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Experimental Investigation on Cold-formed Steel Beams under Pure Bending

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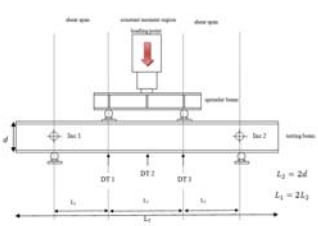
Graphical abstract



Abstract

This paper presents the flexural behaviour of cold-formed double lipped channels beams under pure bending action. Two channel sections are bolted back-to-back to form an I-shape structural beam member. A series of six experiment tests were carried out on beam specimens DC200 and DC250, each with 200 mm depth and 250 mm depth respectively. The thickness of beam section is 2 mm and the design yield strength is 350 N/mm^2. All beams failed at local buckling at top-flange due to lateral instability of the cold-formed steel structural members. The moment resistance for DC200 is 17.87 kNm and DC250 is 31.53 kNm. The experimental results are compared to theoretical resistance prediction based on British Standard and Eurocode. The comparison showed that the experimental moment capacity is lower than the theoretical bending moment resistance but higher than theoretical buckling moment resistance from Eurocode. This showed that a better agreement is achieved between experimental data and Eurocode buckling moment resistance for cold-formed steel beam under pure bending.

Keywords: Flexural behaviour; bending test; buckling moment; cold-formed steel, lipped channel sections

Abstrak

Kertas kerja ini membentangkan sifat lenturan rasuk keluli tergelek sejuk di bawah tindakan lenturan tulen. Dua channel dihimpunkan berkembar dan diperketatkan dengan bolt untuk membentuk rasuk struktur bentuk-I. Satu siri ujian lenturan telah dijalankan ke atas spesimen rasuk DC200 dan DC250, dengan kedalaman 200 mm dan 250 mm masing-masing. Ketebalan keratan rasuk adalah 2 mm dan kekuatan reka bentuk adalah 350 N/mm^2. Semua rasuk gagal pada momen kilasan sisi di bahagian atas bebibir akibat ketakstabilan sisi anggota keluli tergelek sejuk. Rintangan momen bagi DC200 adalah 17.87 kNm dan DC250 adalah 31.53 kNm. Keputusan eksperimen dibanding dengan ramalan teori yang berdasarkan British Standard dan Eurocodes. Perbandingan tersebut menunjukkan bahawa rintangan momen lenturan eksperimen adalah lebih rendah daripada ramalan teori momen lenturan tetapi lebih tinggi daripada ramalan teori momen rintangan kilasan sisi mengikut Eurocode. Ini menunjukkan bahawa persetujuan baik dicapai di antara keputusan eksperimen dengan ramalan teori Eurocode momen rintangan kilasan sisi bagi rasuk keluli tergelek sejuk.

Kata kunci: Sifat lenturan; ujian lenturan; momen kilasan sisi; keluli tergelek sejuk, keratan ‘channel’ berbibir

1.0 INTRODUCTION

Light steel framing is one of the popular choices in residential buildings construction. Light steel members are prefabricated off-site and assembled on-site according to its modular unit [1]. Through the use of light steel framing, massive construction works are shifted into factory, leaving the construction site cleaner and safer. Light steel framing that utilised cold-formed steel section has some highlighted benefits such as high strength-to-weight ratio as compared to hot-rolled sections and concrete block, accelerating sustainable construction development as cold-formed steel is a reusable green material and rapid construction compared with conventional concrete structures. The typical design strengths for cold-formed steel section are 350 N/mm^2, 450 N/mm^2 and 550 N/mm^2 [2]. The cold-formed sections are composed of steel plates or sheets in roll-forming machines. There are three methods of forming, namely cold-roll forming, press brake operation and bending brake operation [3].

Cold-formed steel sections are generally applied in the construction on both primary and secondary structural members. They are different from the hot-rolled steel sections in structural behaviour where the thin-walled cold-formed sections are slender. Buckling is the most concern mode of failure in cold-formed steel design.

Several researches had been carried out on the flexural tests on cold-formed steel beams [4-6]. Maduliat *et al.* [4] studied on inelastic behaviour and design of cold-formed channel sections in bending. Forty-two tests were carried out to investigate the pure bending behaviour. Maduliat *et al.* found that current international codes are conservative for cold-formed channel section design. The results showed that cold-formed channel sections have indicated that sections with low slenderness exhibit significant inelastic behaviour and resulting in the capacities exceeding the first yield values.

Post-failure behaviour of box section beams was experimentally studied by Kotelko *et al.* [5]. From the non-linear post-buckling analysis, the load capacity and energy absorption of a beam was recorded during experiment. The theoretical analysis generally underestimates the post-failure mechanism of the hollow cold-formed steel beams. Kotelko *et al.* found that the strain hardening effects have contributed to the inadequacy of the analytical study.

