Load-carrying Capacity Estimation on Cold-formed Thin-walled Steel Columns with Built-up Box Section

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Abstract

The use of cold-formed thin-walled steel structural members has increased in recent years, and most of their sections are open section with only one symmetrical axis, which would likely fail by twisting and interaction with the others buckling mode, such as local buckling and distortional buckling. To improve the ultimate strength of columns, built-up box section can be used. In this paper, a series of loading capacity tests on high-strength cold-formed steel columns with built-up box section are presented, including 21 axially-compressed columns and 19 eccentrically-compressed columns subjected to bending moments about weak axis as well as strong axis. The test specimens are built up by two channel sections with two intermediate stiffeners in the web, and they connect at their flanges using self-drilling screws. It was shown that distortional buckling and twisting do not occur and the ultimate load-carrying capacity is 10 to 20 percent higher than the sum of the ultimate load-carrying capacity of each lipped channel section columns. According to the test results and theoretical analysis, an improved method based on the suggestion of current China code 'Technical code of cold-formed thin-walled steel structures' (GB50018-2002) considering the plate-coupling effect was proposed to estimate the ultimate load-carrying capacity of built-up box section column. With the proposed method, the calculated results are close and conservative to the test results.

Introduction

The use of high-strength cold-formed thin-walled steel structural members has increased in re-cent years, especially in low-rise and multi-story residential buildings and portal steel frame structures. High strength steel sections have higher strength, lower ductility, and larger width-to-thickness

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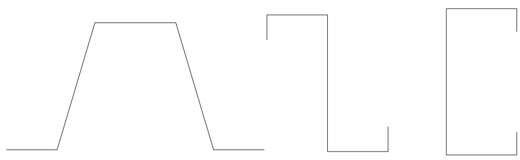
ratio, which are different from the ordinarily used cold-formed thin-walled steel. The national design code for cold-formed thin-walled steel structures, ‘Technical code of cold-formed thin-wall steel structures’(GB50018-2002), have no provision to estimate the ultimate load-carrying capacity of high-strength cold-formed thin-walled members with thickness less than 2mm. Meanwhile, cold-formed steel section are usually formed in singly-,point-,or non-symmetric open sections as shown in Fig.1. These open sections have a relatively small torsional stiffness compared to closed sections. So open sections would likely fail by twisting and interaction with the others buckling mode, such as local buckling and distortional buckling, depending on the dimension of the cross sections and the length of the members. Box-shaped sections made by connecting two channel sections tip to tip are often found in using in cold-formed steel structures due to their relatively large torsional stiffness and their favorable radius of gyration about both principal axes (1977). But when the width-to-thickness ratio of the built-up sections is relatively large, local buckling will decrease the full section strength of the member. Therefore, the cold-formed steel built-up closed sections with two intermediate stiffeners in the web are investigated in this paper.



(a) Hat section (b) Z-section (c) Channel section

Fig.1 Cold-formed steel section

In the past decades, there are many test data on cold-formed thin-walled steel open section columns performed by researchers all over the world, such as Rhodes and Harvey(1977), Thomasson(1978), Mulligan and Peköz(1984), Lau and Hancock(1988), Weng and Peköz(1990), Kwon and Hancock(1992),Young and Rasmussen(1998), Young(2005), and some other researchers as summarized by Yu(2000). Meanwhile, the high strength cold-formed thin-walled channel sections columns with two inter-mediate stiffeners in the web were researched by SHEN and LI (2008). However, not many test data have been reported on cold-formed thin-walled steel built-up closed section columns. De Wolf et al.(1974) conducted column tests on cold-formed steel box-shaped sections built up by two plain channel sections connected at their flanges. The webs of the box-shaped sections were flat and local buckling occurred during the tests. The column strengths were influenced by local buckling. However, the use of intermediate stiffeners could improve the situation. Ben Young et al(2008) proposed the design methods of cold-formed thin-walled steel built-up closed sections with one intermediate stiffener. The web of build-up closed sections displayed the distortional buckling which can reduce the ultimate strength of members. Therefore, the behavior and design of cold-formed thin-walled steel built-up sections with two intermediate stiffeners in the webs are investigated in this paper.

Experimental investigation

Material properties

The structural steel grade of the test sections is G550 (AS1397-2001). The minimum specified yield stresses of the test sections is 550MPa. Tensile coupons tested were previously conducted for flat coupons cut from the fabricated sections. The coupons were prepared and tested according to the Chinese Standard, 'Metallic materials—Tensile testing—Method of test at ambient temperature' (GB/T228-2002). All coupons were tested in a 20kN capacity displacement controlled testing machine. The coupon test results are shown in table 1 and the typical stress-strain relations are shown in Fig.2. The 0.2% proof stress was used as the corresponding yield stress in calculating the design strength of columns. The table 1 contains the experimental yield stress($f_{0.2}$), ultimate tensile strength(f_u), initial Young's modulus(E), and elongation after fracture(δ). The experimental yield stress (2% offset) were higher than the nominal yield stress. The elongation ranged from 10.7%-11.7% with the average being 11.2%, which is significantly lower than that of mild steel.

