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Structural performance of cold-formed steel column bases with bolted moment connections

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Abstract. This paper presents a thorough investigation into the structural performance of cold-formed steel column bases using double lipped C sections with bolted moment connections. A total of four column base tests with different connection configurations were carried out, and it was found that section failure under combined bending and shear was always critical. Moreover, the proposed column bases were demonstrated to be structurally efficient attaining moment resistances close to those of the connected sections.
In order to examine the structural behaviour of the column base connections, a finite element model was established using shell and spring elements to model the sections and the bolted fastenings respectively. Both material and geometrical non-linearities were incorporated, and comparison between the test and the numerical results was presented in details. The design rules originally developed for bolted moment connections between lapped Z sections were adopted and re-formulated for the design of column base connections after careful calibration against the test data. Comparison on co-existing moments and shear forces at the critical cross-sections of the column bases was fully presented. It was shown that the proposed design and analysis method was structurally adequate to predict the failure loads under combined bending and shear for column bases with similar connection configurations.

Key words: column bases; bolted moment connections; section failure under combined bending and shear; finite element modelling.

1. Introduction

Cold-formed steel sections are light-weight materials and suitable for building construction owing to their high structural performance. The most common sections are lipped C and Z sections, and the thickness typically ranges from 1.2 mm to 3.2 mm, and sections with yield strengths from 250 to 450 N/mm^2 are commonly available. In general, cold-formed steel members are used as secondary members in building construction, and they are connected onto primary structural members through web cleats as pinned or moment connections, depending on the connection configurations. For typical applications of cold-formed steel sections, there are many design recommendations on cold-formed steel structures available in the literature such as AISI (1996), AS/NZ 4600 (1996), BS5950 (1998), Eurocode 3: Part 1.3 and Hong Kong Steel Code (2004). Moreover, a number of design guides and commentaries are also available to assist structural engineers to design cold-formed steel structures

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(Chung 1993, Hancock 1998, Lawson *et al*. 2002, Yu 2000).

Since the last decade, there is a growing trend to use cold-formed steel sections as primary structural members in building construction, such as low to medium rise residential houses and portal frames of modest span. Therefore, it is important to develop efficient moment connections between cold-formed steel members to allow for practical and efficient framing.

A review on various codes of practice for design of cold-formed steel structures shows that a number of design methods are available for the evaluation of section capacities and member resistances of typical sections against both local and overall buckling. However, for connection design, these codes only provide design rules for the load carrying capacities of individual fastenings with bolts, screws and welds while no guidance on the structural behaviour of beam-to-beam and beam-to-column connections is available at all. Hence, it is highly desirable to provide design rules for the general analysis and design of bolted connections between cold-formed steel members.

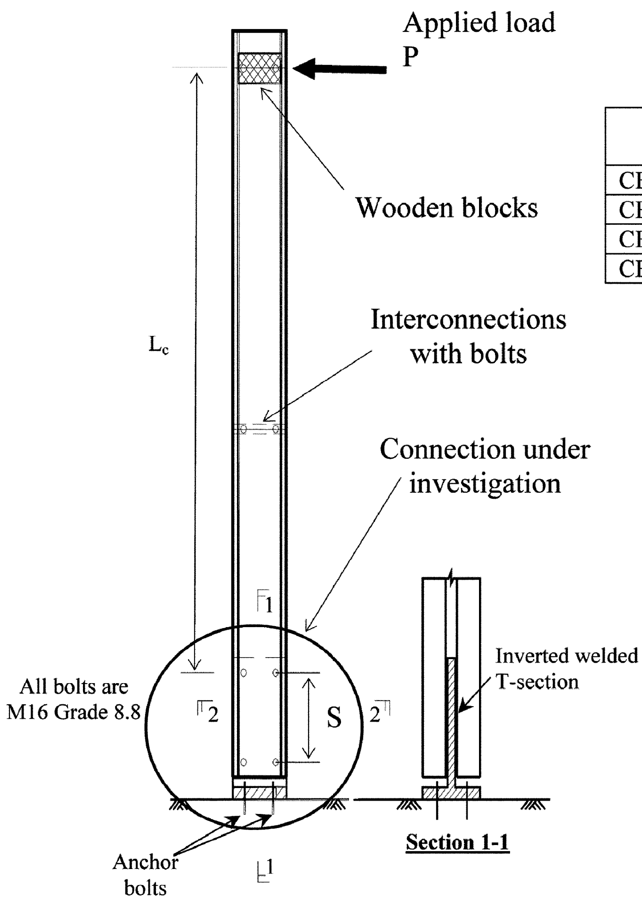
Although simple connections in cold-formed steel members were commonly used in practice for many years, little development was made on their use. Hot rolled steel angles used in hot rolled steel construction for simple connections were directly adopted to form simple connections between cold-formed steel members in many applications, and such usage was widely believed to be over-provided. An experimental investigation on simple connections between cold-formed steel sections using web cleats of folded cold-formed steel strips was reported by Chung and Lawson (2000), and a set of complementary design rules for strength assessment of these web cleats was also provided. Moreover, a number of experimental investigations on bolted moment connections between cold-formed steel members in typical building construction were also carried out by Chung and Lau (1999), Wong and Chung (2000a, 2000b and 2002) and Chung and Wong (2001). High structural efficiency of such bolted moment connections between cold-formed steel members was demonstrated. Furthermore, an extensive experimental investigation on bolted moment connections between lapped Z sections was reported by Ho and Chung (2004), and a set of design rules against section failure of connected sections under combined bending and shear was proposed by Chung and Ho (2004) after careful calibration against test data.