Pastor and Roure [6] studied about the open cross-section beams under pure bending. There were twenty-two beams of U-sections and Omega-sections (in Ω-shape) tested in a four-point bending configuration. The experimental results were compared to the results from finite element analysis. The behaviour of the tested beam showed higher resistance than theoretically prediction by Eurocode 3 with regard to cross-sections classification.

The application of cold-formed steel beams in Malaysia construction industry is not popular. The open cross-section cold-formed beam flexural behaviour has not yet been studied in depth. Cold-formed structures have lateral instabilities in structural performance. Cold-formed steel sections are easy to buckle in several way, such that local buckling, lateral torsional buckling and distorsional buckling. This behaviour needs to be studied to increase the reliability of the use of cold-formed steel structures.

This paper presents the experimental investigation on the flexural behaviour of bolted back-to-back cold-formed steel channel beams under pure bending. Two types of cold-formed steel beam with 200 mm depth (DC200) and 250 mm depth (DC250) respectively are studied. Both sections are 2 mm in thickness and with the design strength of 350 N/mm^2. The beams are laterally restrained to prevent distortional and lateral-torsional buckling. The load-deflection flexural behaviours are recorded for further analysis and discussion.

■2.0 EXPERIMENTAL INVESTIGATION

2.1 Specimens Preparation

Two cold-formed steel channel sections are assembled back-to-back to form an I-beam as shown in Figure 1. The standard properties and nominal values for the design strength for both beams and bolts are given Table 1. The channels are bolted together with 400 mm spacing for DC200 and 500 mm spacing for DC250. These spacing are used in the loading points to ensure effective forces transferring through the bolted connections.

The bolt type that used for assembling the channel sections is 12 mm of Grade 8.8 bolts with 25 mm length. The bolts are used to prevent premature failure caused by weak connection between C-channels. The procedures for test specimens preparation and test setting up are shown in Figure 2.

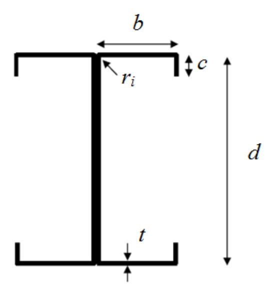
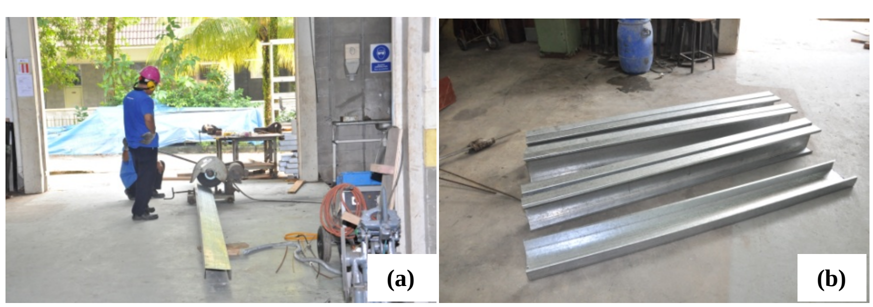


Figure 1 Shape of the cold-formed steel beam section

Table 1 Material properties

Beam Section	**Depth, *d* (mm)**	**Width, *b* (mm)**	**Lip, *c* (mm)**	**Radius of corner, r_i (mm)**	**Thickness, *t* (mm)**	**Design Strength, f_y (N/mm^2)**
DC200	200	73	17	4	2	350
DC250	250	77	18	4	2	350
Bolt	**Diameter, *d* (mm)**	**Length, (mm)**	**Shear strength, $F_{v,Rd}$ (N/mm^2)**	**Tensile Strength, $F_{T,Rd}$ (N/mm^2)**	**Bearing Strength, $F_{b,Rd}$ (N/mm^2)**	
M12 Grade 8.8	12	25	375	560	1000	



