw shear, respectively.

Except for specimen 2.5-65-180-wm, all the other test samples failed due to either local buckling or distortional buckling of the clip-angle. While the FC also experienced excessive deformation due to the twisting of the HRS beam, the ultimate failure of the test specimen was due to CA failure.

3.4. Unexpected failures

A series of unexpected failures occurred during the testing of specimen 2.5-65-180-wm. To begin with, screw shear failure in the flange cleat and failure due to twisting of the CFS column occurred. The screws were replaced with M6 bolts to prevent screw shear failure, although the failure now shifted to the CFS column. Now, the CFS column was replaced with a 12 mm thick built-up HRS column since the objective is to study the behavior of the clip angle connector until the load reaches the ultimate. This time the failure occurred due to bolt (12 mm diameter M4.6 grade) shear in the loading leg of the clip-angle. To preclude the bolt shear failure the bolt grade was replaced from M4.6 to M10.9; this led to distortional buckling failure of the clip-angle. This desirable clip angle failure result was used in evaluating the shear capacity of the clip angle (refer to Fig. 7).

3.5. Screwed and bolted FC (flange cleat)

The flange cleat made the welded clip-angle reach the same strength in shear with either the outstanding leg of FC screwed or bolted to the CFS column as shown in Figs. 8(a) and 8(b) respectively. The out-of-plane movement of the CFS column has no significant effect on the shear strength of the welded clip angle [17]. Hence, it can be deduced that the shear strength of the welded CA is unaltered in both the CFS column and the HRS column in case of 2.5-65-180-WM specimen.

Ancillary tests were conducted using the flange cleat bolted and screwed to the column in 2-125-150-WM-R and 2-125-150-WM specimens, respectively. The variation in shear strength of the flange-cleat with its outstanding leg bolted and screwed to the column is marginal (<10%) as well as failure mode and load versus displacement response of these specimens were similar, as shown in Figs. 9 and 8, respectively.

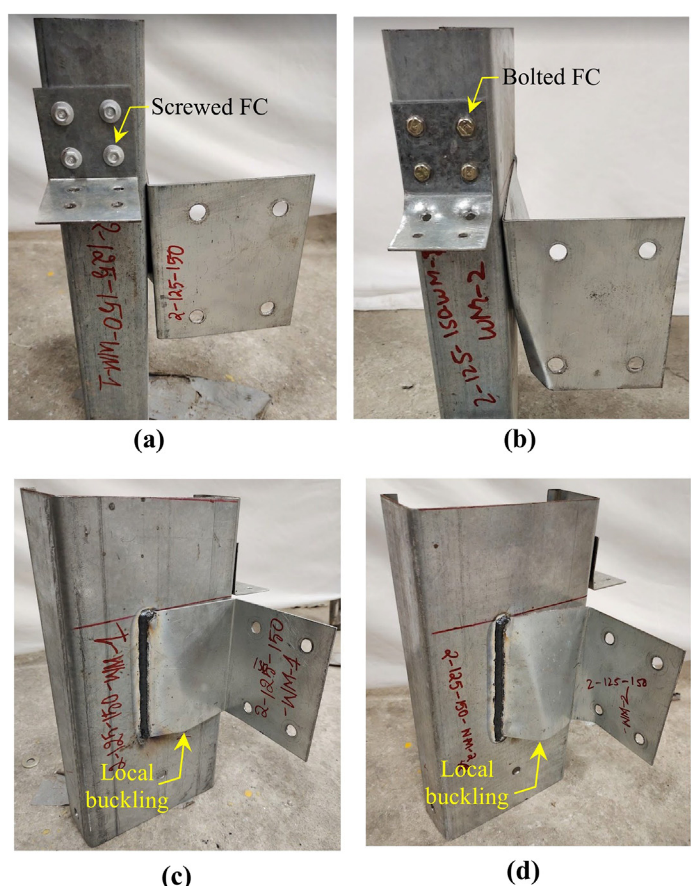

Fig. 8. Specimen 2-125-150-wm: (a) FC screwed; (b) FC bolted; (c) Local buckling in screwed FC specimen; (d) Local buckling in bolted FC specimen.

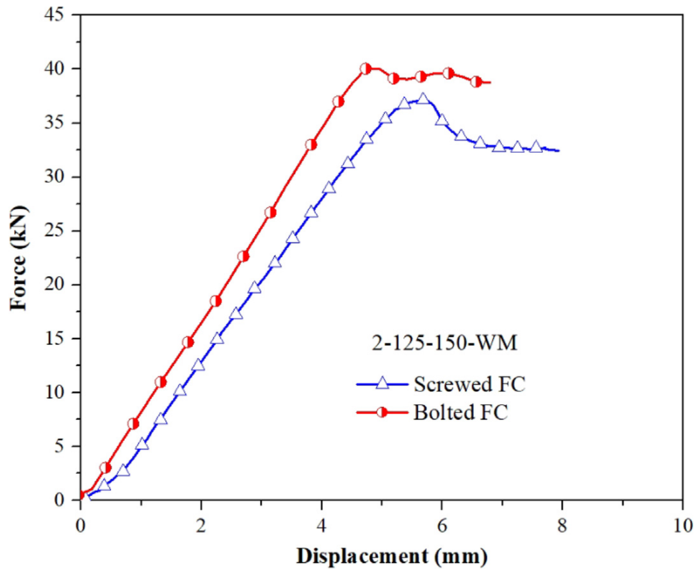

Fig. 9. Plots of ancillary tests.

4. Analysis of results

4.1. Development of an empirical equation

Using the curve-fitting method, an empirical equation was developed to determine the shear strength of the welded moment connection formed with welded clip-angle and screwed flange-cleat as illustrated in Fig. 10 and Table 3.

The suggested empirical shear equation for the welded moment connection is

$$V_{WM-emp} = 0.457\,(\lambda)^{-0.8}\,V_y \tag{1}$$

The suggested equation exhibits good agreement with the experimental results with a mean of unity, a standard deviation of 0.16, and a coefficient of variation of 0.16.

4.2. Comparing with existing design methods

The shear capacity of the welded clip-angle in a shear connection can be obtained from the following equation suggested by Mallepogu and Madhavan [17]. Its application is limited to welded shear connections only.

$$V_{WS} = 0.275\,(\lambda)^{-0.8}\,V_y \tag{2.1}$$

In addition, the proposed empirical shear equation (Eq. (1)) in Section 4.1 is also applicable only for the welded moment connection. The designers often have to rely on many equations for determining the shear strength of the welded clip-angle in a shear connection [17] and moment connection. Therefore, the authors propose a new design method that is applicable for both moment and shear connections by modifying the existing design method for the welded shear clip-angle by Mallepogu and Madhavan [17]. In this approach, the shear strengths of the welded moment connection are quantified in terms of the shear

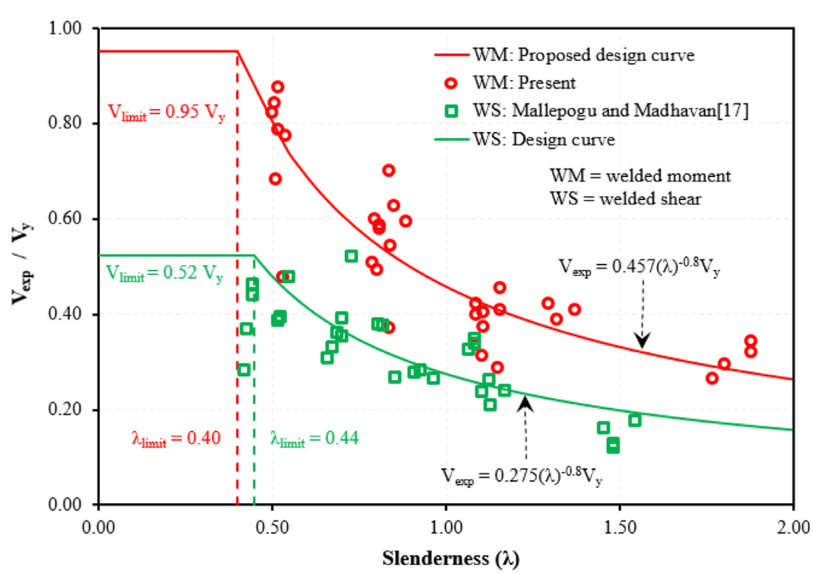

Fig. 10. Empirical equations for WM and WS connections.