2. Scope of work

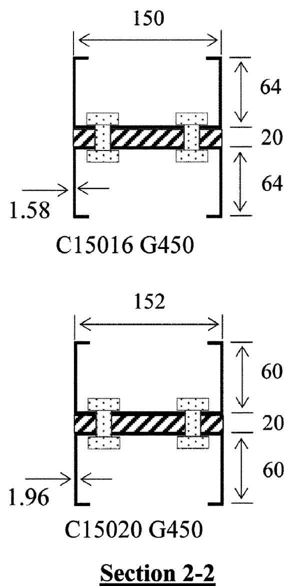
This paper presents a thorough investigation into the structural performance of cold-formed steel column bases using double lipped C sections with bolted moment connections. A total of four column base tests with different connection configurations were carried out to investigate their structural behaviour including moment resistances and typical modes of failure. Moreover, a finite element model was established using shell and spring elements to model the sections and the bolted fastenings respectively. Both material and geometrical non-linearities were incorporated, and comparison between the test and the numerical results was presented in details. The analysis and design rules originally developed for bolted moment connections between lapped Z sections were adopted and re-formulated to design the column base connections after careful calibration against test data. Comparison on the structural behaviour of those column base connections predicted from the numerical model and the proposed method was also presented.

3. Column base connection tests

A total of four cold-formed steel column base tests using double lipped C sections back-to-back with



Test	Section	S (mm)	L_c (mm)
CB180T16	C15016DS	180	1215
CB240T16	C15016DS	240	1155
CB180T20	C15020DS	180	1280
CB240T20	C15020DS	240	1220



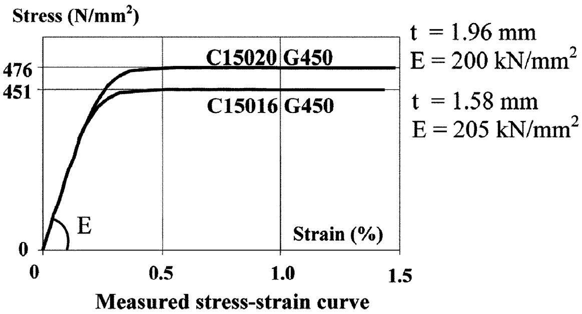


Fig. 1 Cold-formed steel column bases

bolted moment connections were carried out, and Fig. 1 illustrates the details of the connection configurations. It should be noted that the double sections were bolted onto the webs of inverted welded T sections which were in turn bolted securely onto the ground with anchored bolts. The thickness of both the webs and the flanges of the T sections was 20 mm while all the bolts were 16 mm in diameter and of grade 8.8. Moreover, interconnections at regular intervals were provided to the test specimens to avoid pre-mature failure of the column bases due to lateral instability.

In order to obtain test data on the structural performance of column bases with connection configurations of practical dimensions, both 1.6 and 2.0 mm thick sections with both 180 and 240 mm long bolt pitches were tested. The test specimens were designated as CB180T16, CB240T16, CB180T20 and CB240T20 as shown in Table 1. The measured yield strengths were found to be 451 and 476 N/mm^2 for 1.6 and 2.0 mm thick sections respectively. The measured dimensions of the sections and the corresponding stress-strain curves were also presented in Fig. 1 for easy reference.

In all tests, lateral loads were applied near the top of the test specimens and the applied loads were increased progressively until unloading occurred. Both the applied loads and the deflections of the loaded points of the test specimens were measured continuously during the tests. Among all the tests, section failure of the connected sections at the cross-sections of the first row of bolts always took place, and initiated the failure of the column base connections, as shown in Fig. 2. The load-deflection curves of all the column base tests were plotted in Fig. 3 in a consistent format for easy comparison. It was

Table 1 Summary of test program and test data

Test	Section	Thickness (mm)	Bolt pitch (mm)	P_{Test} (kN)	Failure mode
CB180T16	C15016 G450	1.6	180	5.97	MV
CB240T16	C15016 G450	1.6	240	6.94	MV
CB180T20	C15020 G450	2.0	180	7.36	MV
CB240T20	C15020 G450	2.0	240	8.29	MV

Notes: MV denotes section failure at the critical cross-section under combined bending and shear.

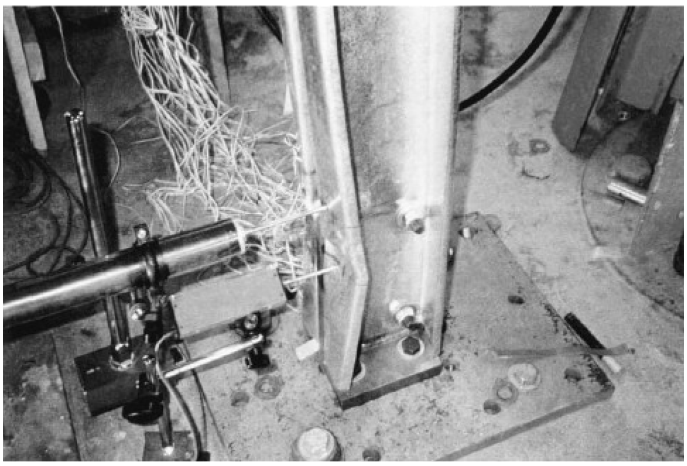


Fig. 2 Section failure at the critical cross-section under combined bending and shear