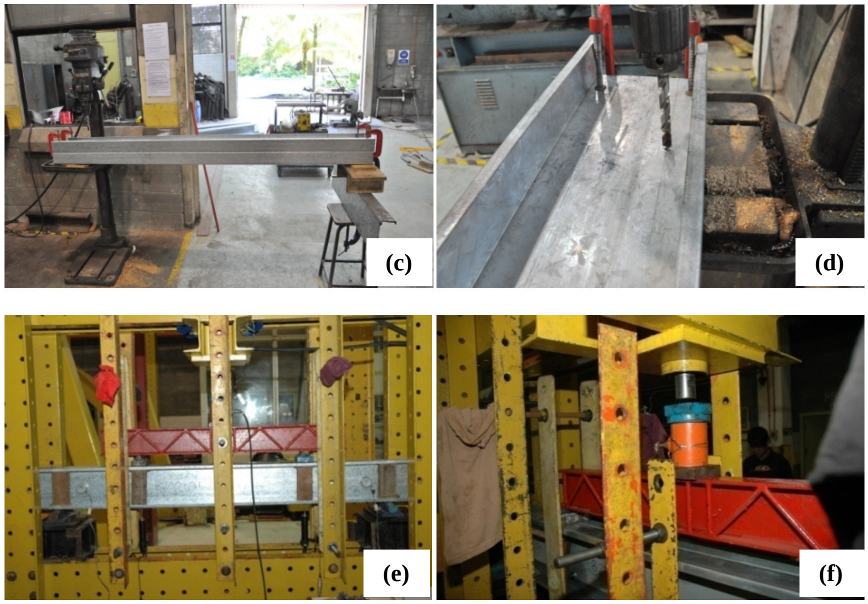


Figure 2 Preparation of test specimens: (a) Cutting process. (b) C-channel had been cut to its desired length. (c) C-channel assembled back-to-back at drilling machine. (d) Hole drilling with 12.5 mm drill-bit. (e) Testing configuration (f) Placement of spreader beam and load cell on the beam specimen

2.2 Test Setting

The four point bending test configuration was set up for experimental investigation. The length of the beam was fixed to 1600 mm for DC200 and 2000 mm for DC250. The difference in length is due to the requirement for shear span and constant moment region. According to Ziemian [7], the shear span, L_2 required is double of the beam depth as shown in Figure 3 and Table 2. The constant moment region, L_1 is normally double of the shear span.

The beam is laterally restrained along the constant moment region to prevent lateral-torsional buckling. The restraints are formed across the steel section and attached to the testing frame. A minimum of 2 mm spacing is required between beam specimens and the restraints, which can prevent the beam to be stiffen by the restraints. Stiffeners are also required at the loading and support points to prevent the beam from crushing under point loads.

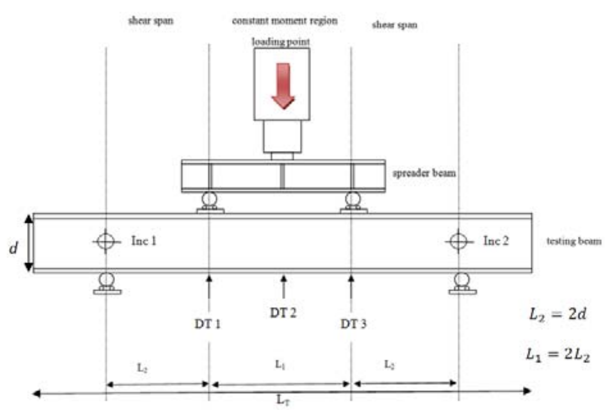


Figure 3 Experiment design

Table 2 Distances and length of the beam specimens

Section	Constant Moment Region, L_1 (mm)	Shear span, L_2 (mm)	Distance $L_1 + 2L_2$ (mm)	Total beam length, (mm)
DC200	800	400	1600	2000
DC250	1000	500	2000	3000

2.3 Location of Data Acquisition System

The data are recorded from three set of displacement transducers (DT) and two set of inclinometers (Inc). One 100-milimeter DT is placed at the middle of the beam, which is the predicted location of maximum deflection. The other two 50-milimeter DT are placed on the beam in-line with the supporting points of the spreader beam. The position of the displacement transducers can achieve the optimum results where the stiffness and structural behaviour of the beam can be predicted from the experimental investigation results. The possibility of the failure location of cold-formed steel beam may occur near to the loading point. The inclinometers are placed at the centre of the beam and 50 mm away from the supports to avoid over crowded with the stiffeners.

■3.0 ANALYTICAL STUDY

The analytical study is carried out with reference to the BS 5950 Part 5 [8] (hereby referred as BS5950-5) and Eurocode BS EN 1993-1-3 [9] (hereby referred as EC3-1-3). The obtained analytical results are compared with the experimental data for validation.

3.1 Bending Moment Resistance of Beam

The bending moment resistance, M_{cx} for DC section according to BS5950-5 [8] is calculated as:

$$M_{cx} = \min(p_o Z_x\ , p_y Z_{xr}) \tag{3.1}$$

where Z_x is the elastic modulus
Z_{xr} is the reduced elastic modulus
p_o is the reduced design strength,

$$p_o = \left\{1.13 - 0.0019\frac{D_w}{t}\sqrt{\frac{p_y}{280}}\right\}, p_o \le p_y$$

D_w is the depth of the beam
t is the thickness of the beam
p_y is the design yield strength

