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DOI:

[10.1016/j.engstruct.2019.109528](https://doi.org/10.1016/j.engstruct.2019.109528)

Document Version

Accepted author manuscript

[Link to publication record in Manchester Research Explorer](#)

Citation for published version (APA):

Tan, J., Wang, Y., Su, M., Zhang, H., & Peng, Y. (2019). Compressive behaviour of built-up hot-rolled steel hollow and composite sections. *Engineering Structures*, 198, 109528. <https://doi.org/10.1016/j.engstruct.2019.109528>

Published in:

Engineering Structures

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Compressive behaviour of built-up hot-rolled steel hollow and composite sections

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Abstract: In this paper, a new type of steel section is proposed comprising adjacent H-sections linked together by intermediate steel plates welded along their flanges. Experiments on three hot-rolled steel built-up H-sections and five columns infilled with concrete are conducted, with results showing that both variants having much better ductility compared to conventional columns (i.e. hollow section columns and concrete filled steel tube (CFST) columns). Experimental results showed that the bearing capacity of the built-up hot-rolled steel hollow section column improved by 45.1% to 55% after it was filled with concrete, and the yield bearing capacities of the specimens ZC-4 and ZC-5 are 26.6% and 17.1% higher than that of ZC-1, which shows that the increase of material consumption can improve the structural behaviour of the specimen. A finite element software ABAQUS is used for parametric study. The newly developed finite element models were validated against test results in terms of failure modes and load-displacement curves. A total of 482 numerical results, 132 hot-rolled steel built-up section columns and 350 concrete-filled built-up section columns, have been generated. Based on the experimental and numerical results, the compressive behaviours of hot-rolled steel built-up section columns and concrete-filled built-up section columns are studied. A new design method for the compression capacity of these two types of columns is proposed. It is found that the newly proposed design method could provide accurate and consistent predictions – the mean value of predicted to experimental results are 0.963 for hollow section columns (with coefficient of variation being 0.061) and 1.04 for composite section columns (with coefficient of variation being 0.043).

Keywords: Built-up sections; Compression bearing capacity; Concrete-filled built-up section tube; H-section; Numerical modelling; Design methods

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1 Introduction

The hot-rolled steel built-up section column is a new type of column built up by welding hot-rolled steel H-section members and steel plates. These built-up section columns could be infilled by concrete and become concrete-filled steel built-up tube columns. The new built-up sections (see Fig. 1) can be directly welded by existing hot-rolled steel H-sections, and they enjoy benefits such as high bearing capacity, less welds, high construction efficiency and low construction cost compared to other built-up sections. Furthermore, this new type of built-up section has overhanging H-section flanges, which can be widely used in the frame system of assembled steel plate shear wall (see Fig. 2(a) and (b)), without additional joint construction. In addition, steel beams can also be directly connected with steel brackets welded on the H-section flanges in advance; this form of joint connection is simple with clear load path, as shown in Fig. 2(c).

A great number of experimental and theoretical studies on the compression performance of concrete-filled steel tube (CFST) columns or special-shaped columns could be found in the literature. Han et al. [1-2] carried out experimental studies on the axial compression behaviour of concrete-filled circular/square/rectangular steel tube columns. Considering the confining factor and concrete strength as variables, the design rules for the compression capacity of CFST with different cross-sections were proposed. A number of researches [3-7] have also been carried out to test T-shaped, L-shaped and cross-shaped CFST columns, and analyse the influence of various parameters on the stability of these special-shaped composite columns. The design rules for the stability bearing capacity of the aforementioned composite columns with axial restraint effect were developed by Liu et al. [3] and Han et al. [7]. Besides, researchers have also started to work on columns with more innovative shapes. For instance, Chen and Young [8, 9] conducted comprehensive investigations on the compressive behaviour of cold-formed steel novel-shaped sections (elliptical and semi-oval sections) through extensive experimental and numerical studies, based on which accurate modified design methods were proposed. Han et al. [10, 11] carried out compression tests on CFST stub columns of five special sections: triangular, semi-circular, D-shaped, 1/4 circular and fan-shaped sections. The results showed that the failure modes of these columns were similar to those of rectangular CFST stub columns. Tu et al. [12, 13] carried out a series of compression tests on multi-chamber CFST columns of short medium-length, and proposed a design method for the stability bearing capacity prediction of multi-chamber CFST composite T-pillars. Besides, there are some other experimental, numerical and theoretical investigations [14, 15] on the compression behaviour of combined cold-formed steel box section

columns. The summary of past research is shown in Table 1.

Although there have been extensive research studies on the compressive behaviour of concrete-filled steel tube columns with different shapes, no study is available on the concrete-filled steel built-up section columns comprised of H-sections. This new type of columns are flexible in shapes, which can be applied in various systems of buildings improve the utilization rate of indoor area and allow for more flexible arrangement of furniture. However, the new type of columns also suffer with disadvantages including large residual stress, difficult welding of the internal plates and the instability of their overhanging flanges. This study investigates the compressive behaviour of three hot-rolled steel built-up section columns and five concrete-filled steel built-up section columns. Numerical models for these columns were developed and validated against test results. Upon validation, the finite element (FE) models were used to perform an extensive parametric study to generate a total of 132 numerical results on hot-rolled steel built-up section columns and 350 numerical results on concrete-filled steel built-up section columns. Based on the newly generated test and numerical results, the design rules for the compression capacity are also proposed herein.