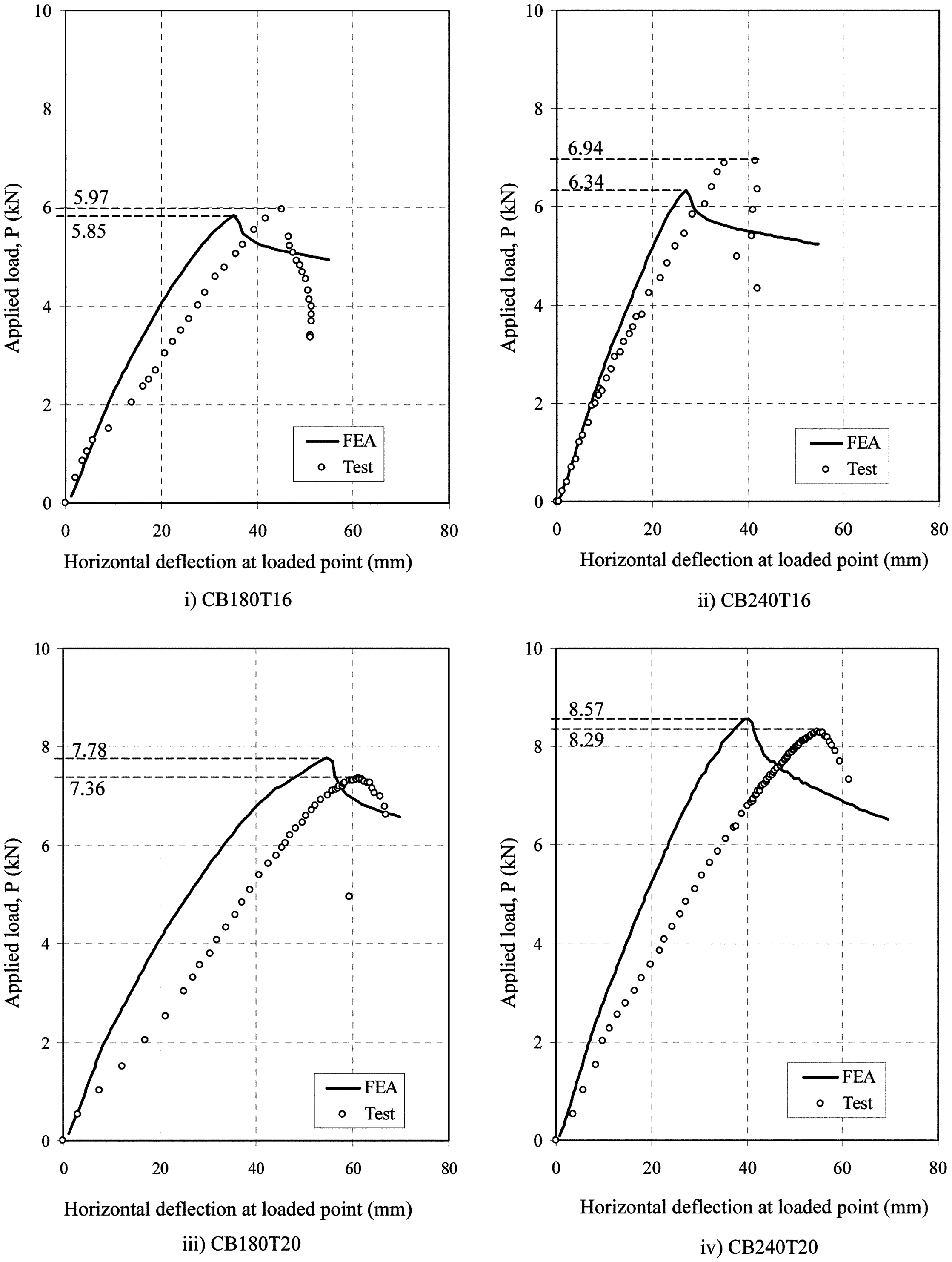


Fig. 3 Load-deflection curves of cold-formed steel column bases

shown that all the column bases rotated linearly under low applied loads, and then exhibited non-linear deformation characteristics when the applied loads increased. It should be noted that section failure in the connected sections tended to occur in a sudden manner with apparent unloading in all tests. The test results of all the tests were summarized in Table 1.

4. Numerical investigation

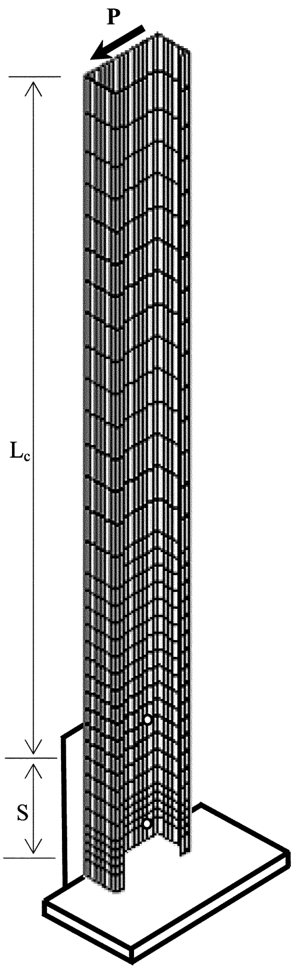
As cold-formed steel sections tend to be very slender when compared with typical hot rolled steel sections, it is expected that deformations in cold-formed steel structures are often significant. Any non-linear effects such as material yielding and secondary moments induced by both local and overall deformations of a structure should be fully incorporated in assessing the structural performance of cold-formed steel structures. This is also applicable to detailed examinations on connections between cold-formed steel sections, and non-linear finite element modelling is considered to be extremely useful to tackle such problems. Moreover, finite element modelling is also able to provide important information on the structural responses of cold-formed steel structures, such as deformed shapes, stress distributions, yielding patterns as well as internal force distributions after rational data analyses. A number of numerical investigations (Wheeler *et al*. 1999, Lim and Nethercot 2003) using commercial finite element packages were carried out to assess the structural behaviour of bolted moment connections. Moreover, advanced finite element modelling using three-dimensional solid elements with material, geometrical and boundary non-linearities were also reported (Chung and Ip 2000, 2001) to examine the load deformation characteristics of bolted fastenings under tension.