Table 3
Empirical shear equation for Welded Moment (WM) connection.

S. No.	Clip-angle configuration: t-A-D-wm	Ultimate strength, V_{exp} (kN)	Slenderness ratio (λ)	Yield shear strength, V_y (kN)	Empirical shear strength, V_{emp-wm} (kN)	V_{exp}/ V_{emp-wm}
1	1.5-65-100-wm	21.44	0.88	36.00	18.17	1.18
2	1.5-95-100-wm	14.80	1.37	36.00	12.79	1.16
3	1.5-125-100-wm	12.44	1.87	36.00	9.96	1.25
4	1.5-65-150-wm	33.89	0.85	54.00	28.16	1.20
5	1.5-95-150-wm	21.05	1.32	54.00	19.82	1.06
6	1.5-125-150-wm	15.94	1.80	54.00	15.43	1.03
7	1.5-65-180-wm	45.50	0.83	64.80	34.29	1.33
8	1.5-95-180-wm	27.45	1.29	64.80	24.14	1.14
9	1.5-125-180-wm	17.25	1.77	64.80	18.79	0.92
10	2-65-100-wm	25.59	0.54	33.00	24.80	1.03
11	2-95-100-wm	17.94	0.84	33.00	17.35	1.03
12	2-125-100-wm	13.54	1.15	33.00	13.47	1.01
13	2-65-150-wm	39.05	0.52	49.50	38.44	1.02
14	2-95-150-wm	28.75	0.81	49.50	26.89	1.07
15	2-125-150-wm	18.58	1.11	49.50	20.87	0.89
16	2-65-180-wm	50.08	0.51	59.40	46.81	1.07
17	2-95-180-wm	35.66	0.79	59.40	32.75	1.09
18	2-125-180-wm	23.79	1.09	59.40	25.42	0.94
19	2.5-65-100-wm	31.27	0.53	65.40	49.77	0.63
20	2.5-95-100-wm	24.28	0.83	65.40	34.61	0.70
21	2.5-125-100-wm	18.91	1.15	65.40	26.78	0.71
22	2.5-65-150-wm	67.21	0.51	98.10	77.15	0.87
23	2.5-95-150-wm	48.40	0.80	98.10	53.64	0.90
24	2.5-125-150-wm	30.70	1.10	98.10	41.51	0.74
25	2.5-65-180-wm	96.99	0.50	117.72	93.95	1.03
26	2.5-95-180-wm	60.05	0.78	117.72	65.33	0.92
27	2.5-125-180-wm	39.54	1.08	117.72	50.55	0.78
28	1.5-125-100-wm-R	11.55	1.87	36.00	9.96	1.16
29	2-125-100-wm-R	15.04	1.15	33.00	13.47	1.12
30	2-65-150-wm-R	43.44	0.52	49.50	38.44	1.13
31	2-95-150-wm-R	29.14	0.81	49.50	26.89	1.08
32	2-125-150-wm-R	20.02	1.11	49.50	20.87	0.96
33	2-125-180-wm-R	25.09	1.09	59.40	25.42	0.99
					Mean	1.00
					Std. deviation	0.16
					COV	0.16

strength of the welded shear connection [17] using a rigidity factor (β). The rigidity factor represents the additional strength derived from moment connection due to the inclusion of flange cleat.

Most of the shear strength is derived from the clip angle in both moment and shear connections. The flange cleat in moment connection ensured that the clip-angle reached its full strength in shear by minimizing or nullifying the beam twisting effect on the clip-angle and strengthening the beam and column flanges. The flange-cleat converts the shear connection between the beam and column [17] into a moment connection (present) by incorporating some rigidity in the connection by connecting the flange portions. This rigidity (β) is quantified in terms of known variables of flange-cleat and clip-angle in the present study.

The shear strength increment in the WM connection is assessed in terms of geometric properties (W/t = width-to-thickness ratio, D/t = depth-to-thickness), material properties of the both CA and FC. A proportional trend was observed between the WM connection ultimate shear strength and yield strength (f_y) and the square root of the D/t ratio of the clip-angle, while a declining trend was noticed between the ultimate shear strength and W/t ratio, as illustrated in Fig. 11.

According to the observation of the trends in Fig. 11, the clip-angle coefficient (X_{CA}) is defined as

$$X_{CA} = \frac{\sqrt{\frac{D}{t_{CA}}}}{\left(\frac{W}{t_{CA}}\right)\left(\frac{f_{y-CA}}{275}\right)^{0.65}} = \frac{\sqrt{Dt_{CA}}}{W\left(\frac{f_{y-CA}}{275}\right)^{0.65}}$$

$$X_{CA} = \frac{\sqrt{Dt_{CA}}}{W\left(\alpha_{CA}\right)^{0.65}} \tag{2.2}$$

Where,

$$\text{Depth factor} = \sqrt{\frac{\text{Depth of CA}}{\text{Thickness of CA}}} = \sqrt{\frac{D}{t_{CA}}} \tag{2.3}$$

$$\text{Width factor} = \frac{\text{Flat width of the welded CA}}{\text{Thickness of CA}} = \frac{W}{t_{CA}} \tag{2.4}$$

$$\text{Material factor}(\alpha_{CA}) = \frac{\text{Yield strength of CA in MPa}}{275} = \frac{f_{y-CA}}{275} \tag{2.5}$$

The value of 275 in the denominator corresponds to the commonly available and the lowest steel grade of the clip-angle used in this study.

Similarly, flange-cleat coefficient is defined as follows

$$X_{FC} = \frac{\sqrt{\frac{g}{t_{FC}}}}{\left(\frac{L}{t_{FC}}\right)\left(\frac{f_{y-FC}}{275}\right)^{0.65}} = \frac{\sqrt{gt_{FC}}}{L\left(\frac{f_{y-FC}}{275}\right)^{0.65}}$$