2 Experimental study

2.1 Test specimens

A total of eight specimens are designed for the test programme, of which specimens Z-1, Z-2 and Z-3 are bare steel built-up section columns, and specimens ZC-1 to ZC-5 are concrete-filled steel built-up section columns. A 10 mm thick end plate is welded to each end of the column. The top end plate has a number of holes concrete filling. The steel tubes of specimens Z-1 and ZC-1 are the same and welded from hot-rolled steel H-section members. The other specimens are welded from H-section steel members and steel plates. The geometries of the employed H-section (HW175×175) are shown in Fig. 3, while the nominal thickness of the utilised steel plate is 10 mm. The cross-sectional dimensions of all the specimens are presented in Fig. 4, where point “O” indicates the centroid of section. The material grade of both the H-section members and steel plates are Q235B, and the grade of the concrete infill is C40. The mix proportions of the concrete are 1: 0.93: 2.28: 0.39 (cement : sand: stone: water (by weight) and the measured slump value is 81 mm. The nominal height of all the columns is 600 mm. A series of tensile coupon tests was carried out to determine the material properties of the H-section members and steel plates according to the GB/T 228-2002 Standard [16]. The locations from which the coupon specimens were taken are shown in Fig. 5. The dimensions of the coupon specimens are presented in Fig. 6. The measured material properties of concrete and steel are presented in Tables 2 and 3, respectively.

2.2 Testing programme

The compression tests were conducted by servo controlled electro-hydraulic testing machine in the Structural Laboratory of Chongqing University. The test configuration and measurement scheme are shown in Fig. 7. The axial displacement of the specimens was measured by four displacement sensors (LVDT) (Fig. 7(a)). The strain of the specimen surface (i.e. the steel built-up section) was measured by strain gauges. Before testing, the specimens were geometrically centred and preloaded to one-tenth of the estimated ultimate load in order to check the operation condition of each instrument (Fig. 7(b) and (c)). During the loading process, multi-stage loading approach was adopted. In the elastic stage, specimens were loaded at 1/10 of the predicted ultimate capacity intervals. Upon observing the plastic behaviour of the specimen, the loading interval was changed to 1/20 of the predicted ultimate capacity. When the specimens were significantly deformed and severely damaged, the tests were stopped.

3 Experimental results and analysis

3.1 Structural behaviours and failure modes

The failure modes of all the specimens are shown in Figs. 8(a) to 8(h). As shown in Figs. 8(a)-8(c), specimens Z-1, Z-2 and Z-3 behaved in a similar way during loading. In the early stage of loading, cracks appeared on the surface near the both ends of the columns. When continuing to load to 90% of the ultimate load, the outstand flanges began to slightly buckle. Since the outward flanges were subjected to less constraint compared to flanges of internal H-sections, local buckling occurred first in the outward flanges. When the load continued to increase, the outstand flanges notably buckled, and the welds of the H-section flanges with large width to thickness ratio in the middle section of the specimen Z-1 was dislocated with "corrugated drum" (see Fig. 8(a)). Specimen Z-2 had single-wave-form buckling at the welded joint between steel plate and flange, as well as double-wave-form buckling at the corner steel plate (see Fig. 8(b)). As for specimen Z-3, single-wave-form buckling occurred at the corner steel plate and double-wave buckling occurred in the flange of H-section (see Fig. 8(c)). Because of the different positions of stiffener plates, the plates with large width to thickness ratio of Z-2 and Z-3 are in different positions, so Z-2 and Z-3 have different buckling deformation.

The internal steel plates in the specimens ZC-1 to ZC-5 were subjected to the lateral restraint of the concrete, and therefore only the out-of-plane local buckling occurs. The failure mode of the specimen was local buckling. The steel plate formed by welding of a steel plate to an H-section flange with large width to thickness ratio of specimens ZC-2 to ZC-5 buckling occurred earlier. As shown in Figs. 8(e) and 8(f), the specimens ZC-2 and ZC-3 had similar failure modes with ZC-5.

As the load increases, the H-section flanges of specimen ZC-2 buckled locally at the welding part of the steel plates, but no local buckling were observed on steel plates at the corner. The two steel plates on the outer side of the specimen ZC-3 buckled near the mid-height, and cracks appeared in the fillet weld of the steel plate at the middle height of the specimen. As shown in Figs. 8(d), 8(g) and 8(h), the specimens ZC-1, ZC-4 and ZC-5 had similar loading responses. Similarly, at the early stage of loading, minor cracks occurred near both ends of the columns. As the load increased, the outstand flanges began to slightly buckle. When the load continued to increase, the outstand flanges of H-section showed notable buckling, and the joints of the H-section flanges in the middle sections of specimens began to have outward local buckling. The flanges of the H-sections at the end of the specimens ZC-1, ZC-4 and ZC-5 display buckling deformation, and local buckling was observed at welding place at the steel plate and H-section flange of mid-height specimens. In the late stage of specimens loading, the welding seam had continuous peeling phenomenon. The ends of all specimens were welded to end plates, which provide restraint to the ends of the specimens. Therefore, the buckling of the specimens was mainly observed at the mid-height of columns.

3.2 Load-displacement curves

The load-displacement curves of the specimens Z-1 to ZC-3 are shown in Fig. 9(a) to 9 (f). Comparing specimen Z-1 (see Fig. 9 (a)) with specimen ZC-1 (see Fig. 9 (d)), it can be seen that the stiffness and ductility of the concrete-filled specimen are found to be improved, and the yield bearing capacity is also increased by 45.1%, which shows that the increase of material consumption can improve the structural behaviour of the specimen. It can be seen from Fig. 9 (b) and 9 (c) that the ultimate bearing capacity of specimen Z-2 is improved compared to that of specimen Z-3, indicating that stiffening of short steel plate is effective to improve the buckling bearing capacity of steel plate. Although the amount of material is the same, it can be seen from Fig. 9 (e) and Fig. 9(f) that the ductility and bearing capacity of ZC-3 is lower than that of ZC-2, with the main reason being that the spacing of internal steel plate in ZC-2 is smaller, which offers a better constraint on concrete. specimen ZC-2 is 52.3% higher than that of specimen Z-2, and the bearing capacity of specimen ZC-3 is 54.9% higher than that of specimen Z-3. The yield bearing capacity and ultimate bearing capacity of pure steel specimens can be significantly enhanced after infilling concrete.