While the formula for bending moment resistance according to EC3-1-3 [9] is calculated as:

$$M_{c,Rd} = \frac{f_{yb} W_{eff,y}}{\gamma_{mo}} \tag{3.2}$$

where f_{yb} is the design strength
$W_{eff,y}$ is the effective elastic modulus
γ_{mo} is the partial safety factor, which is taken as 1.0

3.2 Buckling Moment Resistance of Beam

In BS5950-5 [8], the lateral buckling moment resistance, M_b for an unrestrained beam is given as below :

$$M_b = \frac{M_E M_Y}{\phi_B + \sqrt{\phi_B^2 - M_E M_Y}} \le M_{cx} \tag{3.3}$$

where, $\phi_B = \frac{M_Y + (1+\eta) M_E}{2}$ (3.4)

M_Y is the elastic moment resistance, $M_Y = Z_{xr} p_o$ (3.5)
M_E is the elastic lateral buckling resistance,

$$M_E = \frac{\pi^2 AED}{2(L_E/r_y)^2} C_b \left\{1 + \frac{1}{20}\left(\frac{L_E}{r_y}\frac{t}{D}\right)^2\right\}^{0.5} \tag{3.6}$$

for $\frac{L_E}{r_y} > 40C_b$; $\eta = 0.002\left(\frac{L_E}{r_y} - 40C_b\right)$;
else $\eta = 0$ (3.7)

$$C_b = 1.75 - 1.05\beta + 0.3\beta^2 \le 2.3 \tag{3.8}$$

β is the ratio of the smaller end moment to the larger end moment in the unrestrained length of the beam. For uniformly distributed load, β is taken as 1.0, and thus C_b equals to 1.0. The effective unrestrained length L_E is given in Clause 5.6.3 of BS5950-5 [8], ranged 0.7L (beam fully restrained towards rotation) to 1.1L (beam not restrained towards any rotation) for different types of support condition. Conservatively L_E is taken as 1.1L in this study.

According to EC3-1-3 [9], lateral buckling moment resistance, $M_{b,Rd}$ for an unrestrained beam is given as below:

$$M_{b,Rd} = \chi_{LT}\frac{W_{eff,y} f_{yb}}{\gamma_{M1}} \tag{3.9}$$

where $\chi_{LT} = \frac{1}{\phi_{LT} + (\phi_{LT}^2 - \lambda_{LT}^2)^{0.5}}$ is the reduction factor for lateral-torsional buckling (3.10)

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2] \tag{3.11}$$

$\lambda_{LT} = \left(\frac{M_{c,Rd}}{M_{cr,LT}}\right)^{1/2}$ is the relative slenderness for lateral-torsional buckling (3.12)

$W_{eff,y}$ is the section modulus of effective section for bending about y-axis
f_{yb} is the design strength of the member.

3.3 Deflection

The theoretical deflection is calculated based on the formula from Davison [10]:

$$\delta_{max} = \frac{PL^3}{6EI}\left[\frac{3a}{4L} - \left(\frac{a}{L}\right)^3\right] \tag{3.13}$$

where δ_{max} is the deflection under two point load
P is the unfactored load
E is the Young's modulus
I is the moment of inertia
L is the effective beam length, which is equal to L_1 + $2L_2$
a is the beam length from support to loading point, which is equal to L_2

■4.0 RESULTS AND DISCUSSION

4.1 Experimental and Analytical Results

Load-deflection curve is used to present the experimental behaviour of the cold-formed steel beam. The results from the inclinometer are also converted into load-deflection curve for further analysis.

The experimental results covered the three specimens for each DC200 and DC250. Figure 4 and 5 show the load-deflection curves and the experimental photo of failure mode for the six tested beams. The experimental data are summarized in Table 3.

The analytical work covers the calculation of bending moment resistance and buckling moment resistance from BS5950-5 and EC3-1-3. Deflections are calculated base on Davison [10]. The results of analytical study are summarized in Table 4. From the analytical work, it is showed that both BS5950-5 and EC3-1-3 give close prediction to the bending moment resistance (M_{cx}) of the cold-formed steel beams. However EC3-1-3 requirements has resulted a significant lower values of buckling moment resistance (M_b) as compared to BS5950-5 (refer to Table 4). Previous research [11] has discussed on the factor that cause the difference between the buckling moment resistances from the two codes. The analytical prediction would be compared to experimental results and the suitability of the analytical prediction will be discussed in Section 4.3 of this paper.