$$X_{FC} = \frac{\sqrt{gt_{FC}}}{L\left(\alpha_{FC}\right)^{0.65}} \tag{2.6}$$

Where,

g = Gauge distance between the fasteners of the flange-cleat

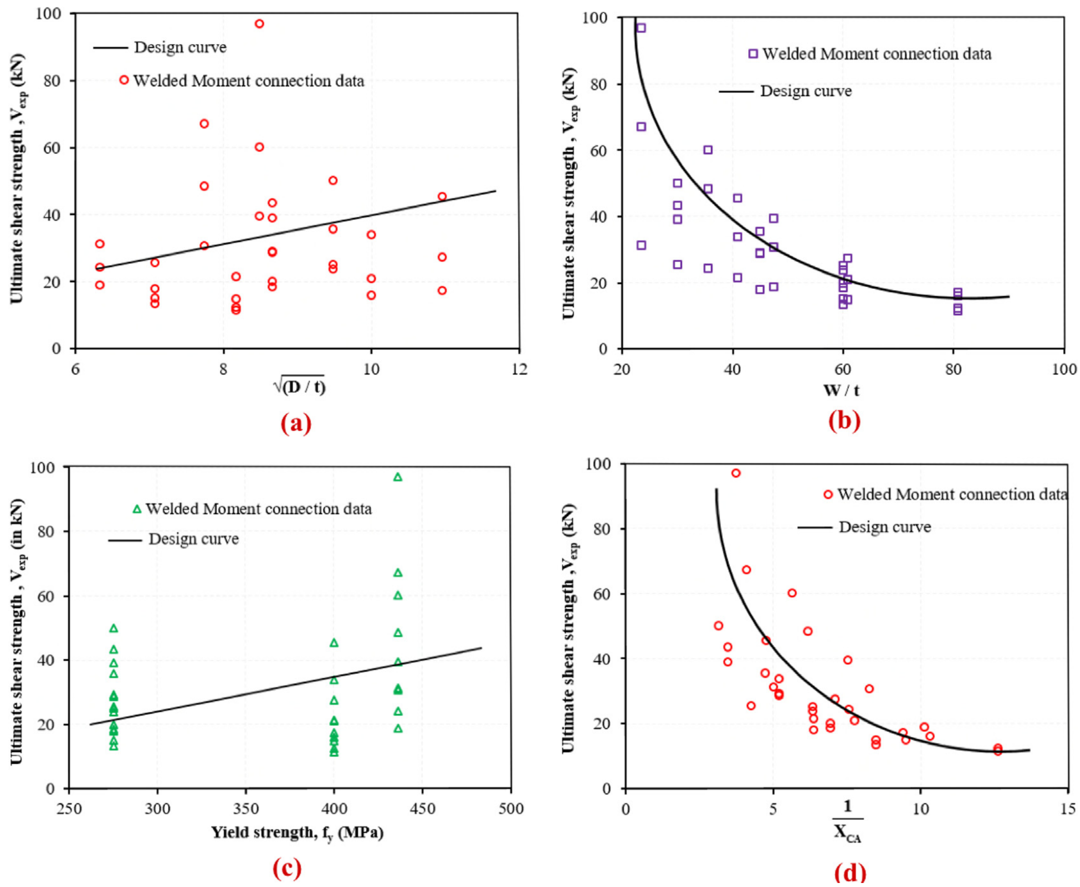

Fig. 11. Trendline plots: (a) shear strength vs $\sqrt{D/t}$; (b) shear strength vs W/t; (C) shear strength vs f_y; (d) shear strength vs $1/X_{CA}$.

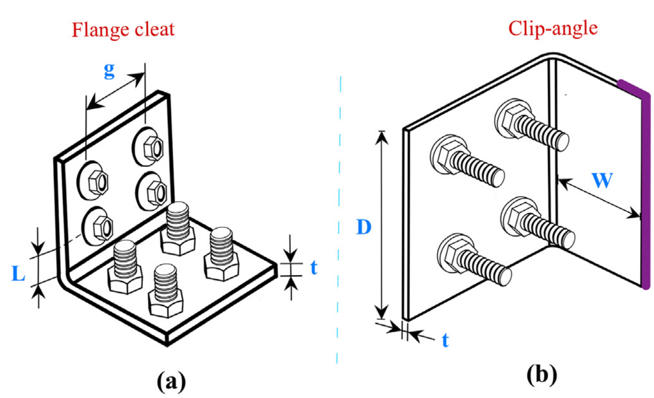

Fig. 12. Design geometric variables of (a) Flange-cleat; (b) Clip-angle.

L = Flat length of flange cleat, which is the distance between the inner fold line to the nearest bolt center-line

t_{FC} = Thickness of the flange cleat

$$\text{Material factor } (\alpha_{FC}) = \frac{\text{Yield strength of FC in MPa}}{275} = \frac{f_{y-FC}}{275} \quad (2.7)$$

The increase in shear strength due to the addition of the flange-cleat to a welded clip-angle shear connection (refer to Fig. 12) is now determined using the rigidity coefficient (β)

$$\beta = A\left(\frac{X_{FC}}{X_{CA}}\right)^{B} \quad (2.8)$$

A and B are the coefficients that correspond to the welded moment configuration. From the statistical analysis of the present experimental data, the values of A and B are evaluated as 0.48 and 0.2.

Therefore, the new shear strength equation for the welded moment connection (V_{WM}) is given by

$$V_{WM} = V_{WS}(1+\beta) \quad (3)$$

$$V_{WM} = 0.275\,(\lambda)^{-0.8}\,V_y(1+\beta) \quad (4)$$

V_{WS} = Shear strength of the CA in welded shear connection

$$\text{Rigidity coefficient } (\beta) = 0.48\left(\frac{X_{FC}}{X_{CA}}\right)^{0.2} \quad (5.1)$$

$$X_{FC} = \frac{\sqrt{t_{FC} \times g}}{L\alpha_{FC}^{\ 0.65}} \quad (5.2)$$

$$X_{CA} = \frac{\sqrt{t_{CA} \times D}}{W\alpha_{FC}^{\ 0.65}} \quad (5.3)$$

$$\text{Slenderness ratio } (\lambda) = \sqrt{\frac{V_y}{V_{cr}}} \quad (5.4)$$

$$\text{Yield shear strength } (V_y) = 0.6f_y(t \times D) \quad (5.5)$$

$$\text{Critical shear strength } (V_{cr}) = f_{cr}(t \times D) \quad (5.6)$$

$$\text{Critical buckling stress } (f_{cr}) = \frac{k\pi^2 E}{12\,(1-\mu^2)}\left(\frac{t}{D}\right)^2 \quad (5.7)$$

$$\text{Elastic buckling coefficient (k)} \ = 2.569\left(\frac{W}{D}\right)^{-2.202} \quad (5.8)$$

$$\text{Aspect ratio of CA} \ = \frac{\text{Flat width(W)}}{\text{Depth(D)}} \quad (5.9)$$

Table 4
New shear equation for Welded Moment (WM) connection.

S. No.	Clip-angle configuration: t-A-D-wm	V_{exp} (kN)	Depth factor $=\sqrt{\frac{D}{t_{CA}}}$	Shear connection strength, V_{n-ws} (kN)	X_{CA}	X_{FC}	β	Moment connection shear strength, V_{n-wm} = Vn − ws(1+β) (kN)	$\frac{V_{exp}}{V_{n-wm}}$
1	1.5-65-100-wm	21.44	8.16	10.93	0.88	0.82	0.67	18.24	1.18
2	1.5-95-100-wm	14.80	8.16	7.69	1.37	0.82	0.72	13.26	1.12
3	1.5-125-100-wm	12.44	8.16	5.99	1.87	0.82	0.77	10.58	1.18
4	1.5-65-150-wm	33.89	10.00	16.94	0.85	0.82	0.64	27.82	1.22
5	1.5-95-150-wm	21.05	10.00	11.93	1.32	0.82	0.70	20.22	1.04
6	1.5-125-150-wm	15.94	10.00	9.29	1.80	0.82	0.74	16.12	0.99
7	1.5-65-180-wm	45.50	10.95	20.63	0.83	0.82	0.63	33.64	1.35
8	1.5-95-180-wm	27.45	10.95	14.52	1.29	0.82	0.68	24.44	1.12
9	1.5-125-180-wm	17.25	10.95	11.31	1.77	0.82	0.72	19.48	0.89
10	2-65-100-wm	25.59	7.07	14.92	0.54	0.82	0.62	24.11	1.06
11	2-95-100-wm	17.94	7.07	10.44	0.84	0.82	0.67	17.41	1.03
12	2-125-100-wm	13.54	7.07	8.10	1.15	0.82	0.71	13.84	0.98
13	2-65-150-wm	39.05	8.66	23.13	0.52	0.82	0.59	36.81	1.06
14	2-95-150-wm	28.75	8.66	16.18	0.81	0.82	0.64	26.56	1.08
15	2-125-150-wm	18.58	8.66	12.56	1.11	0.82	0.68	21.09	0.88
16	2-65-180-wm	50.08	9.49	28.17	0.51	0.82	0.58	44.52	1.12
17	2-95-180-wm	35.66	9.49	19.71	0.79	0.82	0.63	32.12	1.11
18	2-125-180-wm	23.79	9.49	15.30	1.09	0.82	0.67	25.50	0.93
19	2.5-65-100-wm	31.27	6.32	29.95	0.53	0.82	0.64	49.03	0.64
20	2.5-95-100-wm	24.28	6.32	20.83	0.83	0.82	0.69	35.23	0.69
21	2.5-125-100-wm	18.91	6.32	16.11	1.15	0.82	0.73	27.93	0.68
22	2.5-65-150-wm	67.21	7.75	46.42	0.51	0.82	0.61	74.81	0.90
23	2.5-95-150-wm	48.40	7.75	32.28	0.80	0.82	0.66	53.72	0.90
24	2.5-125-150-wm	30.70	7.75	24.98	1.10	0.82	0.70	42.56	0.72
25	2.5-65-180-wm	96.99	8.49	56.53	0.50	0.82	0.60	90.48	1.07
26	2.5-95-180-wm	60.05	8.49	39.31	0.78	0.82	0.65	64.95	0.92
27	2.5-125-180-wm	39.54	8.49	30.42	1.08	0.82	0.69	51.44	0.77
28	1.5-125-100-wm-R	11.55	8.16	5.99	1.87	0.82	0.77	10.58	1.09
29	2-125-100-wm-R	15.04	7.07	8.10	1.15	0.82	0.71	13.84	1.09
30	2-65-150-wm-R	43.44	8.66	23.13	0.52	0.82	0.59	36.81	1.18
31	2-95-150-wm-R	29.14	8.66	16.18	0.81	0.82	0.64	26.56	1.10
32	2-125-150-wm-R	20.02	8.66	12.56	1.11	0.82	0.68	21.09	0.95
33	2-125-180-wm-R	25.09	9.49	15.30	1.09	0.82	0.67	25.50	0.98
								Mean	1.00
								Std. deviation	0.16
								COV	0.16