Table1 Material properties of columns

Specimen	t (mm)	$f_{0.2}$ (MPa)	f_u (MPa)	E (MPa)	Elongation Ratio δ
S1001	1.00	613	623	2.02	11.70%
S1002	1.00	617	619	2.14	10.70%
S1003	1.00	615	618	1.98	11.10%

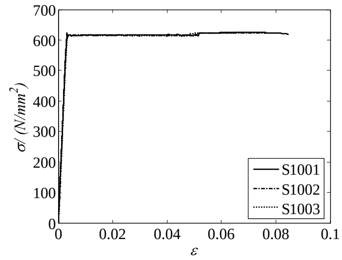


Fig. 2 Typical stress versus strain relation

Specimen tests

The test specimens of built-up sections were first brake pressed from structural steel sheets to form the channel sections with two intermediate stiffeners in the web, then two of the channel sections were connected at their flanges using self-drilling screws to form the built-up sections, as shown in Fig.3. The space of screws was 300 or 600mm. The sections selected are used as the main members in low-rise and multi-story residential buildings in Chinese. The section geometry of lipped channel sections and the built-up section are given in Fig.3. The detailed actual cross-sectional dimensions are summarized in table 2 and table 3 for axially-compressed and eccentrically-compressed columns respectively. The internal radii of the corners and intermediate stiffeners are 4.0, 5.0, and 2.5mm for r_1, r_2, and r_3 respectively. The nominal widths of the flange, web, and lip of the test sections are 50, 100, and 12mm respectively. The nominal section properties are shown in table 4. The nominal length of the test members are from 200mm to 3000mm. The test specimens were labeled so that the thickness of specimens, the height of web, approximate slenderness ratio of specimens, load patters, axial of instability and sequence number of same specimens could be included. DS means build-up sections. The first four numbers of the specimen label indicates the height and thickness of specimens, the second two numbers refers to the approximate slenderness ratio, the third letters indicates the load patters, the next last letter displays the instability axial, the sequence number of same specimens is appended at the label end, such as DS1010-30-AC-Y-1 as showed in Fig.4.

Table 2 Geometries of axially compressed specimens

specimen	Nominal length(mm)	Actual length(mm)	web(mm)		flange(mm)		lip(mm)	
			h_1	h_2	b_1	b_2	a_1	a_2
DS1010-10-	200	197.5	100.09	98.82	53.07	49.56	13.22	12.20
AC-Y-1	200	197.5	101.23	103.62	53.31	50.21	12.77	13.03
DS1010-10-	200	197.9	100.65	100.01	53.00	49.93	13.38	12.21
AC-Y-2	200	197.9	100.78	99.74	53.08	49.89	13.58	11.68
DS1010-30-	600	596.8	99.89	99.27	53.44	49.83	13.37	12.08
AC-Y-1	600	596.8	100.13	98.85	53.41	49.92	12.89	12.10
DS1010-30-	600	596.8	100.07	98.80	53.56	49.59	12.80	11.98
AC-Y-2	600	596.8	99.97	98.49	53.36	49.86	13.25	12.32
DS1010-30-	600	598.9	100.53	100.11	53.59	49.81	12.82	12.26
AC-Y-3	600	598.9	99.91	99.60	53.62	49.95	13.31	11.93
DS1010-50-	1000	997.5	99.97	98.97	53.44	49.62	13.07	12.03
AC-Y-1	1000	997.5	100.25	99.55	53.39	49.76	13.10	12.49
DS1010-50-	1000	997.0	100.18	99.24	53.38	50.00	13.60	11.60
AC-Y-2	1000	997.0	100.16	99.37	53.46	49.74	13.12	12.02
DS1010-75-	1500	1497.0	99.81	98.27	53.39	49.10	12.80	11.64
AC-Y-1	1500	1497.0	99.77	98.41	36.27	49.53	12.66	11.66
DS1010-75-	1500	1497.0	99.81	98.27	53.39	49.10	12.80	11.64
AC-Y-2	1500	1497.0	99.77	98.41	52.94	49.53	12.66	11.66
DS1010-75-	1500	1500.0	101.62	102.58	52.89	49.24	12.12	13.27
AC-Y-3	1500	1500.0	101.43	103.27	52.62	49.27	12.04	12.70
DS1010-100-	2000	2000.0	99.79	98.63	53.27	49.34	12.89	11.46
AC-Y-1	2000	2000.0	99.82	99.39	52.91	49.28	13.21	11.29
DS1010-100-	2000	2000.0	99.81	98.27	53.39	49.10	12.80	11.64
AC-Y-2	2000	2000.0	99.77	98.41	52.94	49.53	12.66	11.66
DS1010-100-	2000	2000.0	100.93	100.03	52.20	49.10	11.85	13.20
AC-Y-3	2000	2000.0	100.74	99.81	52.31	48.71	11.13	13.21
DS1010-120-	2500	2500.0	100.96	100.19	53.06	49.75	12.08	13.61