Table 3 Summary of experimental results

Specimen	Max. Load, kN	Maximum Deflection, (mm)			Initial Stiffness, kN/m	Maximum Moment, kNm
		DT 1	DT 2	DT 3		
DC200-01	87.70	14.48	14.91	13.79	7.462	17.54
DC200-02	88.10	12.41	13.14	11.64	7.630	17.62
DC200-03	92.30	14.95	16.00	15.90	7.392	18.46
DC250-01	122.10	14.48	14.77	12.86	10.02	30.50
DC250-02	129.90	13.05	14.85	13.83	9.971	32.50
DC250-03	126.30	10.37	11.64	9.94	10.95	31.58

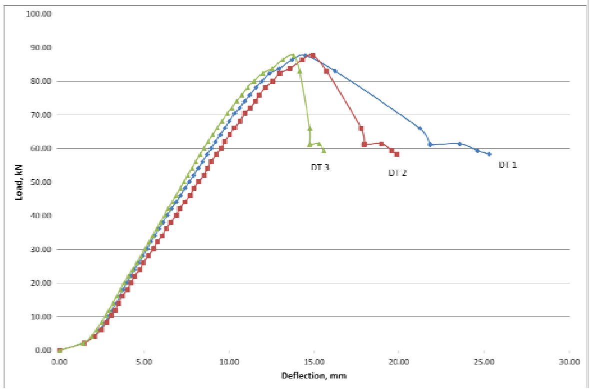


Figure 4(a) Load-deflection curve for DC200-01

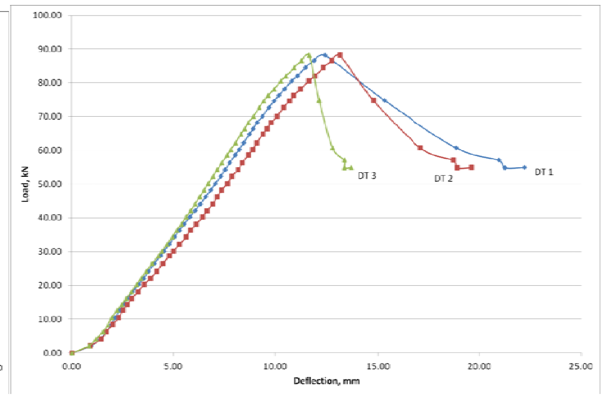


Figure 4(b) Load-deflection curve for DC200-02

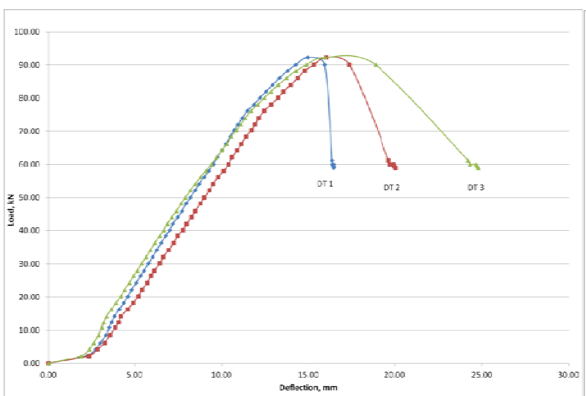


Figure 4(c) Load-deflection curve for DC200-03

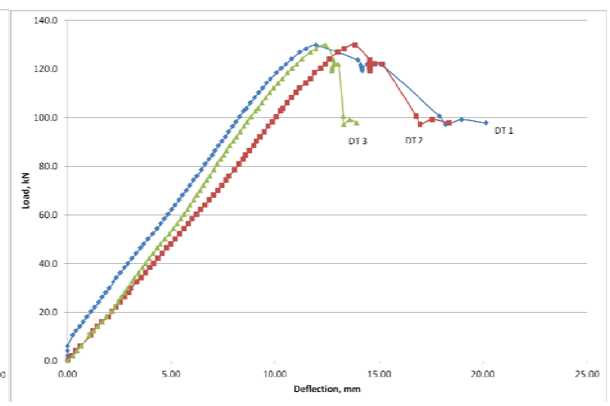


Figure 4(d) Load-deflection curve for DC250-01