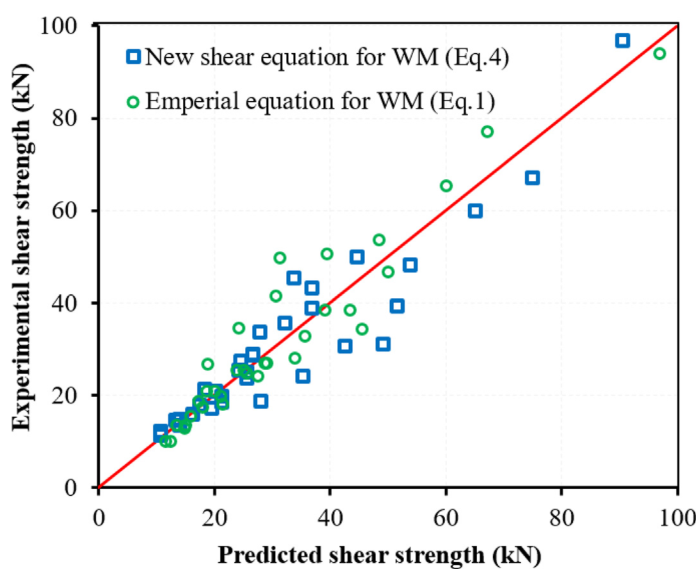

Fig. 13. Experimental and predicted shear strength of WM connection.

As shown in Fig. 13, a good agreement between the suggested empirical equation and the new equation for the welded moment connection was observed in terms of the mean, standard deviation, and coefficient of variation having the same values of 1, 0.16, and 0.16 respectively (refer to Tables 3 and 4). Hence, the proposed new shear equation is reliable and efficient for the welded moment connection.

Limits of applicability:

The proposed empirical (Eq. (1)) and new shear equations (Eq. (4)) for the WM connection with the welded clip-angle and screwed flange-cleat are applicable under these limitations range

Maximum depth of the beam = 200 mm

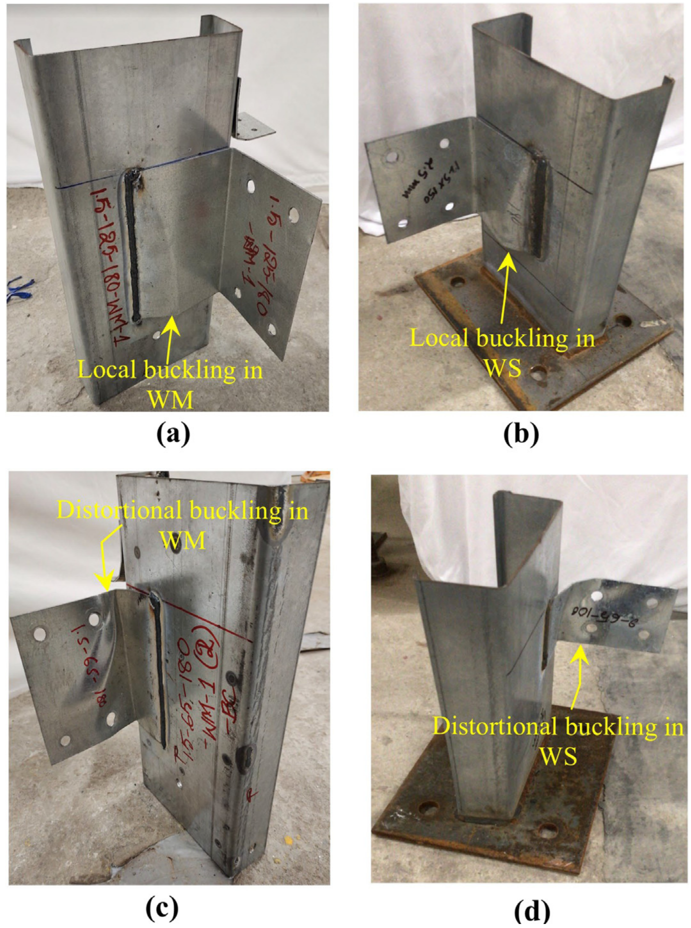

Fig. 14. Failure modes inWM (present) and WS [17]: (a) WM-local buckling; (b) WS-local buckling; (c) WM-distortional buckling; (d) WS-distortional buckling.

Yield strength of the clip-angle = 275 MPa to 435 MPa

Thickness of clip-angle = 1.5 mm to 2.5 mm

Aspect ratio (W/D) = 0.34 to 1.21

It should be noted that Eq. (4) for the WM connection is also applicable to the WS connection with welded clip-angle,

5. Welded moment versus welded shear connection

5.1. Failure modes of WM and WS

The welded clip-angle fails in distortional buckling when the aspect ratio (W/D) $<$ 0.8 and in local buckling failure when W/D $\geq$ 0.8

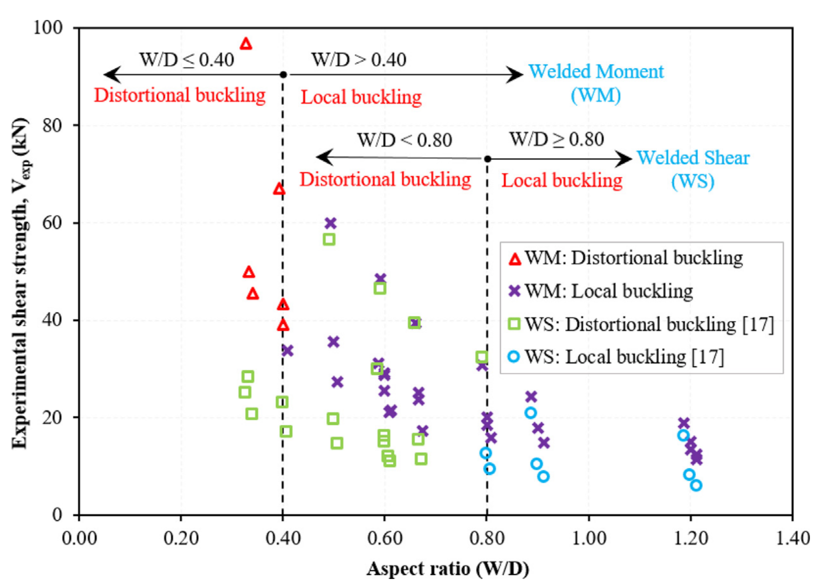

Fig. 15. Failure modes comparison of WM and WS connections.