AC-Y-1	2500	2500.0	100.92	100.04	53.00	49.55	12.51	13.53
DS1010-120-	2500	2500.0	100.78	100.05	53.07	49.55	11.82	13.85
AC-Y-2	2500	2500.0	100.92	100.24	53.00	49.52	12.14	13.57
DS1010-150-	3000	3000.0	99.78	98.45	53.46	49.79	13.67	11.55
AC-Y-1	3000	3000.0	100.89	100.27	53.61	49.47	11.73	13.27
DS1010-150-	3000	3000.0	100.71	99.85	53.19	49.37	11.87	12.91
AC-Y-2	3000	3000.0	100.57	99.09	53.27	49.25	11.93	13.25
DS1010-50-	2000	1997.0	99.85	98.63	52.96	49.49	12.71	11.39
AC-X-1	2000	1997.0	99.80	98.67	53.25	49.31	12.67	11.82
DS1010-50-	2000	2000.0	99.75	97.91	52.85	49.58	12.96	11.82
AC-X-2	2000	2000.0	99.74	98.39	53.13	49.43	12.40	11.88
DS1010-75-	3000	3000.0	100.63	99.78	53.45	49.37	11.97	13.07
AC-X-1	3000	3000.0	99.84	98.98	53.90	49.96	11.64	12.92
DS1010-75-	3000	3000.0	99.92	98.46	53.53	49.83	12.76	11.62
AC-X-2	3000	3000.0	100.86	100.46	53.66	49.48	11.45	13.23

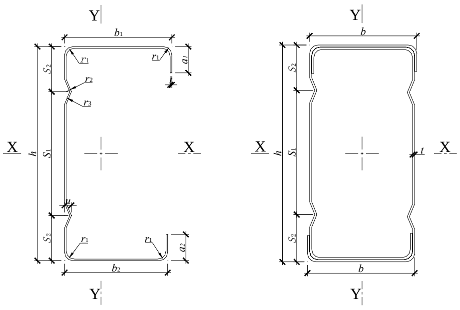
Table 3 Geometries of eccentrically compressed specimens

specimen	Nominal length (mm)	Actual length (mm)	web(mm)		flange(mm)		lip(mm)	
			h_1	h_2	b_1	b_2	a_1	a_2
DS1010-50-	1000	996.8	100.02	98.85	53.53	49.79	12.61	11.76
EC1-Y-1	1000	996.8	100.09	98.08	53.36	49.83	13.14	12.17
DS1010-50-	1000	997.0	100.67	99.63	53.64	49.82	13.11	12.13
EC1-Y-2	1000	997.0	100.12	99.67	53.64	49.82	13.11	12.13
DS1010-100-	2000	2000.0	100.74	99.96	52.49	48.80	11.54	13.40
EC1-Y-1	2000	2000.0	100.78	100.08	52.37	49.32	11.94	13.36
DS1010-100-	2000	2000.0	100.93	100.03	52.20	49.10	11.85	13.20
EC1-Y-2	2000	2000.0	70.74	99.81	52.31	48.71	11.13	13.21
DS1010-150-	3000	3000.0	99.72	99.00	53.09	49.45	12.82	11.48
EC1-Y-1	3000	3000.0	99.75	98.53	52.97	49.33	12.76	11.46
DS1010-150-	3000	3000.0	99.76	98.42	52.97	49.47	12.35	11.79
EC1-Y-2	3000	3000.0	99.75	98.28	53.36	49.49	12.71	11.45
DS1010-15-	600	598.0	100.97	101.55	52.21	49.30	11.43	13.51
EC1-X-1	600	598.0	102.14	101.23	52.26	49.87	11.60	13.23
DS1010-15-	600	603.0	100.96	101.43	52.57	49.44	11.54	13.40
EC1-X-2	600	603.0	100.97	101.39	52.81	49.64	11.58	13.27
DS1010-25-	1000	1000.0	101.25	101.59	53.56	50.13	11.54	13.37
EC1-X-1	1000	1000.0	101.07	101.71	53.43	50.27	11.29	13.51
DS1010-25-	1000	1000.0	100.96	101.46	52.09	49.78	11.42	13.69
EC1-X-2	1000	1000.0	101.03	101.56	51.98	49.41	11.47	13.58
DS1010-35-	1400	1396.0	99.75	98.63	52.80	49.64	12.88	11.54
EC1-X-1	1400	1396.0	99.80	98.74	53.04	49.14	12.63	11.38
DS1010-35-	1400	1400.0	101.17	101.93	52.86	49.35	11.54	13.29
EC1-X-2	1400	1400.0	100.63	101.38	52.21	49.15	11.67	13.47
DS1010-50-	2000	2000.0	100.77	100.11	52.36	49.24	11.66	13.26
EC1-X-1	2000	2000.0	101.05	99.89	51.96	49.56	11.97	13.33
DS1010-50-	2000	2000.0	100.74	99.75	52.45	49.08	11.41	13.31
EC1-X-2	2000	2000.0	100.00	98.78	51.44	48.66	11.50	12.74
DS1010-65-	2500	2500.0	100.73	100.04	53.14	49.58	12.28	13.28
EC1-X-1	2500	2500.0	100.78	100.00	52.88	49.50	12.42	13.62
DS1010-65-	2500	2498.0	99.90	98.71	53.37	49.36	12.78	11.36
EC1-X-2	2500	2498.0	99.83	98.44	53.30	49.62	12.43	11.33
DS1010-65-	2500	2498.0	99.90	98.71	53.37	49.36	12.78	11.36
EC1-X-3	2500	2498.0	99.83	98.44	53.30	49.62	12.43	11.33
DS1010-75-	3000	3000.0	99.70	98.51	53.83	49.42	12.75	11.47
EC1-X-1	3000	3000.0	100.81	100.44	53.59	49.68	11.69	13.50