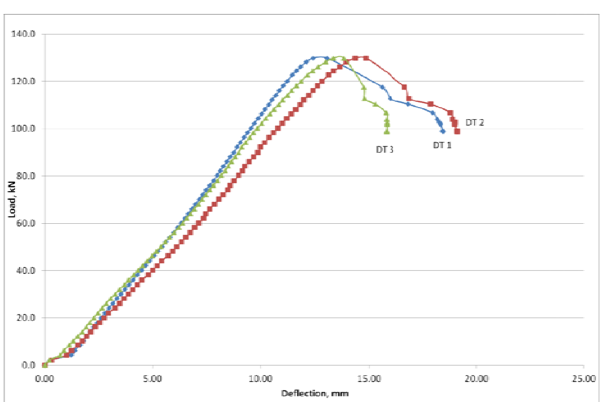


Figure 4(e) Load-deflection curve for DC250-02

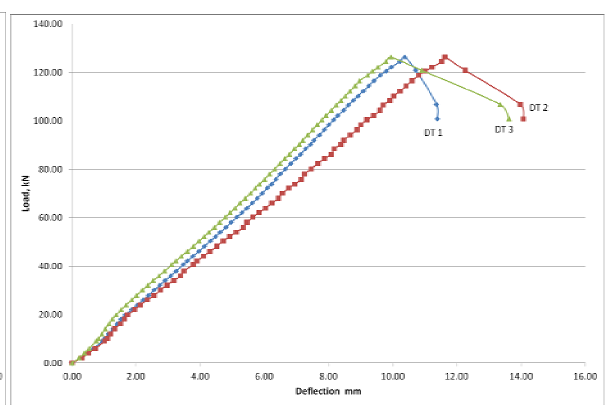


Figure 4(f) Load-deflection curve for DC250-03

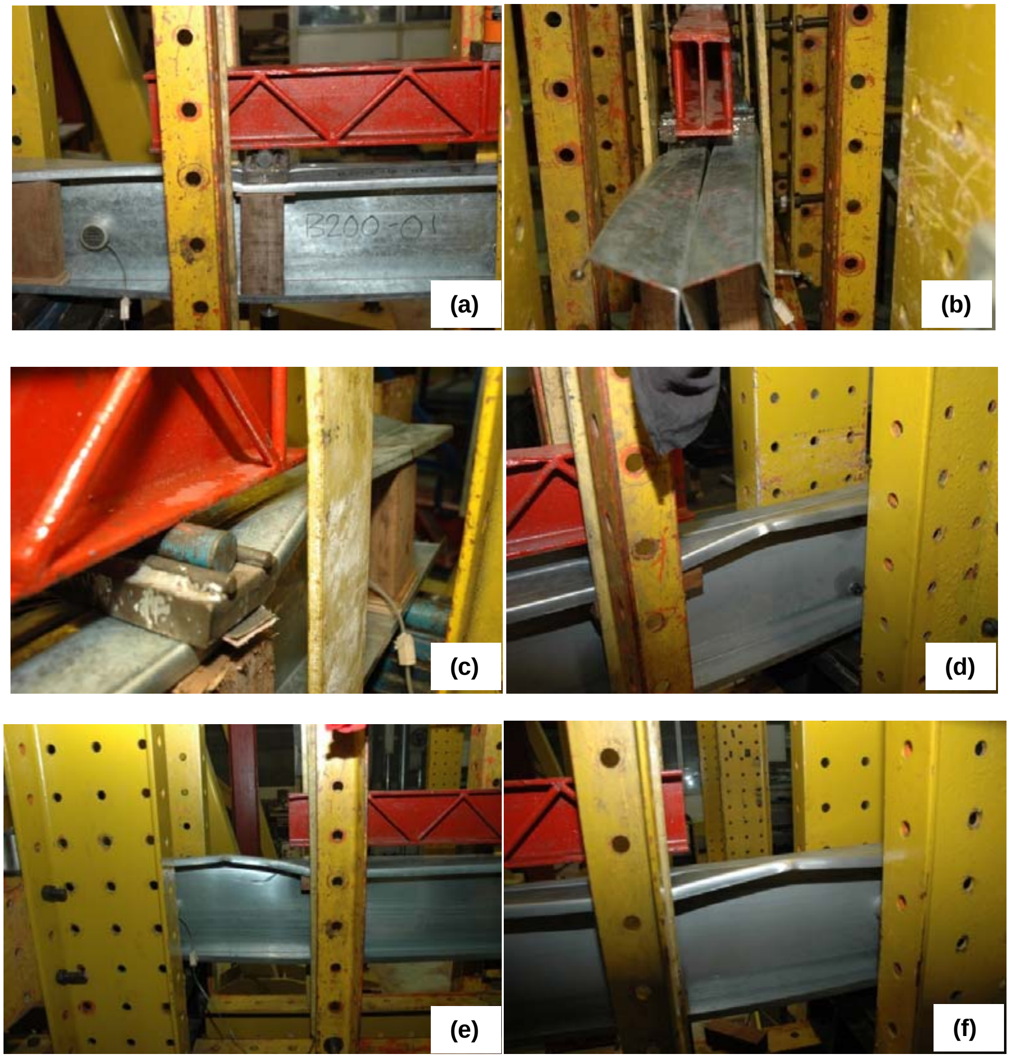


Figure 5 Failure mode (local buckling at top flange) for all tested beams (a) DC200-01 (b) DC200-02 (c) DC200-03 (d) DC250-01 (e) DC250-02 (f) DC250-03

Table 4 Analytical results of moment resistance and deflection

Resistance	DC 200 with 1600 mm Length		DC 250 with 2000 mm Length	
	BS 5950-5	**EC 3-1-3**	**BS 5950-5**	**EC 3-1-3**
Bending Moment Resistance, M_{cx} (kNm)	27.6	27.8	36.5	35.7
Buckling Moment Resistance, M_b (kNm)	27.0	17.0	34.4	21.0
Deflection, δ_{max} (mm)	6.16*		10.17*	
Note: * deflections are calculated based on Davison [10].				