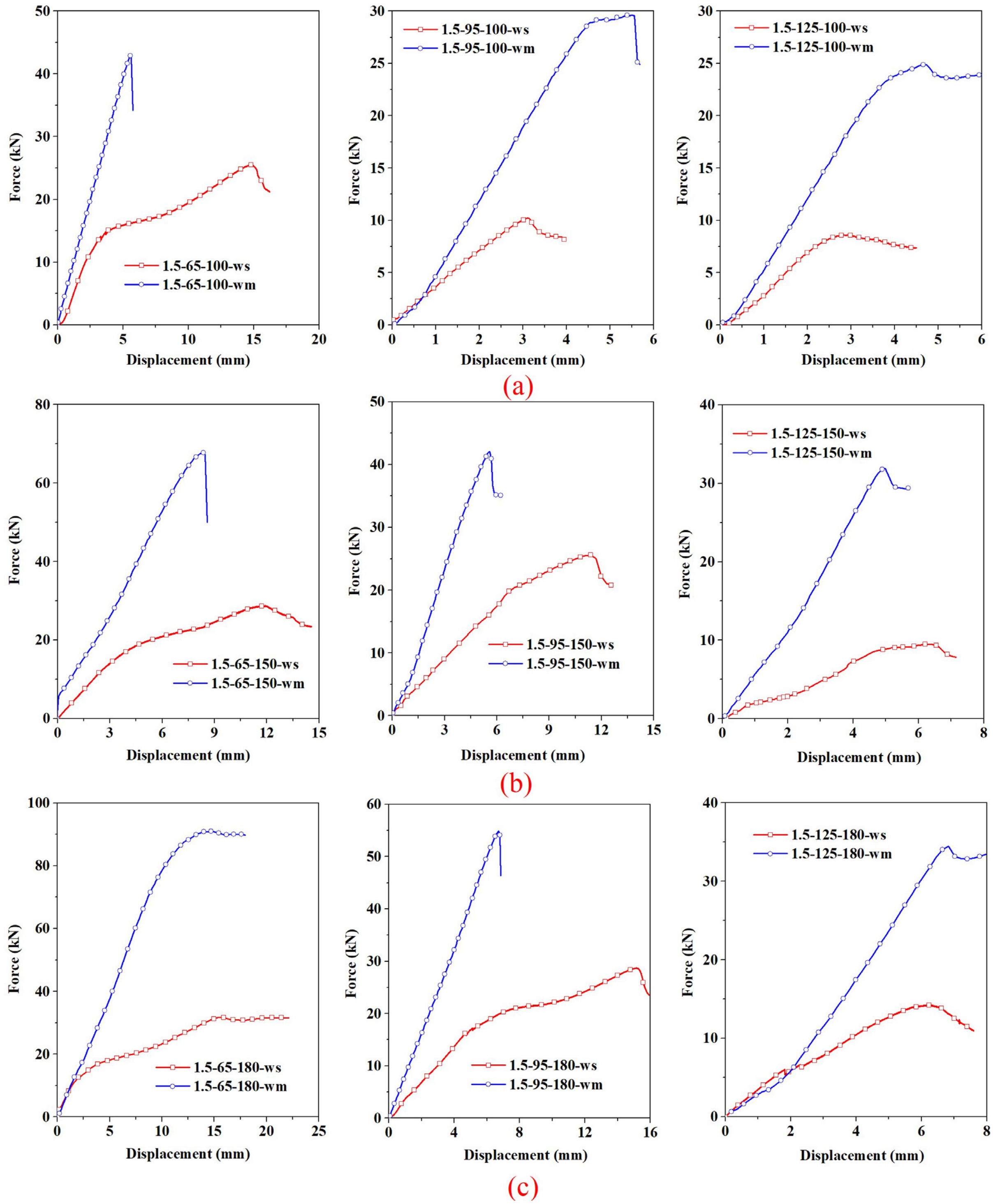

Fig. 16. WM vs WS plots of 1.5 mm clip-angles: (a) 100 mm depth; (b) 150 mm depth; (c) 180 mm depth.

in a shear connection, as per Mallepogu and Madhavan's study [17]. However, in the current study, the same welded clip-angles were found to fail in distortion for W/D ≤ 0.4 and in local buckling for W/D > 0.4, as shown in Figs. 14 and 15. The distortional buckling in the clip-angle is evident when the beam rotates excessively resulting in dislocation of the CA leg and out of plane movement of the cross-section. Some of the clip-angles (0.4 < W/D < 0.8) that were subjected to distortional buckling in the WS connection were found to fail in local buckling in the WM connection (refer to Fig. 15). This means that the flange-cleat provided in the WM connection can minimize the beam twisting and arrested distortional buckling failure. At the same time, the increased shear strength in these clip-angles in WM connection is contributed by the flange-cleat and failure mode change.

In WS connection: The load offset from the beam shear center results (refer to Fig. 1) in torsional deformation of the beam cross-section and produced a twisting moment in the beam. The high twisting moment in the beam results in distortional buckling of the clip-angle [17], while the low twisting moment results in local buckling.

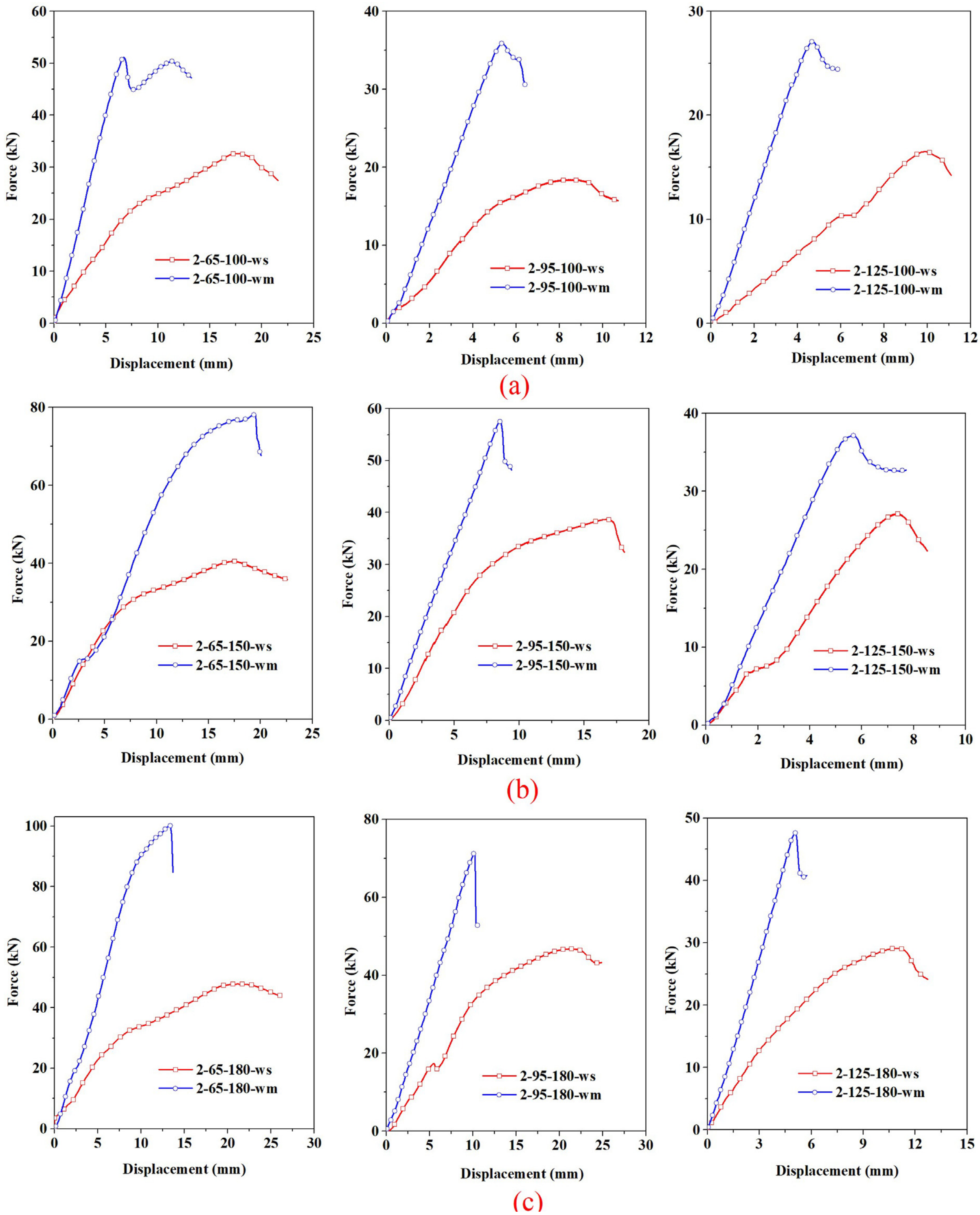

Fig. 17. WM vs WS plots of 2 mm clip-angles: (a) 100 mm depth; (b) 150 mm depth; (c) 180 mm depth.