DS1010-75-	3000	3000.0	100.75	100.06	53.77	49.53	11.78	13.14
EC1-X-2	3000	3000.0	101.11	100.76	53.60	49.62	12.04	13.18

Table4 Property of nominal section

Size of specimen	A/mm^2	i_x/mm	i_y/mm	I_x/mm^4	I_y/mm^4
SS1010	218	41.01	18.26	366725	72708
DS1010	436	41.02	20.34	733856	180378



(a) channel section (b) build-up section

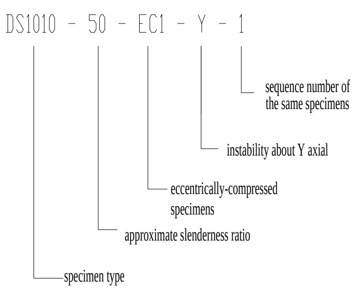


Fig.3 Sectional geometries Fig.4 Specimen labeling rule

The 300kN capacity servo-controlled hydraulic testing machine system was used to apply compressive force for the stud specimens with length of 200mm. Hydraulic jack and support frame were used to apply compressive force for the other specimens. Load, strain, and displacement were recorded automatically by a date acquisition instrument and showed directly on the screen of the computer in this system. After geometric and physical alignment completed, compressive loads can be subjected onto specimens by increments until the failure of them. Loading modes include three types, axial compression and eccentrical compression about strong and weak axial. Eccentrical value equals to half of the radius of gyration.

The braces were fixed to prevent the member from bending along Y axis and rotation about X axial for the members bending about the strong axis(X axial).

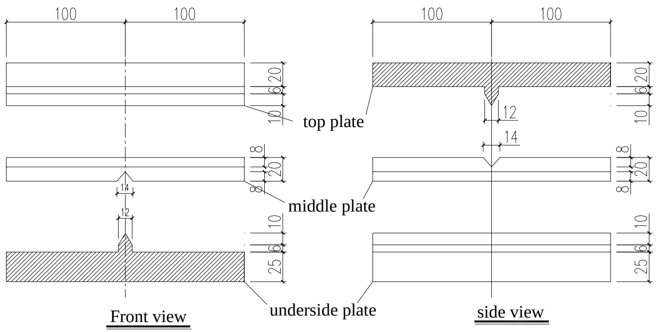


Fig.5 The bidirectional-hinged support

(a)Top support seat (b) below support seat
Fig.6 The actual bidirectional-hinged support

The tested members were bidirectional-hinged at each end supported with three plates, as shown in Fig.5 and Fig.6. A hoop-plate was applied in the test in order to avoid crush occurring at the end of these too thin members as shown in Fig.7.

Strain gauges and lateral displacement transducers were placed at mid-height of the columns, as shown in Fig.8. Furthermore, four axial displacement transducers were employed to measure axial shortening and rotation of support seat. These strain gauges were used for alignment and to confirm buckling stress and experimental loading eccentricity.

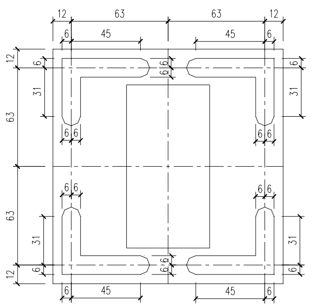
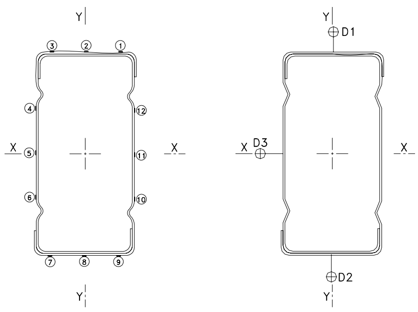


Fig.7 hoop-plate of specimens



(a)Strain gauges (b) displacement transducer
Fig.8 Gauges arrangement

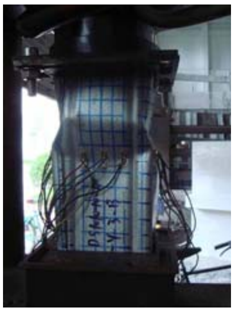
Test results

Axially compressive columns

(1)The specimens with the length being less than 500mm are considered as stud columns. Load and deformation were linear when load was applied, and the magnitude of the deformation was low. As the load was increased, one single channel column yielded firstly, and another single channel column also yielded subsequently. Then the built-up columns failed.