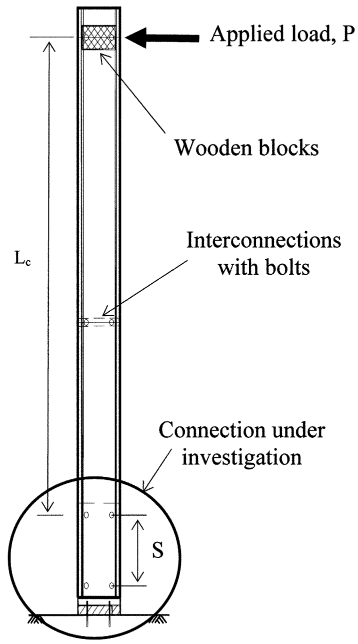
4.1. Finite element model

In this study, the general purpose finite element package ABAQUS (Version 6.4, 2004) was adopted for the numerical simulation of the cold-formed steel column bases, and for simplicity, only the full length of a C section was modeled in the column base tests due to symmetry, as shown in Fig. 4. The following features were incorporated into the finite element model:

- Iso-parametric four-node three-dimensional shell elements with reduced integration, S4R, were used to model the C sections.
- The measured stress-strain curves obtained from coupon tests were adopted in the material models of the C sections together with von Mises yielding criteria.
- Geometric non-linearity was incorporated into the finite element model to allow for large deformation in the C sections.
- Initial geometrical imperfection was provided to the C sections as the first eigenmode of the sections obtained from elastic critical buckling analysis. The maximum magnitude of the imperfection was taken to be 0.25 times the section thickness.
- The webs of the inverted T sections were modeled as rigid supports, and hence, no deformation of the T sections was possible.

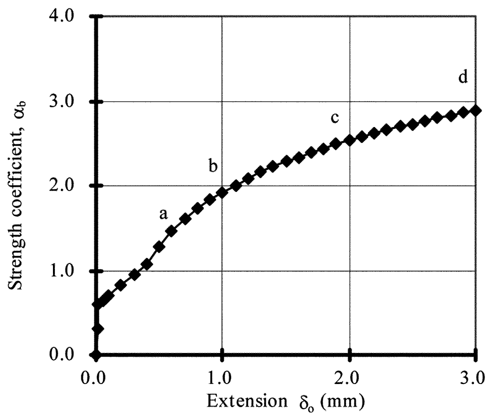
It should be noted that in the previous research work by Chung and Ip (2000, 2001), three dimensional eight-node iso-parametric solid elements SOLID45 were employed to model all components of a typical bolted connection in lap shear tests. Moreover, contact interfaces between various components including the cold-formed steel strip, the bolt, the washer and the hot-rolled steel plate were modeled by contact elements CONTAC49 so that the intuitive assumption on the position





Test	Section	S (mm)	L_c (mm)
CB180T16	C15016DS	180	1215
CB240T16	C15016DS	240	1155
CB180T20	C15020DS	180	1280
CB240T20	C15020DS	240	1220

Note: By symmetry, only half of the test specimen is modeled.



Normalized bearing deformation curve

Bolt force obtained from lap shear test, F_b

$F_b = \alpha_b\, d\, t\, f_u$

where

α_b is the strength coefficient

$= 30\,\delta_o$ when $\delta_o \leq 0.02$ mm

$= 1.25\,(\delta_o - 0.02) + 0.6$ when $0.02 < \delta_o \leq 0.4$mm

$= 0.85 \ln\left(\dfrac{\delta_o - 0.05}{0.35}\right) + 1.075$ when $0.4 < \delta o \leq 3.0$mm

d is the diameter of bolt,

t is the thickness of steel strip, and

f_u is the ultimate strength of steel strip.

Fig. 4 Finite element model of column bases and the size of the contact areas was not required. However, it required huge computational resources with large memory capacities and long running times.

In the current study, it was important to reckon that while bearing deformation always took place at the C sections whenever the bolt forces due to local moments were large, the primary objective of the study was to examine section failure at the critical cross-sections of connected C sections under combined bending and shear. Hence, simplification on the modelling of the bolts was structurally acceptable, and highly desirable in order to provide numerical solutions without excessive computational resources. Consequently, between the webs of the sections and the webs of the inverted T sections, all the bolts were modeled with out-of-plane springs as well as in-plane springs. The stiffness of the out-of-plane springs was assumed to be extremely large to prevent any separation between the webs of the C and the T sections while the stiffness of the in-plane springs was assumed to follow the normalized load-bearing deformation curve of standard lap shear tests. For details of the normalized load-bearing deformation curve, refer to Chung and Ho (2004).

4.2. Numerical results

The load-deflection curves of the four column bases obtained from the finite element models were plotted onto the same graphs of the measured curves in Fig. 3 for direct comparison. It was shown that both the numerical and the measured curves agreed very well over the entire deformation ranges. Fig. 5 illustrates both the deformed shapes and the stress distributions of the column base connections. It was shown that the predicted deformed shapes were very similar to mode of failure shown in Fig. 2. The maximum load resistances of the column bases obtained from the finite element models are also summarized in Table 2 for direct comparison with the measured values.