In WM connection: The specimens with an aspect ratio of 0.4 to 0.8 failed in the distortional buckling in the WS connection [17]. While the same specimens failed in local buckling in the WM connection after including the flange-cleat. Hence, it is evident that flange-cleat is effective in resisting the free twisting of the beam and minimizing the corresponding twisting moment.

5.2. Load versus displacement curves of WM and WS

The shear capacity of the WM connection is higher than the WS connection, albeit their initial stiffness and ductility were not improved in some clip-angles. From the force versus displacement plots comparison of WM versus WS curves (refer to Figs. 16, 17, and 18), it can be

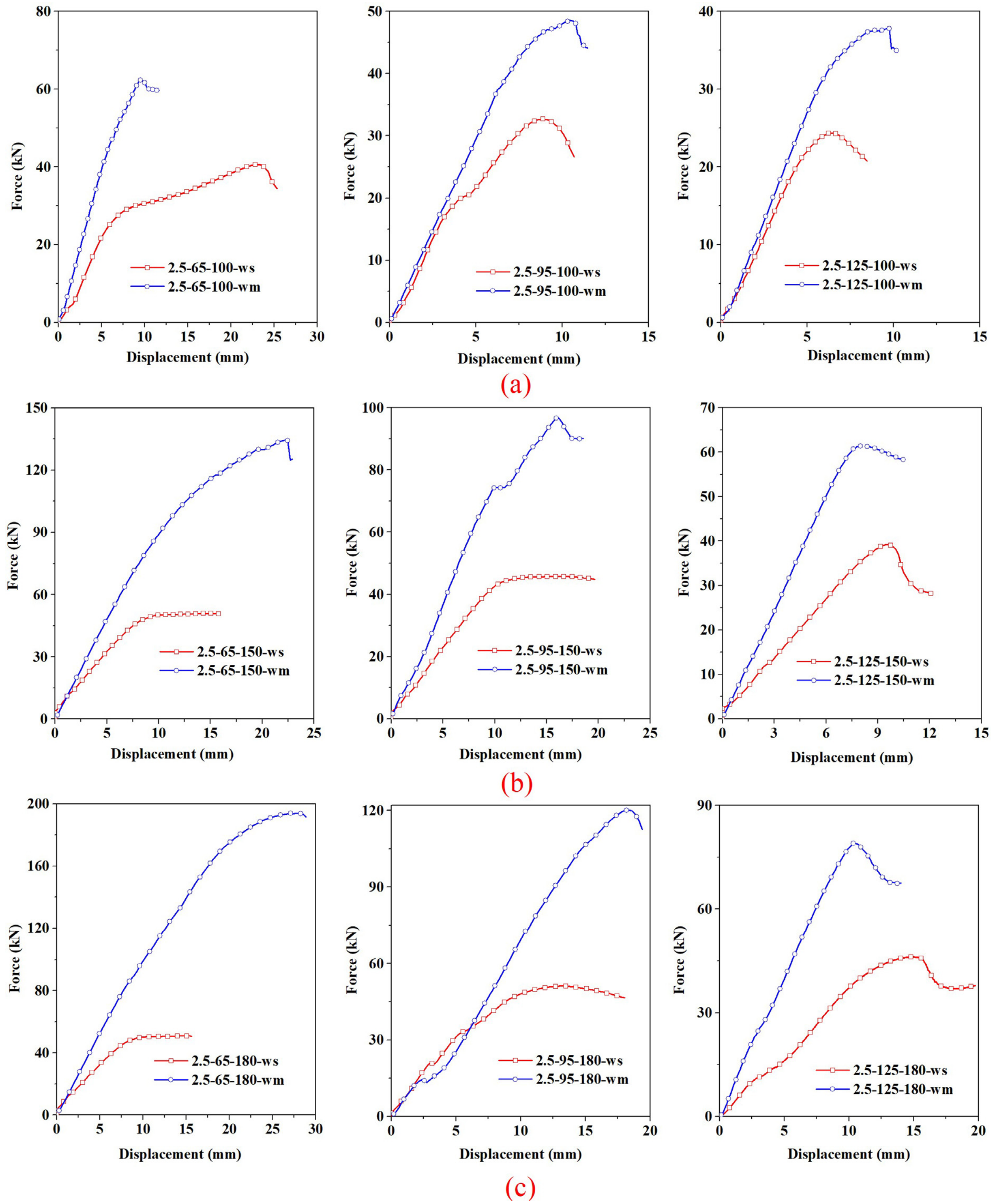

Fig. 18. WM vs WS plots of 2.5 mm clip-angles: (a) 100 mm depth; (b) 150 mm depth; (c) 180 mm depth.

observed that the increase in the initial stiffness of the WM connection over the WS connection was higher in case of the 1.5 mm clip-angles than that of 2 mm clip-angle. In comparison, this stiffness increase is marginal in the case of 2.5 mm thick clip angles. In the case of 2.5 mm thick clip-angles, there is little or no increase in the ductility in the WM connection over the WS connection, even after the inclusion of the flange cleat. In contrast, a considerable decrease in the ductility was found in 2 mm and 1.5 mm clip-angles after adding the flange-cleat. This is because the 2.5 mm clip-angle being a higher thickness than the 2 mm flange-cleat, resisted most of the shear load and beam twisting. Therefore, adding flange cleats with a lower thickness ($t_{FC} < t_{CA}$) than the clip-angle results in less shear capacity increment and ductility.

From Fig. 19 and Table 5, the performance ratio of the welded moment connection over the welded shear connection is above the

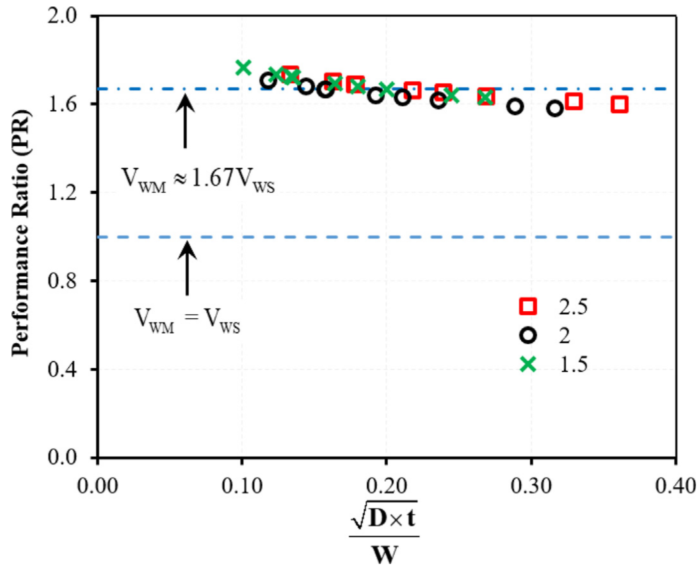

Fig. 19. Performance ratio versus Aspect ratio plot.

Table 5
Performance ratio of WM connection.