The cooperative ability of the two single channel columns was weak. The failure mode of stud columns is shown in Fig9a.

(2)The load and deformation of the intermediate columns whose slenderness ratio are less than 50 were linear when load was applied, and the magnitude of the deformation was low. As the load increased gradually to the ultimate load, crippling failure occurred abruptly. The failure mode of intermediate columns is shown in Fig9b.




(a) DS1010-10-AC-Y-2 (b) DS1010-50-AC-Y-2


(c) DS1010-150-AC-Y-1 (d) DS1010-75-AC-X-2

Fig.9 Buckling mode of axially compressed specimens

(3)The final failure shapes of the long columns whose slenderness ratio are more than 50 were mainly the overall flexural buckling mode about the weak axial. The load and deformation of the long columns were linear when load was applied. As the load was increased, local buckling occurred in the lips and the larger deformation occurred in the middle of the specimens. And then, the loads were up to the maximum, the flanges and the webs failed. The specimens displayed a significant post-buckling strength reserve. The failure mode of long columns is shown in Fig9c.

(4)Load and deformation of the specimen of instability about the strong axis were linear when load was applied. As the load was increased, local buckling occurred in the web and flange in the middle of the specimens firstly. The deformation increased gradually due to elastic local buckling.

Then the transverse displacement along the Y axial increased, instability about strong axial occurred. The failure mode of columns instability about the strong axis is shown in Fig9d.

(5)The ultimate strength have little different for specimens with different space of connecting screw. The ultimate strength with 300mm in the space of connecting screw were little higher than that of 600mm.

(a)DS1010-50-EC1-Y-1 (b) DS1010-50-EC1-Y-2

Fig.10 Buckling of eccentrically compressed specimens

Eccentrically compressive columns

(1)The failure modes of all specimens were flexural buckling as shown in Fig. 10. The load and deformation of columns were linear when load was applied. As the load increased gradually to the ultimate load, crippling failure occurred abruptly.

(2)As the load was increased, one single channel column yielded firstly, and another single channel column also yielded subsequently for the stud columns. Then the built-up columns failed. The cooperative ability of the two single channel columns is weak.

(3)Local buckling occurred for most of the specimens with eccentricity about weak axial because of the larger width-to-thickness.

(4)The space of connecting screw had nothing to do with the ultimate strength of the specimens.

Comparison of test strengths with design strengths

Introduction of design methods

Three different design methods are used to estimate the ultimate strength of the build-up sections specimens and all use the Chinese current code 'Technical Code of Cold-formed Thin-Walled steel structures' (GB50018-2002) considering the plate-coupling effect: 1) The ultimate strength of the built-up section is equal to the total of the ultimate strength of two single channel sections, 2) The load-carrying capacity is the ultimate strength of built-up section with flange considered as stiffened element, and the thickness of flange of built-up section is equal to the total of thickness of flange of two single channel sections, 3) The load-carrying capacity is the ultimate strength of built-up section with flange considered as partially stiffened element, and the thickness of flange of built-up section is equal to the total of thickness of flange of two single channel sections.

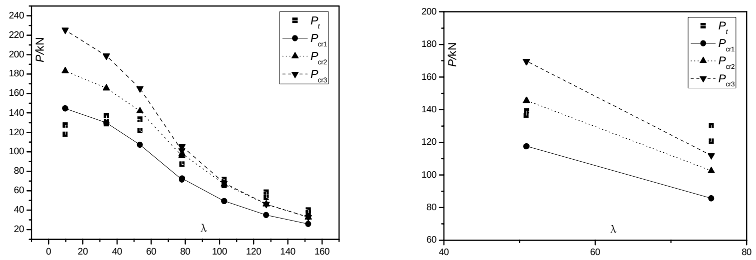
Design methods compared with test results

The design ultimate load-carrying capacity P_{Cr1}, P_{Cr2}, and P_{Cr3} of the total 21 axially compressive and 19 eccentrically compressive specimens were shown in Table 5 and Table 6, where P_{Cr1} is the total of the ultimate strength of two single channel sections and P_{Cr2}, and P_{Cr3} are the ultimate strength of built-up sections considering the flanges as stiffened element and partially stiffened element based on Chinese current code 'Technical Code of Cold-formed Thin-Walled steel structures' (GB50018-2002) considering the plate-coupling effect respectively. Meanwhile, the ultimate load-carrying capacities of tests are shown in Table 5 and Table 6. The test strengths of the cold-formed steel built-up section axially and eccentrically compressive columns are compared with the nominal design strengths obtained using the Chinese code in different methods, as shown in Fig. 11 and Fig.12 respectively.