In order to assess the structural adequacy of the finite element models, a model factor, ψ_{FEM}, is established which is defined as follows:

$$\psi_{FEM} = \frac{\text{Measured load resistance from test, } P_{Test}}{\text{Predicted load resistance obtained from finite element analysis, } P_{FEM}} \tag{1}$$

It is shown that the model factors of the finite element models range from 0.95 to 1.09 with an average value of 1.01. Consequently, the proposed finite element model is shown to be adequately accurate in predicting the load resistances of column base connections.

Furthermore, extensive data post-processing on the finite element results was performed and the following detailed information was obtained:

- the forces, F_{m1} to F_{m4}, developed in the in-plane springs connecting the webs of both the C and the T sections, and
- both the shear forces, V, and the moments, M, of the C sections at various cross-sections along the column height.

Detailed information on the force distributions of all the column base connections at failure was presented in Table 3 while typical data of the bolt forces F_{m1} to F_{m4}, the shear forces V and the moments M of test CB180T16 was presented in Fig. 6. It was shown that both the bending moments and the shear forces at the critical cross-sections were large. Moreover, as only the webs of the C sections were bolted onto the webs of the inverted welded T sections, there was a discontinuity of load path in the flanges of the C sections. It should also be noted that a sudden moment transfer from the C sections to the inverted welded T sections took places at the critical cross-sections through the first row of bolts.

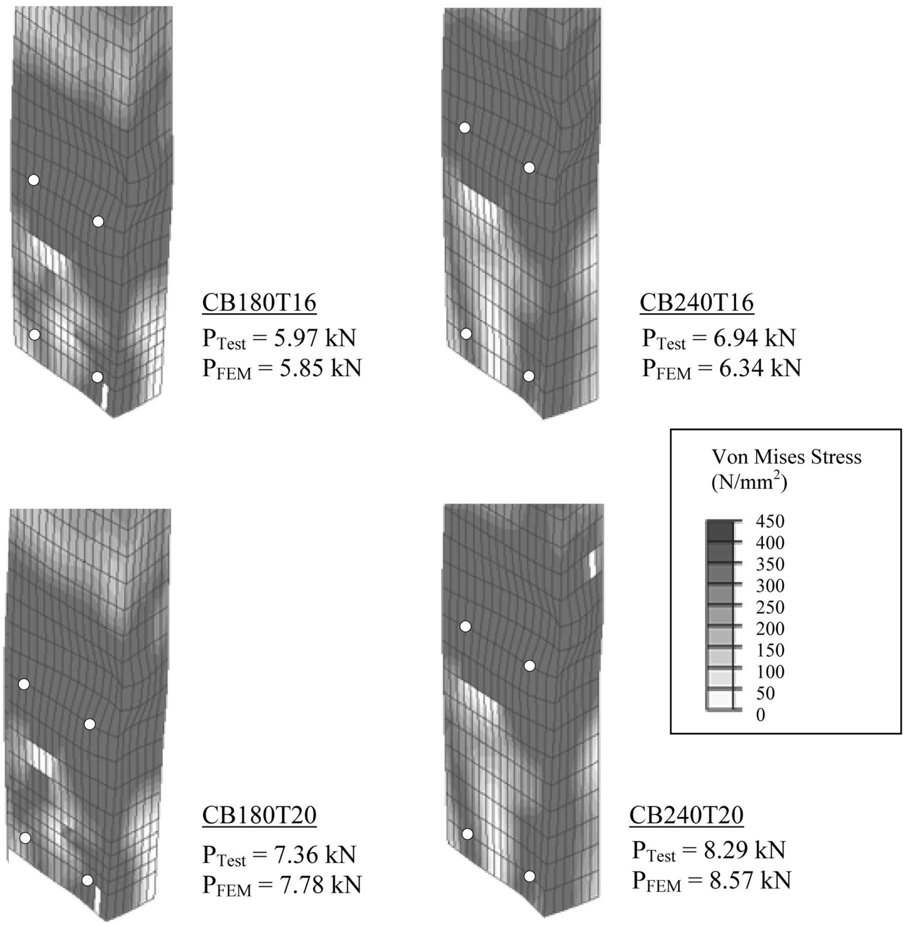


Fig. 5 Critical cross-sections of column base connections

Table 2 Summary of load resistances

Test	Measured load resistance P_{Test} (kN)	Predicted load resistance P_{FEM} (kN)	Model factor ψ_{FEM}	Predicted load resistance P_{Design} (kN)	Model factor ψ_{Design}
CB180T16	5.97	5.85	1.02	5.71	1.05
CB240T16	6.94	6.34	1.09	6.00	1.16
CB180T20	7.36	7.78	0.95	6.81	1.08
CB240T20	8.29	8.57	0.97	7.39	1.12

Notes:
a) P_{FEM} is determined with finite element models.
b) P_{Design} is determined with proposed design and analysis method.
c) The model factors are defined as follows:

$$\psi_{FEM} = \frac{P_{Test}}{P_{FEM}}$$

$$\psi_{Design} = \frac{P_{Test}}{P_{Design}}$$

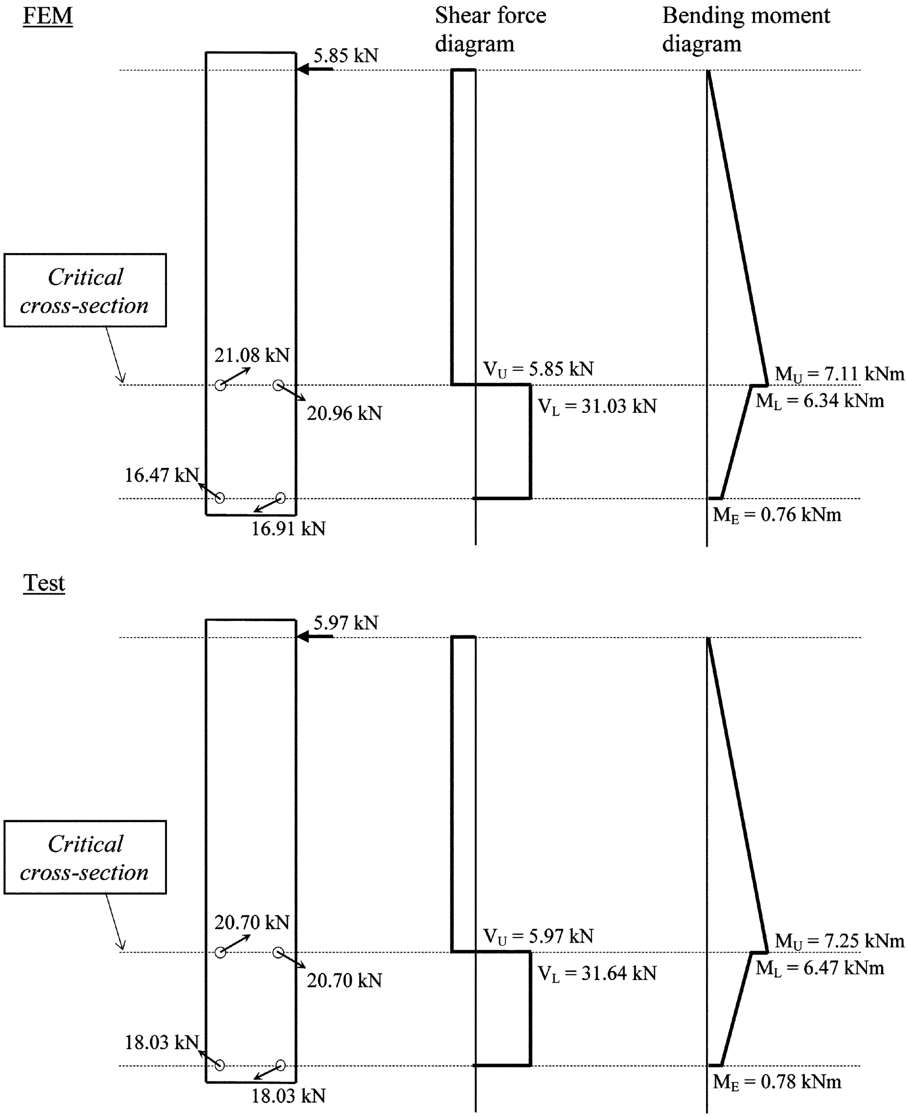


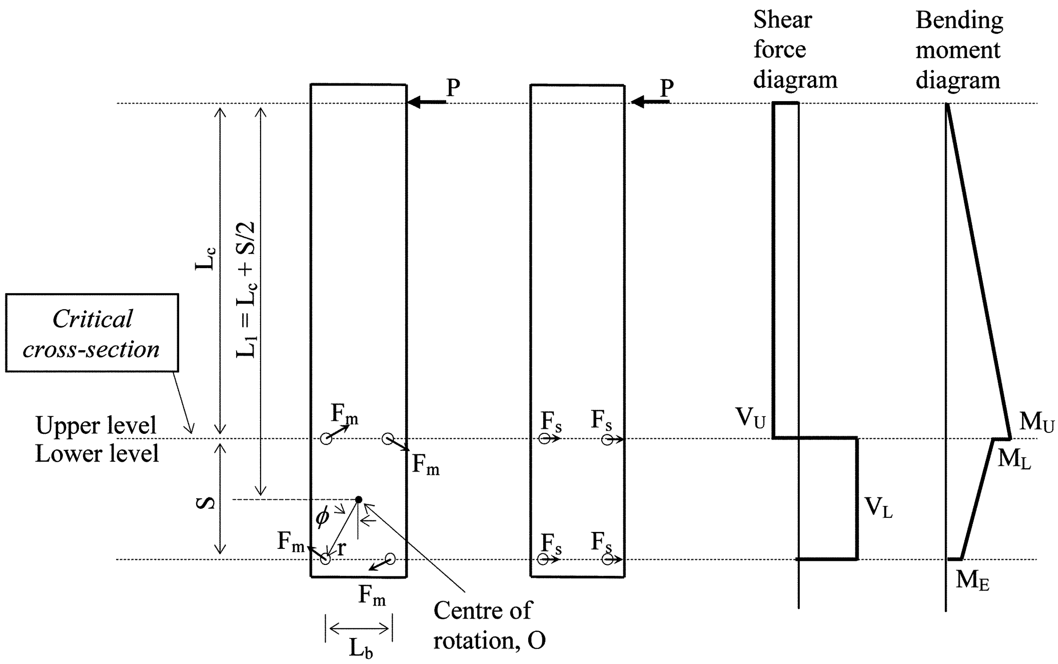
Fig. 6 Internal force distribution of Test CB180T16

5. Proposed analysis and design method

Based on an extensive experimental and theoretical investigation on bolted moment connections between cold-formed steel lapped Z sections, an analysis and design method for internal force distribution of the connections is presented by Chung and Ho (2004) for practical design of purlin systems. The proposed method is adopted in the present study, and re-formulated for the analysis and design of column base connections using cold-formed steel double C sections back-to-back. Consider the column base connection shown in Fig. 7, the following assumptions are adopted:

a) The centre of rotation of the connection, O, is coincided with the bolt group center.

b) The magnitude of the bolt force due to moment, F_m, is proportional to the distance between the bolt