The yield bearing capacities of the specimens ZC-4 (see Fig. 9(g)) and ZC-5 (see Fig. 9(g)) are 26.6% and 17.1% higher than that of ZC-1, due to increasing of material consumption. However, the ductility of ZC-4 is lower than that of ZC-5, since the steel plate in the middle section of ZC-4 has large width to thickness ratio, which makes it easier to buckling under axial

compression. The ductility of both the concrete-filled columns are greater than that of the bare steel column. Meanwhile, the external steel tube can provide confinement to the concrete infill.

3.3 Load-strain curves

Taking specimens Z-1 and ZC-1 as examples, the load-strain responses of the considered columns during loading are analysed. Fig. 10 and Fig. 11 show the load-strain curves for the specimens Z-1 and ZC-1, respectively, where "negative value" represents compressive strain and "positive value" represents tensile strain. Please be noted that a discrepancy of strain readings was observed near the ultimate load, which is believed due to the reliability of electrical strain gauge measurement (especially during unbalance event, yielding, cracking, etc.) [17, 18].

As shown in Fig. 10, when the applied load is about 4000 kN (corresponding to the yield strain of the steel at $1600\ \mu\epsilon$), the strains of specimen Z-1 in three directions are similar and the specimen Z-1 is at the elastic stage. Continue to load to 80% of the ultimate load, the strains of the shear wall at the three directions started to differ from each other, the specimen is in the elastoplastic stage. During the loading process, the vertical strain at the measured point "4" changed from compression to tension (Fig. 10(b)). That was because that the steel plate at the measured point "4" is subjected to compression at early stage and in a state of complex stress as the load increases.

As shown in Fig. 11, the measured point "2" was located at the junction of the web and flange, and it had a strong constraint and a strong bearing capacity, the vertical strain at the measured point "2" was larger than those at other measured points. The steel plate at the flange joint at the measured point "5" was subjected to the out-of-plane expansion and compression of concrete, and the calculated horizontal and vertical strains are below the yield strain of steel.

4 Finite element analysis

4.1 Development of models

The finite element (FE) analysis was performed using ABAQUS package. The simplified secondary plastic flow model in literature [19] was adopted for steel material, and the stress-strain curve was divided into the elastic region, the yielding platform, the strain hardening region and the secondary yielding region and the parameters are determined according to the test results of steel properties. The specific mathematical expression is shown as follows:

$$\sigma = \begin{cases} E_s \varepsilon, & \varepsilon \leq \varepsilon_y \\ f_y, & \varepsilon_y < \varepsilon \leq 0.1 \varepsilon_y \\ 0.01 E_s (\varepsilon - \varepsilon_y) + f_y, & 0.1 \varepsilon_y < \varepsilon \leq k_2 \varepsilon_y \\ f_u, & \varepsilon > k_2 \varepsilon_y \end{cases} \quad (1)$$

$$k_2 = 0.1 + \frac{f_u - f_y}{0.01 E_s \varepsilon_y} \quad (2)$$

198 Where, σ is the stress of the steel plate; E_s is the elastic modulus of steel; f_y is the yield strength
199 of steel; f_u is the ultimate strength of steel; ε_y is the yield strain of steel.

200 The constitutive model of concrete subjected to compression and tension was also referred to
201 Liu [19]. And the stress-strain relationship of concrete under uniaxial compression can be
202 determined by Eq. (3).

$$y = \begin{cases} 2x - x^2 & (x \leq 1) \\ \frac{x}{\beta_0 (x-1)^\eta + x} & (x > 1) \end{cases} \quad (3)$$

$$x = \frac{\varepsilon}{\varepsilon_0}, y = \frac{\sigma}{\sigma_0}, \sigma_0 = f'_c, \xi = \sum \frac{f_y A_s}{f_{ck} A_c} \quad (4)$$

$$\varepsilon_0 = \varepsilon_c + 800 \cdot \xi^{0.2} \cdot 10^{-6} \quad (5)$$

$$\varepsilon_c = (1300 + 12.5 \cdot f'_c) \cdot 10^{-6} \quad (6)$$

$$\eta = 1.6 + 1.5/x \quad (7)$$

$$\beta_0 = \frac{(f'_c)^{0.1}}{1.2 \sqrt{1 + \xi}} \quad (8)$$

203 Where, ξ is confining factor; f'_c is the axial compressive strength of concrete cylinder; f_{ck} is the
204 characteristic strength of concrete.

205 In the FE models, the steel H-section and steel plate are simulated by eight-node uncoordinated
206 three-dimensional solid element (C3D8I), while the core concrete is simulated by eight-node
207 reduced integral three-dimensional solid element (C3D8R). The appropriate element sizes were
208 selected in the FE models based on the sensitivity analysis and the balance between accuracy and
209 time cost [20, 21]. Therefore, mesh size sensitivity analysis has been carried out in this study.
210 The column is pin-supported. The top end of the column is coupled with a reference point and the
211 lateral displacement of the reference point is limited. The load was applied to this reference point.
212 The normal direction of the interface between steel tube and concrete adopts the "hard
213 contact"[22], while the tangential direction adopts the coulomb friction contact with the friction
214 coefficient as 0.6 [19].