S. No.	Clip-angle configuration: t-A-D	$\frac{\sqrt{D \times t}}{W}$	Performance Ratio, PR = 1+β
1	1.5-65-100-wm	0.20	1.67
2	1.5-95-100-wm	0.13	1.72
3	1.5-125-100-wm	0.10	1.77
4	1.5-65-150-wm	0.24	1.64
5	1.5-95-150-wm	0.16	1.70
6	1.5-125-150-wm	0.12	1.74
7	1.5-65-180-wm	0.27	1.63
8	1.5-95-180-wm	0.18	1.68
9	1.5-125-180-wm	0.14	1.72
10	2-65-100-wm	0.24	1.62
11	2-95-100-wm	0.16	1.67
12	2-125-100-wm	0.12	1.71
13	2-65-150-wm	0.29	1.59
14	2-95-150-wm	0.19	1.64
15	2-125-150-wm	0.14	1.68
16	2-65-180-wm	0.32	1.58
17	2-95-180-wm	0.21	1.63
18	2-125-180-wm	0.16	1.67
19	2.5-65-100-wm	0.27	1.64
20	2.5-95-100-wm	0.18	1.69
21	2.5-125-100-wm	0.13	1.73
22	2.5-65-150-wm	0.33	1.61
23	2.5-95-150-wm	0.22	1.66
24	2.5-125-150-wm	0.16	1.70
25	2.5-65-180-wm	0.36	1.60
26	2.5-95-180-wm	0.24	1.65
27	2.5-125-180-wm	0.18	1.69
			1.67

unity, which indicates the increase in the shear capacity. The shear strength increase can be quantified as follows

Performance ratio (PR)

$$= \frac{\text{Nominal shear strength of Moment connection, } V_{WM}}{\text{Nominal shear strength of Shear connection, } V_{WS}} = (1 + \beta)$$

The flange cleat resists the free twisting of the beam due to the shear center effect (refer to Fig. 1), enhancing the WM connection's shear capacity over the WS connection. From Table 5 and Fig. 19, it can be concluded that the WS connection's shear strength is improved by 58% to 77% in the WM connection with the inclusion of the flange-cleat. Approximately all the WM connection experienced an average of 67% shear strength increment compared to the WS connection. The PR decreased with the increase of the geometric variable ($\frac{\sqrt{D\times t}}{W}$). Hence the clip-angles with a lesser value $\frac{\sqrt{D\times t}}{W}$ is recommended for the effective use of the flange-cleat.

6. Reliability studies

Reliability analysis was carried out on the proposed shear equations for the welded moment connection to ascertain safe design loads from the nominal shear strength. The design factors were calculated for a pre-defined target reliability value, and vice versa is also possible. The second approach is selected in this study. According to the Chapter K of AISI S 100 [20], the target reliability value (ϕ) was calculated using the below equation.

$$\phi = C_\phi \times M_m \times F_m \times P_m \times e^{-\beta_0 \sqrt{V_M{}^2 + V_F{}^2 + C_P V_P{}^2 + V_Q{}^2}} \tag{6}$$

Where, C_Φ= calibration coefficient = 1.52 for LRFD method
= 1.42 for LSD method
M_m = mean of the material factor
F_m = mean of the fabrication factor
P_m = mean of the professional factor
β_0 = target reliability index value = 3.5 for connections for LRFD method
= 4 for connections for LSD method
V_M = coefficient of variation of the material factor
V_F= coefficient of variation of fabrication factor
C_P = correction factor $= \left(1 + \frac{1}{n}\right) \times \left(\frac{m}{m-2}\right)$; if $n \geq 4$
n = number of tests conducted
m = degrees of freedom = n −1
V_P = coefficient of variation of experimental results ≤ 6.5
V_Q = coefficient of variation of load effects
= 0.21 for LRFD and LSD methods
Ω = safety factor = $\frac{1.6}{\phi_{LRFD}}$

The design factors (resistance and safety factors) for the proposed empirical shear equation and the new shear equation for the welded moment connection were found to have the same values (refer to Table 6); this indicates the good agreement between these equations. The design factors of 0.54, 0.43, and 2.94 were determined for the welded moment connection for the LRFD, LSD, and ASD methods. From Mallepogu and Madhavan's research work [17], the design factors for the WS connections were determined as 0.48, 0.38, and 3.32 for the LRFD, LSD, and ASD methods.

Table 6
Reliability analysis of welded moment connection.

Description	Empirical equation for Welded Moment connection Eq. (1)	Proposed new eq. for Welded Moment connection Eq. (4)
No. of tests conducted (n)	33	33
Degrees of freedom (m)	32	32
Mean value of professional factor (P_M)	1.00	1.00
Standard deviation (σ)	0.16	0.16
Coefficient of variation of test results (V_p)	0.16	0.16
correction factor (C_p)	1.10	1.10
Mean value of material factor (M_M)	1.10	1.10
Mean value of fabrication factor (F_M)	1	1
Calibration coefficient (C_Φ): LRFD	1.52	1.52
Calibration coefficient (C_Φ): LSD	1.42	1.42
Coefficient of variation of material factor (V_M)	0.08	0.08
Coefficient of variation of fabrication factor (V_F)	0.15	0.15
Coefficient of variation of load effects (V_Q)	0.21	0.21
Target reliability index (β_o): LRFD	3.5	3.5
Target reliability index (β_o): LSD	4	4
Resistance factor (Φ): LRFD	0.54	0.54
Resistance factor (Φ): LSD	0.43	0.43
Safety factor (Ω): ASD	2.94	2.94

7. Conclusions

This research endeavours to study the shear performance of CFS welded moment (WM) connection in a beam-to-column connection with the welded clip-angle and screwed flange-cleat. The existing equation for the shear capacity of welded shear connection (WS) is further improvised to make it applicable to WM connection. An empirical shear equation that is applicable to WM connection is also suggested. Two failure modes were identified in the welded clip-angle moment and shear connections: local buckling and distortional buckling. Local buckling failure occurs in welded clip-angle for an aspect ratio (W/D) > 0.4 in the WM connection and W/D ≥ 0.8 for the WS connection. In contrast, distortional buckling failure occurs for W/D ≤ 0.4 in the WM connection and W/D < 0.8 in the WS connection. The shear strength was increased by 67% after the flange-cleat was added to the welded clip-angle, representing a performance ratio of 1.67 in this study. The flange-cleat with a thickness less than or equal to the thickness of welded clip-angle ($t_{CA \leq} t_{FC}$) is recommended to achieve efficient shear performance. The design factors of 0.54, 0.43, and 2.94 were suggested for calculating the design shear capacity of the WM connection as per LRFD, LSD, and ASD methods, respectively.

Symbols and abbreviations

CFS cold-formed steel
HRS hot-rolled steel
WM welded moment connection
WS welded shear connection
LRFD load and resistance factor design
LSD limit state design
ASD allowable strength design
f_y steel yield strength
f_u steel ultimate strength
E young's modulus of steel
μ steel Poisson's ratio
CA clip-angle
FC flange-cleat
W flat width of outstanding leg of welded clip-angle
A total width of outstanding leg of clip-angle
D depth of clip-angle
W/D aspect ratio of welded clip-angle
V_{exp} experimental ultimate shear strength
V_y yield shear strength
V_{emp-wm} Predicted empirical shear strength of WM connection
t_{CA} thickness of the clip-angle
t_{FC} thickness of the flange-cleat
α_{CA} material factor of the clip-angle
α_{FC} material factor of the flange-cleat
L flat length of the flange cleat
g gauge length of the flange cleat
X_{CA} clip-angle coefficient
X_{FC} flange-cleat coefficient
β rigidity coefficient
V_{WM} shear strength of welded moment connection
V_{WS} shear strength of welded shear connection
λ slenderness ratio of the clip-angle
V_{cr} critical shear strength of the clip-angle
k elastic buckling coefficient
V_{WM} nominal shear strength of the welded moment connection
V_{WS} nominal shear strength of the welded shear connection
PR Performance ratio
Φ resistance factor (LRFD, LSD)
C_Φ calibration coefficient
M_m mean of material factor
F_m mean of fabrication factor
P_m mean of professional factor
β_0 target reliability index value
V_M coefficient of variation for material factor
V_F coefficient of variation for fabrication factor
V_P coefficient of variation for experimental results
V_Q coefficient of variation for load effects
Ω safety factor (ASD)
V_d design shear strength

CRediT authorship contribution statement

Nagaraju Mallepogu: Writing – original draft, Investigation, Formal analysis, Data curation, Conceptualization. **Mahendrakumar Madhavan:** Writing – review & editing, Supervision, Resources, Project administration, Funding acquisition, Conceptualization.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

Acknowledgments

The first author would like to thank the Ministry of Education (MoE) of India for the financial support in the form of a fellowship for this research.