Table 5 Comparison of axially compressive columns between test results and calculated values

Specimen	λ	P_t/kN	P_{cr1}/kN	P_{cr2}/kN	P_{cr3}/kN
DS1010-10-AC-Y-1	9.71	118.00	144.73	225.61	183.34
DS1010-10-AC-Y-3	9.73	127.50	144.51	225.27	183.21
DS1010-30-AC-Y-1	33.77	128.78	129.96	198.74	165.61
DS1010-30-AC-Y-2	33.77	137.25	129.92	198.59	165.40
DS1010-30-AC-Y-3	33.87	130.60	129.85	198.85	165.43
DS1010-50-AC-Y-1	53.47	133.58	107.22	165.04	142.15
DS1010-50-AC-Y-2	53.44	121.84	107.14	165.11	142.20
DS1010-75-AC-Y-1	78.02	96.46	71.39	101.25	95.87
DS1010-75-AC-Y-2	78.02	87.25	72.18	105.09	97.46
DS1010-75-AC-Y-3	78.17	87.20	72.66	105.57	98.00
DS1010-100-AC-Y-2	102.75	71.50	49.25	67.81	66.72
DS1010-100-AC-Y-3	102.75	65.46	49.24	67.81	66.73
DS1010-100-AC-Y-4	102.75	65.89	49.38	67.83	66.61
DS1010-120-AC-Y-2	127.34	58.61	35.02	46.17	46.17
DS1010-120-AC-Y-3	127.34	52.88	34.97	46.09	46.09
DS1010-150-AC-Y-1	151.92	39.99	25.78	32.74	32.74
DS1010-150-AC-Y-2	151.92	36.96	25.77	32.67	32.67
DS1010-50-AC-X-1	50.88	136.55	117.52	169.66	145.38
DS1010-50-AC-X-2	50.95	139.30	117.62	169.67	145.57
DS1010-75-AC-X-2	75.33	120.74	85.71	111.96	102.59
DS1010-75-AC-X-3	75.33	130.40	85.59	111.82	102.48

As shown in Table 5 and Fig. 11, the specimens with slenderness ratio less than 50 have less cooperative ability to work together, and the ultimate load-carrying capacity of the specimens with length being 200mm are even lower than that of total of two single channel sections. But the ultimate load-carrying capacity of the specimens with slenderness ratio more than 50 are agreement with the ultimate strength estimated using the Chinese code considering flange as partially stiffened element and higher 20 percent than the total ultimate strength of two single channel sections.

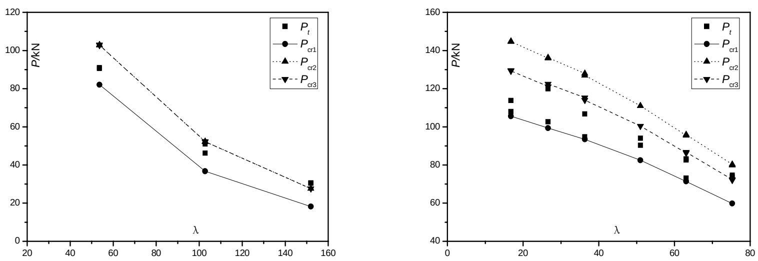


(a) Instability about weak axial (b) Instability about strong axial

Fig.11 Comparison of axially compressed columns between test results and calculated values

Table 6 Comparison of eccentrically compressive columns between test and calculated values

specimen	λ	P_t/kN	P_{cr1}/kN	P_{cr2}/kN	P_{cr3}/kN
DS1010-50-EC1-Y-1	53.6	91.0	82.1	103.0	103.0
DS1010-50-EC1-Y-2	53.6	90.6	82.1	102.8	102.8
DS1010-100-EC1-Y-1	102.8	46.2	36.7	52.3	52.3
DS1010-100-EC1-Y-2	102.8	51.0	36.7	52.2	52.2
DS1010-150-EC1-Y-1	151.9	30.4	18.2	27.6	27.6
DS1010-150-EC1-Y-2	151.9	30.6	18.2	27.6	27.6
DS1010-15-EC1-X-1	16.8	113.8	105.6	144.8	129.6
DS1010-15-EC1-X-2	16.8	108.0	105.6	144.7	129.3
DS1010-25-EC1-X-1	26.6	102.6	99.3	136.0	121.2
DS1010-25-EC1-X-2	26.6	120.0	99.3	136.3	122.5
DS1010-35-EC1-X-1	36.3	94.8	93.4	128.0	115.3
DS1010-35-EC1-X-2	36.3	106.8	93.4	127.0	114.0
DS1010-50-EC1-X-1	51.0	94.0	82.5	111.0	100.4
DS1010-50-EC1-X-2	51.0	90.3	82.5	111.0	100.4
DS1010-65-EC1-X-1	63.1	73.1	71.4	95.5	86.3
DS1010-65-EC1-X-2	63.1	82.6	71.4	95.9	86.7
DS1010-65-EC1-X-3	63.1	83.2	71.4	95.9	86.7
DS1010-75-EC1-X-1	75.3	74.6	59.8	80.3	72.4
DS1010-75-EC1-X-2	75.3	73.8	59.8	79.9	72.0