4.2 Failure Mode Observation

Refer to Figure 4, the deflection at the mid-span of the beam as recorded by DT2 are higher than the deflection below the two load points as recorded by DT1 and DT3. The observation shows good agreement between theoretical predictions to the physical behaviour of the beam under pure bending. The average deflection recorded by DT1 and DT3 has insignificance difference with 1.2% for DC200 and 3.3% for DC250. It shows that the loads transferred by the spreader beam are balanced. This small gap influenced by the imperfection in the beam section. Idealised same deflection at both loading points has difficulty to be achieved in physical experimental investigation. Continuously monitoring on DT1 and DT3 during experiment is made to prevent the beams from premature failure due to unsymmetrical loading. The ultimate load resistance by the three DC200 beams (as shown in Figure 4(a) to Figure 4(c)) and DC250 (Figure 4(d) to Figure 4(f)) are averagely very close. These observations show that the experimental investigations were carried out as required and thus increase the reliability of the collected data.

Figure 5 shows the failure mode of all six tested beams. The process of flexural collapse was initiated by a buckle in the compression flange. All tested beam fail at the local buckling of top flange near to the point load. For cold-formed sections, due to their open and thin cross-sectional geometry, it gives better bending rigidity about one axis at the expense of low torsional rigidity and low bending rigidity about a perpendicular axis [12]. This leads to three major buckling problem in cold-formed members, namely lateral-torsional buckling, distortional buckling and local buckling. Since the beams were restrained to prevent from lateral-torsional buckling and distortional buckling, the beams experienced local buckling at the top flange. The failure mode observation shows a significant difference of behaviour between cold-formed steel and hot-rolled steel beams. Within desired classification, hot-rolled steel beams are able to achieve plastic and elastic moment resistance (M_c) when they are restrained against lateral movement. These bending moment resistance are hardly achieved by cold-formed steel beams due to the local buckling of the top flange.

The increment of beam depth has contributed to stiffening of the beam. More loads can be sustained by DC250 with low deflection according to the experimental data. From Table 3, the stiffness and strength of the cold-formed beam increase as the beam depth increase. The DC200 has the average initial stiffness of 7.49 kN/mm while the DC250 has average stiffness of 10.31 kN/mm. The DC200 has achieved the average maximum moment of 17.87 kNm and 31.53 kNm for DC250.

4.3 Comparison between Experimental and Analytical Results

The comparison between codes of practice and experimental data are carried out for validation. The deflection from experimental and analytical results are given in Table 5. The calculated bending moment resistance from BS5950-5 and EC3-1-3 are compared with the obtained data from experiment as shown in Figures 6 and 7.

Table 5 Summary of deflection from experimental and analytical results

Specimen	Max. Load, kN	Maximum Experimental Deflection, mm			Maximum Analytical Deflection, mm (Davison [10])
		DT 1	DT 2	DT 3	
DC200-01	87.70	14.48	14.91	13.79	6.04
DC200-02	88.10	12.41	13.14	11.64	6.08
DC200-03	92.30	14.95	16.00	15.90	6.37
Average for DC200	**89.37**	**13.95**	**14.68**	**13.78**	**6.16**
DC250-01	122.10	14.48	14.77	12.86	9.85
DC250-02	129.90	13.05	14.85	13.83	10.48
DC250-03	126.30	10.37	11.64	9.94	10.19
Average for DC250	**126.10**	**12.63**	**13.75**	**12.21**	**10.17**

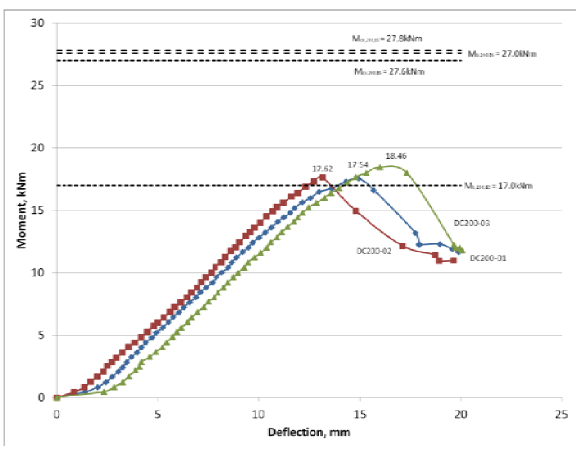


Figure 6 Moment-deflection curve for DC200 with theoretical moment resistance and buckling moment resistance, namely $M_{cx,200,BS}$ = 27.6kNm, $M_{cx,200,EC}$ = 27.8kNm, $M_{b,200,BS}$ = 27.0kNm, $M_{b,200,EC}$ = 17.0kNm