Appendix. Design example

Determine the shear strength of a welded clip-angle between the CFS beam and column. The geometric details of the clip-angle are shown in Fig. 20. and other details are given below. Design the flange-cleat to increase the load carrying capacity of the connection (see Table A.1).

Yield strength of the clip-angle = 300 MPa
Young's modulus, E = 2×10^5 MPa
Poisson's ratio, $\mu = 0.3$
Corner radius of clip-angle = $1.5 \times$ t = 3 mm

Solution:

A.1. Shear connection design

Flat-width of the outstanding leg of the clip-angle, $W = 75 - 3 - 2 = 70$ mm

Aspect ratio of the clip-angle, $W/D = 70/150 = 0.47 < 0.8$

∴ The clip-angle will be subjected to distortional buckling in a shear connection.

Elastic buckling coefficient, $\mathrm{k} = 2.569\left(\frac{\mathrm{W}}{\mathrm{D}}\right)^{-2.202} = 13.76$

Critical buckling stress, $\mathrm{f_{cr}} = \frac{\mathrm{k}\pi^2\mathrm{E}}{12\left(1-\mu^2\right)}\left(\frac{\mathrm{t}}{\mathrm{D}}\right)^2 = 442.18$ MPa

Critical shear strength of the clip-angle, $\mathrm{V_{cr}} = \mathrm{f_{cr}}(\mathrm{t\ D}) = 132.65$ kN

Yield shear strength of the clip-angle, $= \mathrm{V_y} = 0.6\mathrm{f_y}(\mathrm{t\ D}) = 54$ kN.

Slenderness ratio of the clip-angle , $\lambda = \sqrt{\frac{\mathrm{V_y}}{\mathrm{V_{cr}}}} = 0.64$

Shear strength of the welded clip-angle, $\mathrm{V_{WS}} = 0.275\lambda^{-0.8}\mathrm{V_y} = 21.22$ kN

The design shear strength of WS connection from Mallepogu and Madhavan [17] study is

For LRFD method:

$$\mathrm{V_{d-LRFD}} = \phi_{\mathrm{LRFD}} \times \mathrm{V_{WS}} = 0.48 \times 21.22 = 10.18 \text{ kN}$$

For LSD method:

$$\mathrm{V_{d-LSD}} = \phi_{\mathrm{LSD}} \times \mathrm{V_{WS}} = 0.38 \times 21.22 = 8.06 \text{ kN}$$

$$\mathrm{V_{d-ASD}} = \frac{\mathrm{V_{WS}}}{\Omega_{\mathrm{ASD}}} = 21.22/3.32 = 6.39 \text{ kN}$$

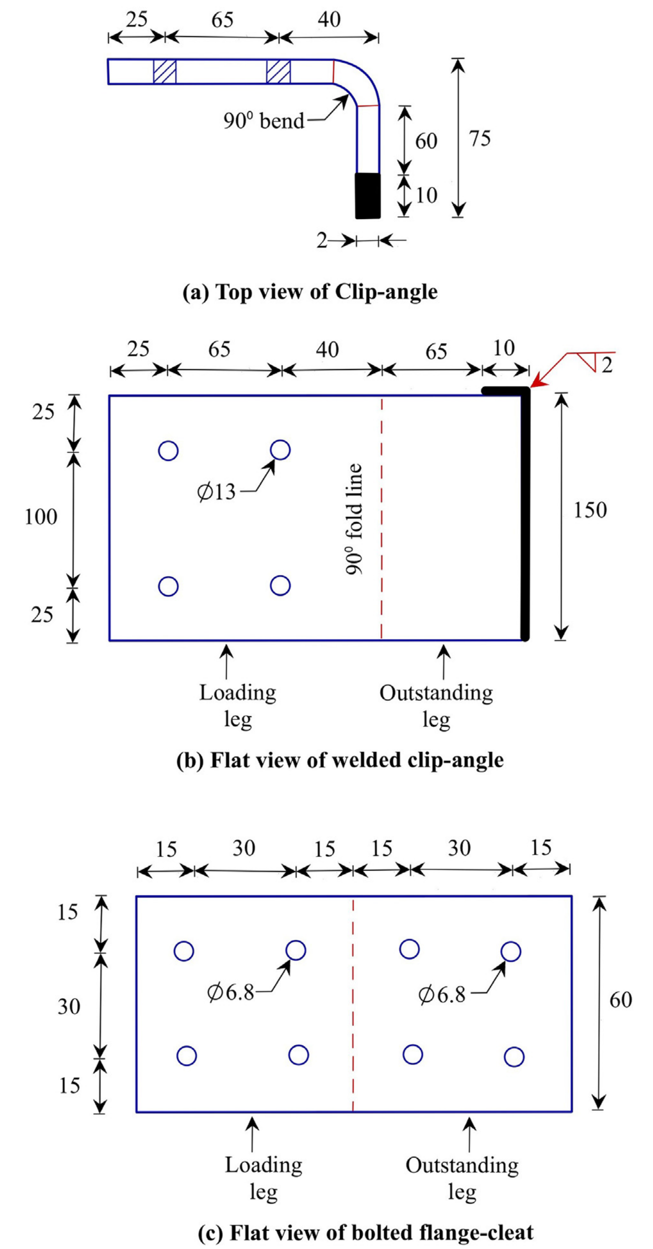

Fig. 20. Design example: (a) Top view of CA; (b) Flat view of CA; (c) Flat view of FC.

A.2. Moment connection design

Following the recommendation of $t_{CA} \le t_{FC}$, 2 mm thickness is selected for the flange-cleat with M6 bolts on both the legs as shown in Fig. 20 (C). The yield strength of flange-cleat is 300 MPa, which is same of the

Table A.1
Design example calculations.

Rigidity factor(β) = 0.60		Welded moment connection	Welded shear connection
		Clip-angle + Flange-cleat $V_{WM} = V_{WS}(1+\beta)$	Clip-angle $V_{WS} = 0.275\ (\lambda)^{-0.8}\ V_y$
Nominal shear strength (kN)	V_n	33.95	21.22
Design shear strength (kN)	V_{d-LRFD}	19.04	10.18
	V_{d-LSD}	15.17	8.06
	V_{d-ASD}	12	6.39

clip-angle material.

$$X_{CA} = \frac{\sqrt{tD}}{W\alpha_{CA}{}^{0.65}} = \frac{\sqrt{2 \times 150}}{70 \times \left(\frac{300}{275}\right)^{0.65}} = 0.234$$

$$X_{FC} = \frac{\sqrt{tg}}{L\alpha_{FC}{}^{0.65}} = \frac{\sqrt{2 \times 30}}{10 \times \left(\frac{300}{275}\right)^{0.65}} = 0.732$$

Rigidity coefficient of welded moment connection,

$$\beta = 0.48\left(\frac{X_{FC}}{X_{CA}}\right)^{0.2} = 060$$

The shear capacity of the welded moment connection ,

$$V_{WM} = V_{WS}(1+\beta) = 33.95 \text{ kN}$$

∴ Shear capacity of connection is increased by 60% after adding flange-cleat along with the clip-angle.

The shear strength of WM connection from the empirical equation, $V_{WM-e} = 0.457(\lambda)^{-0.8}\, V_y = 35.27 \text{ kN} \approx 33.95 \text{ kN}(V_{WM})$

Therefore, the empirical shear equation and new design shear equation have good match with each other.

The design shear strength of WS connection using new shear equation

For LRFD method:

$$V_{d-LRFD} = \phi_{LRFD} \times V_{WM} = 0.54 \times 35.27 = 19.04 \text{ kN}$$

For LSD method:

$$V_{d-LSD} = \phi_{LSD} \times V_{WS} = 0.43 \times 35.27 = 15.17 \text{ kN}$$

For ASD method:

$$V_{d-ASD} = \frac{V_{WS}}{\Omega_{ASD}} = 35.27/2.94 = 12 \text{ kN}$$

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