215 The residual stress of the hot-rolled steel built-up section columns and concrete-filled built-up

section columns due to welding [23-25]; the residual stresses were carefully measured and the codified predictive models were evaluated and the initial global geometric imperfection of the specimens were considered in the FE model. The residual stress is added to the FE model in the form of initial stress and the size and distribution of residual stress in the FE model are shown in Fig. 12 [1]. The residual stress distribution are incorporated for both flanges and steel plates. The initial geometric imperfection was taken as the first global buckling mode obtained by the buckling analysis by ABAQUS. The magnitude of the imperfection was chosen as 1/1000 of the height of the specimen [7].

4.2 Model validation

Fig. 13 shows the failure modes of the specimens obtained by FE modelling, which agree well with the failure modes observed in experimental programme. Fig. 9 compares the numerical and experimental load-deformation curves of all the specimens. Although the failure modes shown in Figs. 8 and 13 are practically identical, all graphs of Fig. 9 presented the numerical result a little more rigid than the experimental one. Table 4 shows that comparison of yield bearing capacities, which are determined based on the yield displacement of the load-displacement curve calculated by the Park method [26]. Good agreements between the tests and FE simulation have been achieved on the initial stiffness, load-displacement curves and yield bearing capacities.

5 Parametric study

In order to study the effects of material strengths and section slenderness on the ultimate bearing capacity of the specimens, this study analyses the key parameters by parametric study based on the validated FE model. In the parametric study, except that the steel constitutive model is modified to an elastic-perfectly plastic model, other parameters and boundary conditions of the FE models were kept the same. Considering that the effect of material strength on the ultimate bearing capacity is similar for different specimens, only the specimen ZC-1 was taken as an example for investigation. However, as for the effects of member slenderness ratio, hollow section columns and concrete-filled section columns were studied based on the specimens Z-1 and ZC-1, respectively.

5.1 Effects of steel yield strength and concrete strength

The load-displacement curves of the specimen ZC-1 with different material strengths are shown in Fig. 14. The steel yield strength and concrete strength have significantly influence on the bearing capacity and structural behaviour of the specimen. The yield strength grade of different steels had minor effect on the initial stiffness of the specimen. With the increase of the yield strength of steel,

the ultimate bearing capacity of the specimen gradually increased. Fig. 14(b) also shows that the concrete strength has limited effect on the initial stiffness of the specimens. With the increase of the concrete strength, the ultimate bearing capacity of the specimen increased gradually, and the displacement corresponding to the ultimate bearing capacity also increased. The higher the concrete strength grade, the bigger the displacement corresponding to the peak load point. After the peak load point, with the increased of displacement, the load-displacement curves showed a trend of asymptotic.

5.2 Effects of member slenderness ratio of hollow section column

Fig. 15(a) shows the relationship between the slenderness ratio λ , and λ is calculated according to Eq. (9) below:

$$\lambda = l \cdot \sqrt{f_y / 235} / \sqrt{I_{\min} / A} \quad (9)$$

Where, λ is the calculated slenderness ratio of the column; l is the calculated length of the columns; $I_{\min}$ is the minimum moment of inertia of cross section; A is the cross section area.

It can be seen that the larger the slenderness ratio, the lower the axial compressive capacity of the specimen and the difference of bearing capacity of different steel strength grade was smaller. Fig. 15(b) shows the normalised φ - λ relation curves of the specimens based on the steel strength grades. It can be seen that the normalised φ - λ curve of steels with different strength grades were basically the same, therefore, the same φ - λ curve can be used for the pure steel columns with different steel strength grades.

5.3 Effects of member slenderness ratio of composite sections

Fig. 16(a) shows the relationship between the slenderness ratio λ of the specimen ZC-1. It can be seen that with the increase of slenderness ratio, the bearing capacity of the specimen gradually decreases and the gap in the bearing capacity of specimens shows a decreasing trend. Fig. 16(b) showed the stability coefficient φ - λ curves based on the strength grade of the steel. With the increase of slenderness ratio, the stability coefficient of the specimen decreases gradually, but the stability coefficients of different steel strengths have little difference.

Fig. 16(c) shows the curves of the slenderness ratio λ and the bearing capacity considering the strength grade of the concrete of the specimen ZC-1. From Fig. 16(c), it can be seen that the normalized φ - λ curves demarcation point of the concrete-filled built-up section columns is larger than that of the hot-rolled steel built-up section columns and the steel strength utilization ratio can be improved and the premature elastic failure can be avoided by lateral restraint of concrete. It can

be seen that as the slenderness ratio increases, the bearing capacity difference of the specimens with different concrete strength grades becomes smaller. Fig. 16(d) shows the normalized $\phi-\lambda$ curves of the specimen ZC-1 based on the concrete strength grade. It can be seen that the concrete strength grade has no effect on the stability coefficient after normalization of the specimen when the slenderness ratio is relatively small. When $50 < \lambda < 100$, the higher the concrete strength grade, the smaller the stability coefficient, with the increase of slenderness ratio, the stability coefficient of the specimen decreases gradually.

6 Development of design rules

In this study, both experimental and numerical studies have been carried out on hot-rolled steel built-up section columns and concrete-filled steel built-up section columns. Effects of governing parameters such as steel strength, concrete strength and section slenderness, have been investigated. Based on the newly generated test and numerical results, design methods of compressive capacity and stability bearing capacity are developed for hot-rolled steel built-up section columns and concrete-filled steel built-up section columns.