Bolt force due to moment, F_m

$$P \times L_1 = 4 \times F_m \times r$$

$$\Rightarrow F_m = \frac{R\,L_1}{4\,r}$$

$$F_{m,h} = F_m \times \cos\phi$$

$$F_{m,v} = F_m \times \sin\phi$$

Bolt force due to shear, F_s

$$F_s = P / 4$$

$$V_U = P$$

$$V_L = 2 \times \left(F_s - F_{m,h}\right)$$

$$M_U = P \times L_c$$

$$M_L = M_U - F_{m,v} \times L_b$$

$$M_E = F_{m,v} \times L_b$$

Fig. 7 Proposed design and analysis method

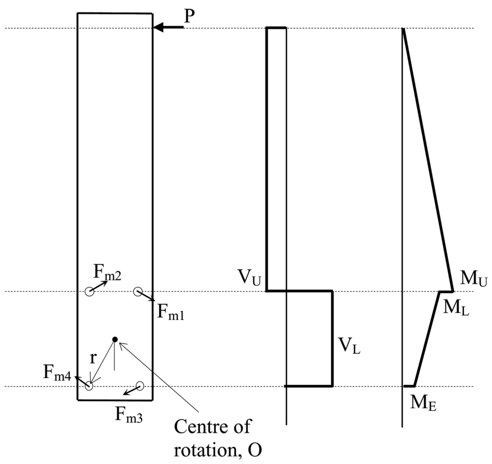
hole and the centre of rotation of the connection, O. Moreover, the direction of F_m is derived from the moment equilibrium consideration of the connection.

c) Similarly, both the magnitude and the direction of the bolt force due to lateral shear force, F_s, are obtained from the force equilibrium consideration of the connection.

After attaining force and moment equilibriums, all the internal forces within the connections are plotted in Fig. 7. Both the shear force and the bending moment diagrams are then readily established, and they are also presented in Fig. 7 together with all the relevant expressions of internal forces. According to the maximum applied loads measured from tests, the corresponding shear forces V_U and V_L, and moments M_U and M_L at the upper and the lower levels of the critical cross-sections of the column base connections respectively are summarized in Table 3. The internal force distribution of test CB180T16 obtained from the proposed method is also illustrated in Fig. 6 for direct comparison with that obtained from finite element modelling. It should be noted that the resultant forces due to both F_m and F_s in the four bolts are different in both magnitude and direction according to both the finite element model and the proposed method.

Table 3 Summary of internal forces within column base connections

Test series		P (kN)	F_{m1} (kN)	F_{m2} (kN)	F_{m3} (kN)	F_{m4} (kN)	V_U (kN)	V_L (kN)	M_U (kNm)	M_L (kNm)	M_E (kNm)
CB180T16	Test	5.97	20.70	20.70	18.03	18.03	5.97	-31.64	7.25	6.47	0.78
	Numerical	5.85	20.96	21.08	16.91	16.74	5.85	-31.03	7.11	6.34	0.76
CB240T16	Test	6.94	18.89	18.89	15.65	15.65	6.94	-28.85	8.02	7.47	0.55
	Numerical	6.34	17.72	17.81	13.82	13.90	6.34	-26.36	7.32	6.84	0.50
CB180T20	Test	7.36	26.71	26.71	23.42	23.42	7.36	-41.13	9.42	8.41	1.01
	Numerical	7.78	28.46	28.56	24.01	23.60	7.78	-43.48	9.96	8.89	1.07
CB240T20	Test	8.29	23.62	23.62	19.74	19.74	8.29	-36.43	10.11	9.43	0.68
	Numerical	8.57	24.67	24.75	19.81	19.87	8.50	-37.36	10.37	9.67	0.70



For structural adequacy of the column base connections, both the applied shear forces and the applied moments at the critical cross-sections should be checked against their respective section capacities as follows:

$$v_U = \frac{V_U}{V_{c,U}} \le 1.0 ; \qquad v_L = \frac{V_L}{V_{c,L}} \le 1.0 \qquad \text{(2a \& 2b)}$$

$$m_U = \frac{M_U}{M_{c,U}} \le 1.0 ; \qquad m_L = \frac{M_L}{M_{c,L}} \le 1.0 \qquad \text{(2c \& 2d)}$$

where

v_U, v_L are the shear force ratios for the upper and the lower levels of the critical cross-section respectively;

m_U, m_L are the moment ratios for the upper and the lower levels of the critical cross-section respectively;

$V_{c,U}$, $V_{c,L}$ are the design shear capacities at the upper and the lower levels of the critical cross-section respectively; and

$M_{c,U}$, $M_{c,L}$ are the design moment capacities at the upper and the lower levels of the critical cross-section respectively.