(a) Instability about weak axial (b) Instability about strong axial

Fig.12 Comparison of eccentrically compressive columns between test results and values

As shown in Table 6 and Fig. 12, the test results of the eccentrically compressive specimens are intermediate between that of total of two single channel sections and that predicted using the Chinese code considering flange as stiffened element and higher 10 to 20 percent than the total ultimate strength of two single channel sections.

Proposed design methods

For built-up section column formed with two channel sections with two intermediate stiffeners in the web, a purposed design method is presented to estimate its ultimate strength based on comparison with ultimate strength between test results and results predicted using three different design methods.

For the axially compressed built-up columns, ultimate load-carrying capacity equal to the total of ultimate strength of single open section if the column is instability about strong axial or weak axial with slenderness ratio being less than 50, and the ultimate load-carrying capacity equal to 1.2 times the total of ultimate strength of single open section if the column is instability about weak axial with slenderness ratio being more than 50.

For the eccentrically compressed built-up columns， ultimate load-carrying capacity equal to the total of ultimate strength of single open section if the column is instability about strong axial, and the ultimate load-carrying capacity equal to 1 or 1.2 times the total of ultimate strength of two single columns (eccentricity prone to the web and lip) if the column is instability about weak axial with the slenderness ratio being less or more than 50 respectively.

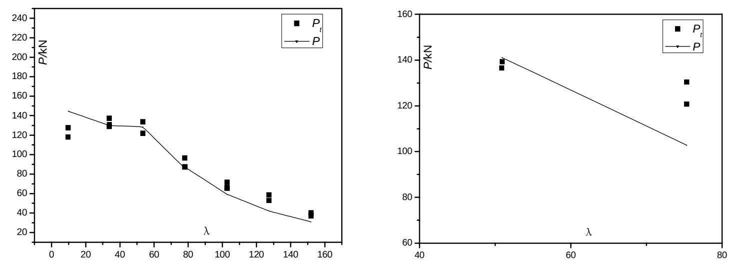
The comparison with ultimate strength between test results and results predicted using the proposed design method are shown in Fig.13, Fig.14 and Table 7, Table 8 for the axially and eccentrically compressive columns respectively. P_t is test results and P is obtained with proposed design methods.

Table 7 Comparison of axially compressed columns between test results and calculated values by proposed method

specimen	λ	P_t/kN	P/kN	P_t/ P	specimen	λ	P_t/kN	P/kN	P_t/ P
DS1010-10-AC-Y-1	9.7	118.00	144.73	0.82	DS1010-100-AC-Y-3	102.7	65.46	59.09	1.11
DS1010-10-AC-Y-3	9.7	127.50	144.51	0.88	DS1010-100-AC-Y-4	102.7	65.89	59.26	1.11
DS1010-30-AC-Y-1	33.7	128.78	129.96	0.99	DS1010-120-AC-Y-2	127.3	58.61	42.02	1.39
DS1010-30-AC-Y-2	33.7	137.25	129.92	1.06	DS1010-120-AC-Y-3	127.3	52.88	41.96	1.26
DS1010-30-AC-Y-3	33.8	130.60	129.85	1.01	DS1010-150-AC-Y-1	151.9	39.99	30.94	1.29
DS1010-50-AC-Y-1	53.5	133.58	128.66	1.04	DS1010-150-AC-Y-2	151.9	36.96	30.92	1.20
DS1010-50-AC-Y-2	53.4	121.84	128.57	0.95	DS1010-50-AC-X-1	50.9	136.55	141.02	0.97
DS1010-75-AC-Y-1	78.0	96.46	85.67	1.13	DS1010-50-AC-X-2	50.9	139.30	141.14	0.99
DS1010-75-AC-Y-2	78.0	87.25	86.62	1.01	DS1010-75-AC-X-2	75.3	120.74	102.85	1.17
DS1010-75-AC-Y-3	78.2	87.20	87.19	1.00	DS1010-75-AC-X-3	75.3	130.40	102.71	1.27
DS1010-100-AC-Y-2	102.7	71.50	59.10	1.21	DS1010-100-AC-Y-3	102.7	65.46	59.09	1.11