6.1 Compression strength

The comparative analysis of the test yield bearing capacity and nominal yield bearing capacity are given in Table 4. The nominal yield bearing capacity N_n of the specimen is calculated according to Eq. (10) below:

$$N_n = A_s f_y + A_c f_{ck} \quad (10)$$

Where, A_s is the sectional area of steel; A_c is the area of concrete ($A_c = 0$ for pure steel specimens).

As can be seen in Table 4, the experimental yield bearing capacity of the steel built-up section columns is basically equal to the nominal yield bearing capacity, while the experimental yield bearing capacity of concrete-filled steel built-up section columns is greater than the nominal yield bearing capacity. It shows that the confinement effect of steel built-up section notably improves the strength of concrete. Fig.17 shows the classification of confinement effect of hot-rolled steel H-section plate. In this study, only the constraint effect of steel components directly contacting concrete is considered, and the contribution of H-section flange A_{s1} is ignored. The improvement of concrete strength is related to the confining factor of the specimen [2]. The specific calculation of the confining factor is shown in Eq. (11). Please note that the contribution of steel plate A_{s3} should be considered when calculating the confining factor of steel plate on concrete A_{c1} and A_{c2} . According to the calculation method of bearing capacity based on superposition theory proposed in [2], the bearing capacity of concrete-filled steel built-up section columns is regarded as the sum of the steel bearing capacity and the concrete bearing capacity considering the hoop

311 effect of steel tube. The detailed calculation was shown in Eq. (12) and Eq. (13).

$$\xi = \frac{f_y A_{sc}}{f_{ck} A_c} \quad (11)$$

$$N_u = f_y A_s + \theta \cdot f_{ck} A_c \quad (12)$$

$$\theta = 0.9\xi - 0.126, \quad (0.5 \leq \xi \leq 4) \quad (13)$$

312 Where, θ is the concrete strength enhancement coefficient; A_{sc} is the cross-sectional area of the
313 steel around the concrete; N_u is the bearing capacity of the axial compressive strength of the
314 columns.

315 Comparison between all test and predicted results are shown in Table 5. As can be seen from
316 Table 5, the design rules proposed in this paper is very accurate for compression strength and
317 coefficients of variation (COV) is very small. Fig. 18 shows the Regression curve for the
318 relationship between the confining factor and the concrete strength enhancement coefficient and
319 Fig. 19 shows comparison between predicted capacities and experimental (and numerical) results
320 respectively. It can be seen that the calculation method proposed in this paper is in good agreement
321 with the finite element calculation results, and has a relatively small error with the test results. The
322 formula presented in this paper can be directly used to calculate the axial compression bearing
323 capacity of concrete-filled built-up section columns.

324 6.2 Stability of columns under axial compression

325 6.2.1 Steel hollow sections columns

326 It can be seen from the analysis results of Fig. 20 that the stability bearing capacity of the columns
327 with different steel strength grades can be calculated by the same stability coefficient φ , and the
328 axial compressive bearing capacity of the pure steel columns can be calculated according to Eq.
329 (14). The calculation formulas are detailed in (15) to (16). As can be seen from Fig. 20(b), the
330 calculated data points are in good agreement with the calculation formula proposed in this paper.
331 Therefore, the calculation accuracy of the stability bearing capacity of hot-rolled steel H-section
332 columns is higher by using the calculation formula proposed in this paper.

$$N_\varphi = \varphi N_u \quad (14)$$

$$\varphi = \begin{cases} 1.0 - 7.1\lambda \cdot 10^{-4} - 0.92\lambda^2 \cdot 10^{-5} & (0 \leq \lambda \leq \lambda_p) \\ \frac{18365}{(\lambda + 21.2)^2} & (\lambda_p < \lambda \leq 400) \end{cases} \quad (15)$$

$$\lambda_p = 138 / \sqrt{f_y} \quad (16)$$

$$\lambda = l \cdot \sqrt{f_y / 235} / \sqrt{I_{\min} / A} \quad (17)$$

Where, N_ϕ is the stability bearing capacity of the steel columns; ϕ is stability coefficient; λ_p is the boundary slenderness ratio of the elastic-plastic failure and elastic failure of the columns: when $\lambda_p \geq \lambda \geq 0$, the specimen is in the elastic-plastic stage when it loses stability. When $\lambda > \lambda_p$, the specimen is elastically unstable.

As can be seen from Fig. 20(b), the calculated data points are in good agreement with the calculation formula proposed in this paper. Therefore, the formula presented in this paper is accurate for calculating the stability bearing capacity of hot-rolled steel H-section columns.

6.2.2 Composite sections columns

It can be seen from the analysis results in Fig. 16 that the steel strength grade has little effect on the normalized ϕ - λ curves of the columns. Therefore, the calculation formulae of the columns stability coefficient proposed in this paper only consider the influence of the concrete grade, and do not consider the influence of the steel grade. Therefore, the ϕ - λ curves of the hot-rolled steel H-section concrete-filled built-up section columns are shown in Fig. 21(a). In order to consider the influence of different concrete grades on the ϕ - λ curves, the variable β_c is introduced, and the stability bearing capacity of the steel-concrete composite columns can be calculated according to Eqs. (9)~(13).

$$N_{\phi c} = \beta_c \phi N_u \quad (18)$$

$$\phi = \begin{cases} 1.0 & (0 \leq \lambda \leq \lambda_0) \\ 1.0 - 5.92\lambda \cdot 10^{-4} - 2.95\lambda^2 \cdot 10^{-6} & (\lambda_0 \leq \lambda \leq 400) \end{cases} \quad (19)$$

$$\beta_c = \begin{cases} 1.0 & (0 < \lambda \leq 75) \\ -0.0016\lambda + A & (75 < \lambda \leq 400) \end{cases} \quad (20)$$

$$A = 0.069 \cdot \left(\frac{f_{ck}}{20}\right) + 0.74 \quad (20 \leq f_{ck} \leq 90) \quad (21)$$

$$\lambda_0 = \pi \sqrt{(220\xi + 450) / [(0.85\xi + 1.18) \cdot f_{ck}]} \quad (22)$$

Where, $N_{\phi c}$ is the stability bearing capacity of the steel-concrete composite columns; λ_0 is the boundary length ratio of plastic failure and elastic-plastic failure of the columns, β_c is the influence coefficient of concrete strength grade.