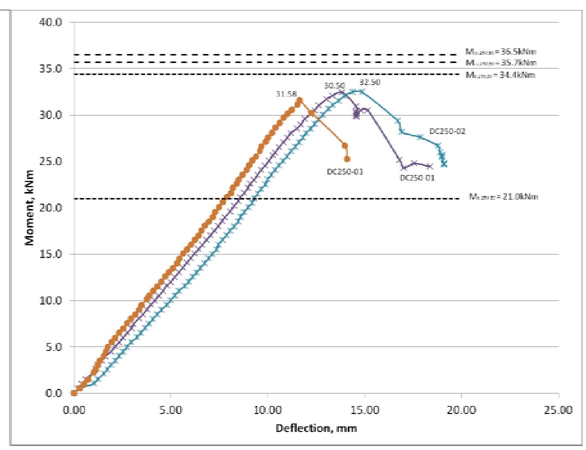


Figure 7 Moment-deflection curve for DC250 with theoretical moment resistance and buckling moment resistance, namely $M_{cx,250,BS}$ = 36.5kNm, $M_{cx,250,EC}$ = 35.7kNm, $M_{b,250,BS}$ = 34.4kNm, $M_{b,250,EC}$ = 21.0kNm

From Table 5, the analytical study has 58.0% different with DC200 experimental data on deflection. Whereas, there is a 26.0% different in DC250. Slenderness of the beams may contribute to the difference between experimental and theoretical deflection, where second order effects may need to be taken into consideration in predicting the deflection of cold-formed steel beam. Nevertheless, all six beams illustrate linear-elastic behaviour before achieving ultimate load resistance, as shown in the moment-deflection curves.

The experimental results from DC200 show that the failure moment is close to the Eurocode EC3-1-3 buckling moment resistance, $M_{b,200,EC}$.For DC250, it was found that none of the codes have showed a precisely representation with experimental data, although experimental data may close to buckling moment resistance of BS5950-5, $M_{b,250,BS}$.

Moreover, refer Figures 6 and 7, it is clearly shown that all tested cold-formed steel beam are unable to achieve the bending moment resistance (M_{cx}) as discussed in the failure mode observation. It also can be seen that the EC3-1-3 has a more conservative buckling resistance (M_b) prediction as compared to BS5950-5, and the experimental data shows that the beams failed with the buckling moment less than the predicted BS5950-5 buckling moment resistance. The comparisons show that EC3-1-3 presents a conservative design in cold-formed beam moment resistance. Although it might not economical in design, but the beam member designed by using Eurocode has higher safety issue. Further investigation on the buckling moment curve with various length and beam depth is necessary in order to achieve reliability in cold-formed steel structures design.

■5.0 CONCLUSION

The bending behaviour of six cold-formed steel beams, formed by double lipped channel section bolted back-to-back are investigated. The experimental results are compared to theoretical prediction based on British Standard BS5950-5 and Eurocode EC3-1-3. From the study, several conclusions can be drawn:

- Strength and stiffness of the beam increases with an increment to the beam depth. The DC200 has achieved the average maximum moment resistance of 17.87 kNm and 31.53 kNm for DC250.
- All beams failed at local buckling of the top flange. This is a significant failure mode observation of cold-formed steel beams as compared to hot-rolled steel beams. The local buckling of the top flange prevents the cold-formed steel beam to attain elastic moment resistance (M_c) that may be achieved by most of hot-rolled steel sections. Cold-formed steel beams should be designed to their buckling moment resistance (M_b) until further investigation is made.
- Analytical study has 58.0% difference with DC200 experimental data and 26.0% for DC250 on deflection. BS5950-5 overestimates the buckling moment of the steel beams while EC3-1-3 gives conservative but safe design values as compared to experimental data.

The experimental data shows all tested beams failed at local buckling. The deflection increase gradually before local buckling occurred. The yield line formed at the top compression flange of the beams. This pattern can represent the flexural behaviour for all tested beams. Since cold-formed beams are easy to buckle, slenderness factor should be included in the flexural behaviour of the member. Increasing in sectional area or beam depth may increase the flexural capacity of the beam.

It is suggested that more tests to be made for further investigation. Various beam dimensions can be used to increase the reliability of flexural behaviour for cold-formed steel beam design. Moreover, the buckling moment for cold-formed steel beam under pure bending should be investigated due to lateral instability of these thin-walled structures. Other structural members like column should be investigated since it is an integral part of the light steel framing system.

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