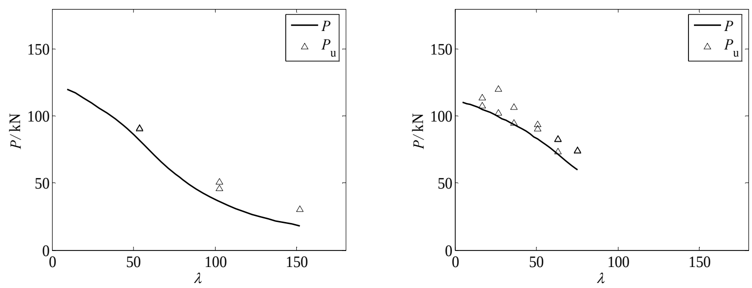
Table 8 Comparison of eccentrically compressed columns between test results and calculated values by suggested method

specimen	λ	P_t/kN	P/kN	P_t/ P	specimen	λ	P_t/kN	P/kN	P_t/ P
DS1010-50-EC1-Y-1	53.6	91.0	98.52	0.92	DS1010-35-EC1-X-1	36.3	94.8	93.4	1.01
DS1010-50-EC1-Y-2	53.6	90.6	98.52	0.92	DS1010-35-EC1-X-2	36.3	106.8	93.4	1.14
DS1010-100-EC1-Y-1	102.8	46.2	44.04	1.05	DS1010-50-EC1-X-1	51.0	94.0	82.5	1.14
DS1010-100-EC1-Y-2	102.8	51.0	44.04	1.16	DS1010-50-EC1-X-2	51.0	90.3	82.5	1.09
DS1010-150-EC1-Y-1	151.9	30.4	21.84	1.39	DS1010-65-EC1-X-1	63.1	73.1	71.4	1.02
DS1010-150-EC1-Y-2	151.9	30.6	21.84	1.40	DS1010-65-EC1-X-2	63.1	82.6	71.4	1.16
DS1010-15-EC1-X-1	16.8	113.8	105.6	1.08	DS1010-65-EC1-X-3	63.1	83.2	71.4	1.16
DS1010-15-EC1-X-2	16.8	108.0	105.6	1.02	DS1010-75-EC1-X-1	75.3	74.6	59.8	1.24
DS1010-25-EC1-X-1	26.6	102.6	99.3	1.03	DS1010-75-EC1-X-2	75.3	73.8	59.8	1.23
DS1010-25-EC1-X-2	26.6	120.0	99.3	1.21					



(a) Instability about weak axial (b) Instability about strong axial

Fig.13 Comparison of axially compressed columns between test results and calculated values by suggested method



(a) Instability about weak axial (b) Instability about strong axial

Fig.14 Comparison of eccentrically compressed columns between test results and calculated values by suggested method

As shown in Fig.13, Fig.14 and Table 7, Table 8, the ultimate strength estimated using proposed methods are close to the test results for the axially and eccentrically compressed columns respectively. So the proposed methods could be used to calculate the ultimate strength of high strength cold-formed thin-walled built-up section columns safety.

Conclusion

A total 21 axially and 19 eccentrically compressed built-up columns were experimental and theoretical studied in this paper. On the base of comparison with ultimate strength between test results and results calculated using proposed design methods, the follow conclusions can be presented.

(1) The cold-formed thin-walled steel built-up sections column made by connecting two channel sections with two intermediate stiffeners in the web at their flanges using self-drilling screws are found having higher ultimate capacities due to their relatively large torsional rigidity and their favorable radius of gyration about both principal axes.

(2) The cold-formed thin-walled steel built-up sections column with larger slenderness ratio has great cooperative ability to work together. The ultimate strength of built-up section columns can increase 20 percent than the total of ultimate load-carrying capacity of single open section members.

(3) For the axially compressed built-up columns, ultimate load-carrying capacity equal to the total of ultimate strength of single open section if the column is instability about strong axial or weak axial with slenderness ratio being less than 50, and the ultimate load-carrying capacity equal to 1.2 times the total of ultimate strength of single open section if the column is instability about weak axial with slenderness ratio being more than 50.

(4)For the eccentrically compressed built-up columns，ultimate load-carrying capacity equal to the total of ultimate strength of single open section if the column is instability about strong axial, and the ultimate load-carrying capacity equal to 1or 1.2 times the total of ultimate strength of two single columns (eccentricity prone to the web and lip) if the column is instability about weak axial with the slenderness ratio being less or more than 50 respectively.

Notation

a_1, a_2 = width of lip (mm);
A = cross-sectional area (mm^2);
b_1,b_2 = width of flange(mm);
E = initial Young’s modulus (MPa);
$f_{0.2}$ = experimental yield stress (MPa);
f_u = ultimate tensile strength(MPa);
h_1, h_2 = width of web (mm);
i_x, i_y = radius of gyration(mm);
I_x, I_y = inertia moment about x and y axial (mm^4);
P_{Cr1}, P_{Cr2}, P_{Cr3}= ultimate strength (kN) calculated by three different method;

P_t = ultimate test load (kN);
P = ultimate strength (kN) calculated by the proposed method;
t = thickness of base metal (mm);
λ = slenderness ratio;
δ = elongation after fracture.

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