Fig. 21(b) and Fig. 21(c) show the regression curve for the relationship of $f_{ck}/20$ - A and β_c - λ , respectively. It can be seen that $f_{ck}/20$ - A and β_c - λ have a good linear relationship and the coincidence degree is better. Fig. 21(d) shows the comparison between the stability coefficient

regression curve and the finite element calculation data point proposed in this paper. It can be concluded that the two are in good agreement. Therefore, it is reasonable and safe to calculate the stability bearing capacity of the combined steel-concrete column with hot rolled h-section steel by using the formula proposed in this paper.

7 Conclusions

Three hot-rolled steel built-up section columns and five concrete-filled built-up section columns comprised of H-sections and steel plates were tested. Corresponding numerical models were developed and validated against test results. Experimental results showed that the outer flange of the H-section member at the corner of the specimen is weakly restrained and first buckled under compression load. The local stability, bearing capacity and ductility of pure steel columns was found to be improved significantly with infilled concrete: (1) The bearing capacity of specimen Z-1, Z-2 and Z-3 was improved by 45.1%, 52.3% and 54.9% with infilled concrete, respectively; (2) The yield bearing capacities of the specimens ZC-4 and ZC-5 are 26.6% and 17.1% higher than that of ZC-1, and the ductility of ZC-4 is lower than that of ZC-5 with the main reason being that the steel plate in the middle section of ZC-4 with large width to thickness ratio. Based on the experimental and numerical results, the design rules for hot-rolled steel built-up section stub columns and concrete-filled built-up section stub columns under compression were proposed based on the superposition theory. In addition, the design rules of the stability bearing capacity of the medium and long columns have also been developed. The new design rules are found to be accurate and consistent (mean $N_e/N_u = 0.963$ and $COV = 0.061$ for hollow sections and mean $N_e/N_u = 1.04$ and $COV = 0.043$ for composite sections), which can provide the calculation basis for the bearing capacity of such sections in practical engineering applications.

Acknowledgments

The research is supported by National Natural Science Foundation of China (51822804), Project of Chongqing commission of science and technology (cstc2018jscx-mszdx0099), the “111” Project (B18062) and the Fundamental Research Fund for the Central Universities (2019CDQYTM028).

Nomenclature

λ	the calculated slenderness ratio of the columns
l	the calculated length of columns
f_y	yield strength of the steel
f_u	the ultimate strength of steel

$I_{\min}$	the minimum moment of inertia of cross section
A	the cross section area
N_n	nominal yield bearing capacity
A_c	the area of concrete ($A_c = 0$ for pure steel specimens)
A_s	the sectional area of steel
f_{ck}	characteristic strength of concrete, defined as $f_{ck} = 0.76f_{cu}$
f'_c	the axial compressive strength of concrete cylinder.
θ	the concrete strength enhancement coefficient
A_{sc}	the cross-sectional area of the steel around the concrete
f_{cu}	characteristic 28-day cubic strength of concrete
N_u	the bearing capacity of the axial compressive strength of the columns
N_ϕ	the stability bearing capacity of the steel columns
$N_{\phi c}$	the stability bearing capacity of the steel-concrete composite columns
ξ	confining factor
ϕ	stability coefficient
λ_p	the boundary slenderness ratio of the elastic-plastic failure and elastic failure
λ_o	the boundary length ratio of plastic failure and elastic-plastic failure
β_c	the influence coefficient of concrete strength grade

References

- [1] Han Linhai. Theory and practice of concrete-filled steel tube structure. Beijing: Science Press, 2004.
- [2] Han Linhai, Yang Youfu. Modern concrete-filled steel tube structure technology. Beijing: China Architecture & Building Press, 2007.
- [3] Xianggang Liu, Chuangze Xu, Jiepeng Liu, Yuanlong Yang. Research on special-shaped concrete-filled steel tubular columns under axial compression. Journal of Constructional Steel Research, 2018, 147: 203-223.
- [4] Zhi-Liang Zuo, Jian Cai, ChunYang, Qing-Jun Chen, Gang Sun. Axial load behavior of L-shaped CFT stub columns with binding bars. Engineering Structures, 2012, 37: 88-98.
- [5] Cao Bing, Dai Shaobin, Huang Jun. Experimental study on axial compressive behaviour of improved composite T-shaped concrete-filled steel tubular columns. Journal of Building Structures, 2014, 35(11): 36-43.
- [6] Tu Yongqing, Wen Qianshan. Bearing capacity calculation of L shaped concrete filled steel tubular columns subjected to axial compression. Journal of Building Structures, 2013, 34(S1): 314-320.
- [7] Zhong Tao, Brian Uy, Lin-Hai Han, Zhi-Bin Wang. Analysis and design of concrete-filled stiffened thin-walled

steel tubular columns under axial compression. *Thin-Walled Structures*, 2009,47(12):1544-1556.

[8] Chen M T, Young B. Cross-sectional behavior of cold-formed steel semi-oval hollow sections. *Engineering Structures*, 2018, 177:318-330.

[9] Chen M T, Young B. Structural performance of cold-formed steel elliptical hollow section pin-ended columns, *Thin-walled Structures*, 2019, 136: 267-279.