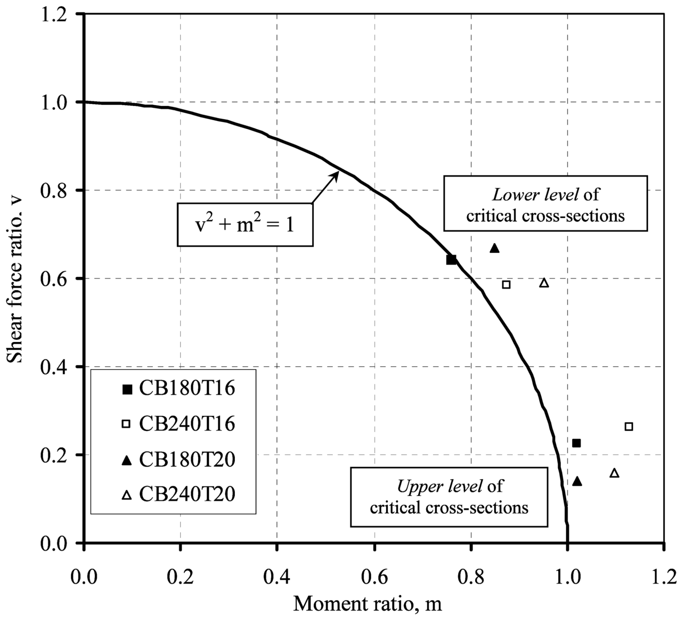


Fig. 8 Interaction curve for combined bending and shear

The critical cross-section of the column base connection is defined as the cross-section at the first row of bolts in the connection, as shown in Fig. 7. It should be noted that as the C sections below the critical cross-section is firmly attached to the webs of the T sections through two rows of bolts, local buckling is unlikely to occur, and hence, $V_{c,L}$ should be taken as the plastic shear capacity of the C sections while $M_{c,L}$ should be taken as the gross moment capacity of the C sections. Moreover, allowance for the presence of bolt holes in the critical cross-sections should be made in evaluating both $V_{c,L}$ and $M_{c,L}$. In order to allow for the restraining effect against shear buckling at the C sections below the critical cross-section due to the presence of the T sections, the design expression against shear buckling of cold-formed steel sections developed for lapped Z sections is adopted.

Furthermore, it is also necessary for both the upper and the lower levels of the critical cross-section to be checked against combined bending and shear as follows:

$$v_U^2 + m_U^2 \leq 1.0 \tag{3a}$$

$$v_L^2 + m_L^2 \leq 1.0 \tag{3b}$$

It should be noted that while the design rules for both shear and moment capacities differ significantly between BS5950: Part 5 and Eurocode 3: Part 1.3, the same quadratic interaction curve for combined bending and shear is adopted in both codes, as shown in Fig. 8.

Table 4 summarizes the shear force ratios and the moment ratios at both the upper and the lower levels of the critical cross-sections of the column base connections of all the tests, and they are plotted in Fig. 8 together with the quadratic interaction curve for combined bending and shear in order to allow for direct comparison. It should be noted that combined bending and shear is often critical at the C sections below the critical cross-sections while bending is always critical at the C sections above the critical cross-sections. Hence, both the bending and the shear effects are very significant near the critical cross-sections, and both effects should be properly quantified in analysis and allowed for in design.

Table 4 Back analysis of test data

Test	P_{Test} (kN)	V_U (kN)	M_U (kNm)	$\frac{V_U}{V_{c,U}}$	$\frac{M_U}{M_{c,U}}$	χ_U	V_L (kN)	M_L (kNm)	$\frac{V_L}{V_{c,L}}$	$\frac{M_L}{M_{c,L}}$	χ_L
CB180T16	5.97	5.97	7.25	0.22	1.02	1.09	31.64	6.47	0.64	0.76	0.99
CB240T16	6.94	6.94	8.02	0.26	1.13	1.34	28.85	7.47	0.59	0.87	1.11
CB180T20	7.36	7.36	9.42	0.14	1.02	1.06	41.13	8.41	0.67	0.85	1.17
CB240T20	8.29	8.29	10.11	0.16	1.10	1.23	36.43	9.43	0.59	0.95	1.26

Notes:
a) A single cold-formed steel C section is considered due to symmetry.
b) χ_1 and χ_2 are the checking values of Eqs. 3(a) and 3(b) respectively.
c) All the shear and the moment resistances are determined according to BS5950: Part 5: 1998 as follows:

	C15016 G450	C15020 G450
Design moment resistance, $M_{c,U}$ (effective section) (kNm)	7.11	9.23
Design moment resistance, $M_{c,L}$ (gross section) (kNm)	8.54	9.89
Design shear resistance, $V_{c,U}$ (shear buckling) (kN)	26.54	51.83
Design shear resistance, $V_{c,L}$ (plastic shear yielding) (kN)	49.25	61.56

In order to establish the adequacy of the proposed method, a model factor, ψ_{Design}, is established which is defined as follows:

$$\psi_{Design} = \frac{\text{Measured load resistance from test, } P_{Test}}{\text{Load resistance obtained from the proposed method, } P_{Design}} \tag{4}$$

The design resistances, P_{Design}, of the column bases derived from the proposed method are also summarized in Table 2 together with the corresponding model factors, ψ_{Design}. It is shown that the values of ψ_{Design} range from 1.05 to 1.16 with an average value of 1.10. Hence, the proposed method is shown to be structurally adequate, and applicable to practical bolted moment connections. However, the proposed method should only be used in those bolted connections with essentially similar configurations to those investigated in the present study.

6. Conclusions

In order to investigate the structural behaviour of cold-formed steel column base connections, a total of four column base connection tests with different practical connection configurations were tested under lateral loads. Among all the tests, section failure of the connected C sections at the critical cross-sections of the connections was found to be always critical. A finite element model was established to predict the structural behaviour of the connections, and both the bolt forces and the internal force distribution along the lengths of the connected C sections were determined after extensive data post-processing. Comparison on the load resistances obtained from both the tests and the models was found to be highly satisfactory.

Moreover, an analysis and design method was proposed to determine the bolt forces of the connections together with both the shear forces and the bending moments of the connected C sections. Based on the quadratic moment shear interaction curve, the predicted load resistances of the column base connections were found to be very close to both the measured and the numerical resistances.

Consequently, both the finite element models and the proposed analysis and design method were considered to be effective to assess the lateral load resistances of typical column base connections, and they are readily available to engineers to design efficient moment connections between cold-formed steel members. However, the proposed method should only be used in those bolted connections with essentially similar configurations to those investigated in the present study.

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