[10] Fa-Cheng Wang, Lin-Hai Han. Analytical behaviour of special-shaped CFST stub columns under axial compression. *Thin-Walled Structures*, 2018, 129: 404-417.

[11] Qing-Xin Ren, Lin-Hai Han, Dennis Lamc, ChaoHoub. Experiments on special-shaped CFST stub columns under axial compression. *Journal of Constructional Steel Research*, 2014, 98: 123-133.

[12] Ma Liya, Tu Yongqing. Multi-chamber combination of L-shaped concrete filled steel tubular columns under axial compression Finite Element Analysis. *Journal of Building Structures*. 2013, 34(S1):306-313.

[13] Zhang Ning, Shen Yanfei, LI Pan, TU Yongqing. Study on performance of multi-cell composite T-shaped concrete-filled steel tubular slender columns under axial compression. *Journal of Building Structures*, 2015, 36(S1):254-261.

[14] Jessica Whittlek, Chris Ramseyer, Buckling capacities of axially loaded, cold-formed, built-up C-channels. *Thin-Walled Structures*, 2009, 47 (2):190-201.

[15] Soumia Kherbouche, Abdellatif Megnounif. Numerical study and design of thin walled cold formed steel built-up open and closed section columns. *Engineering Structures*, 2019,179:670-682.

[16] GB/T 228-2002, Metallic materials-Tensile testing at ambient temperature. General administration of quality supervision, inspection and quarantine of PRC, 2002.

[17] Sallam, H.E.M. (2010). Discussion of "Flexural strengthening of steel bridges with high modulus CFRP strips" by David Schnerch and Sami Rizkalla, J. of Bridge Engineering, 15(1), 117-117.

[18] Sallam H , Badawy A . Flexural Behavior of Strengthened Steel-Concrete Composite Beams by Various Plating Methods. *Steel Construction*, 2010, 66(8):1081-1087.

[19] Wei Liu. Research on mechanism of concrete-filled steel tubes subjected to local compression. School of Civil Engineering, Fuzhou University. Dissertation for the doctoral degree, 2005. In Chinese.

[20] M. Atta, A.A. Abd-Elhady, A. Abu-Sinna, H.E.M. Sallam. Prediction of failure stages for double lap joints using finite element analysis and artificial neural networks. *Engineering Failure Analysis*, 97, 242–257.

[21] El-Emam H , Elsis A , Salim H , Sallam H. Fatigue Crack Tip Plasticity for Inclined Cracks. *International Journal of Steel Structures*, 2018, 18(2):443-455.

[22] Su M N , Young B , Gardner L . Deformation-based design of aluminium alloy beams. *Engineering Structures*, 2014, 80:339-349.

[23] Sun, Y., Liang, Y., Zhao, O. (2019). Testing, numerical modelling and design of S690 high strength steel welded I-section stub columns. *Journal of Constructional Steel Research*, 159, 521–533.

[24] Sun, Y., Zhao, O. (2019). Material response and local stability of high-chromium stainless steel welded I-sections. *Engineering Structures*, 178, 212–226.

- 436 [25] Liang, Y., Zhao, O., Long, Y. and Gardner, L. (2019). Stainless steel channel sections under combined
437 compression and minor axis bending – Part 1: Experimental study and numerical modelling. Journal of
438 Constructional Steel Research, 152, 154–161.
- 439 [26] Tang Jiuru. Seismic Resistance of Joints in Reinforced Concrete Frames [M]. Southeast university press,1989.
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Fig. 1 Schematic drawing of cross-sections of tested columns

Fig. 2 Schematic diagram of steel frame system

Fig. 3 Dimensions of hot-rolled steel H-section

Fig. 4 Dimensions and detail information of tested specimens

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Fig. 5 Coupons taken from the hot-rolled steel
H-section

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Fig. 6 Dimensions of tensile coupons

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Fig. 7 Test setup and arrangement of measuring points

(a) Z-1

(b) Z-2

(c) Z-3

(d) ZC-1

(e) ZC-2

(f) ZC-3

(g) ZC-4

(h) ZC-5

Fig. 8 Failure modes of the tested specimens

Fig. 9 Experimental and numerical load-displacement curves

Fig. 10 Load-strain curves of specimen Z-1

Fig. 11 Load-strain curves of specimen ZC-1

Fig. 12 Measured residual stress distribution

Fig. 13 Failure modes of specimens obtained from finite element modelling

(a) Steel yield strength f_y

(b) Concrete strength f_{ck}

Fig. 14 Effects of material strengths on the loading behaviour of specimen ZC-1

Fig. 15 Effects of material strengths on the ϕ - λ curves of specimen Z-1

(a) Steel yield strength f_y

(b) The normalized ϕ - λ curves

(c) Concrete strength f_{ck}

(d) The normalized ϕ - λ curves

Fig. 16 Effects of material strengths on the ϕ - λ curves of specimen ZC-1

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Fig. 17 Constraint effect classification of the steel plate in specimens

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Fig. 18 Regression curve for the relationship between the confining factor and the concrete strength enhancement coefficient

Fig. 19 Comparison between predicted capacities and experimental (and numerical) results

(a) The typical φ - λ curves of pure steel specimens

(b) Comparison between calculation model and stability factor of
finite element results

Fig. 20 Regression curve for the relationship of stability coefficients φ between Eq.(5) and Finite Element of
steel hollow sections columns

(a) The typical φ - λ curves of composite specimens (b) the $f_{ck}/20$ - A curve (c) the β_c - λ curve

(d) Comparison between calculation model and stability factor of finite element results

Fig. 21 Regression results of stability coefficients φ between Eq.(11) and Finite Element of composite sections columns

Table 1 Summary of key findings in literature

Researchers	Specimens	Specimen parameters	Number of results	Key conclusions
Han et al. [1-2]	Concrete-filled circular/square/rectangular steel tube columns	f_y : 200-700 MPa f_{cu} : 30-120 MPa ξ : 0.2-5	Circular:356,square:240 and rectangular: 73(Exp)	- Proposal of new design rules.
Liu et al. [3]	L-shaped and T-shaped CFST stub columns	f_y : 200-700 MPa f_{cu} : 30-120 MPa ξ : 0.4-4	18 (Exp) and 160 (FE)	- Proposed design formulae.
Cai et al. [4]	L-shaped CFT stub columns with binding bars	f_y : 465 MPa f_{cu} : 47.60 MPa	7 (Exp)	- Setting binding bars could improve the bearing capacity and ductility of specimens by 7% and 26%.
Cao et al.[5]	Improved composite T-shaped concrete-filled steel tubular columns	λ : 9.6-22.4 ξ : 0.83-1.88Steel ratio : 0.097 - 0.209	18 (Exp)	- The strength grade of concrete was changed from C30 to C50, the axial bearing capacity of specimens was increased by about 15%. - Proposed a design method.
Tu et al. [6]	L-shaped CFST columns	f_y : 310-465 MPa f_{ck} : 36.2-45.7 MPa	6 (FE)	- D/B is about 2.5, steel tube has the best restraint effect on concrete.
Tao et al. [7]	Concrete-filled stiffened thin-walled steel tubular columns	f_{cu} : 25.5-51.2 MPa;	12 (Exp)	- DBJ13-51-2003 and Eurocode 4 are suitable for calculating the bearing capacity of tested composite columns.
Chen and Young [8]	Cold-formed steel semi-oval hollow section beams	f_y : 419-555 MPa f_u : 530-625 MPa	20 (Exp)	- Proposed accurate modified design methods.
Chen and Young [9]	Cold-formed steel novel-shaped section columns	f_y : 340-533 MPa f_u : 392-653 MPa	5 (Exp) and 20 (FE)	- Proposed the modified continuous strength method.
Han et al. [10]	Triangular, semi-circular, D-shaped, 1/4 circular and fan-shaped section columns	f_y : 384-441 MPa f_u : 532.1-538 MPa	44(Exp)	- The failure modes of these columns were similar to those of rectangular CFST stub columns.
Han et al. [11]	Triangular, semi-circular, D-shaped, 1/4 circular and fan-shaped section columns	f_y : 235-690 MPa f_c' : 24-70 MPa f_{ck} : 20-56 MPa	40(FE)	- Column ultimate strengths increase, with increasing of both material strengths, but higher strength enhancement efficiency as f_y increases
Tu et al. [12, 13]	Multi-cell composite T-shaped CFST slender and short columns	f_y : 313-333 MPa f_u : 398-422 MPa f_{cu} : 77.2 MPa; λ : 12.2-69.4	15(Exp)	- Proposed a simplified formula for bearing capacity.
Whittlek et al. [14]	Cold-formed, built-up C-channel columns	-	153(Exp)	- Conservatively designed members for the capacity specified based on the unmodified slenderness ratio and C4.5 fasteners and spacing.
Kherbouche et al. [15]	Thin-walled cold-formed steel built-up open and closed section columns	f_y : 310 MPa f_u : 436 MPa E_s : 210 GPa	71 (FE)	- Design method (DSM-t) are more conservative.

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Table 2 Material properties of concrete

Concrete cube (Unit: mm)	28-day compressive strength (MPa)			
	Specimen 1	Specimen 2	Specimen 3	Average
150×150×150	39.60	42.22	38.98	40.27

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Table 3   Material properties of steel

Specimens	Thickness(mm)	Yield strength(MPa)	Ultimate strength(MPa)	Young's Modulus(GPa)	Yield strain($\mu\epsilon$)
Flange of H-section	10.35	271	445	190	1421
Web of H-section	6.90	338	480	202	1673
Steel plate	9.18	335	475	196	1709

Table 4 Comparison of yield bearing capacity between test and predicted results

Specimens	Nominal area	Nominal area of	Nominal yield	Test yield	Finite	N_e / N_n	N_{FE} / N_e
	of steel	concrete	bearing	bearing	element		
	section(mm ²)	section(mm ²)	capacity	capacity	results		
			N_n (kN)	N_e (kN)	N_{FE} (kN)		
Z-1	14490	0	3983.4	4035	4046	1.01	1.01
Z-2	20332	0	6230.7	5903	6145	0.95	1.04
Z-3	20332	0	6230.7	5780	5912	0.93	1.02
ZC-1	14490	50964	5453.3	5853	5950	1.07	1.02
ZC-2	20332	88257	8776.3	8993	8880	1.02	0.99
ZC-3	20332	88257	8776.3	8951	9210	1.02	1.03
ZC-4	17250	74209	7048.4	7412	7523	1.05	1.02
ZC-5	17121	60384	6606.5	6854	7043	1.04	1.03

Table 5 Comparisons between all test and predicted results

Specimens		N_e	N_u	N_e/N_u	Mean, N_e/N_u	COV, N_e/N_u
Hollow sections	Z-1	4035	3983.4	1.01	0.963	0.061
	Z-2	5903	6230.7	0.95		
	Z-3	5780	6230.7	0.93		
Composite sections	ZC-1	5853	5453	1.07	1.04	0.043
	ZC-2	8993	8897	1.01		
	ZC-3	8951	8705	1.03		
	ZC-4	7412	7048.4	1.05		
	ZC-5	6854	6606.5	1.04		