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Design of square and rectangular CFST cross-sectional capacities in compression

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Abstract: Eurocode EN 1994-1-1 currently covers design of concrete-filled steel tubular (CFST) columns using normal strength materials with nominal steel yield strength (f_y) not more than 460 MPa and concrete characteristic cylinder strength less than 50 MPa. As high strength materials have been increasingly used in construction, this paper aims to extend current design equation for square and rectangular CFSTs in EN 1994-1-1 to incorporate high strength materials. A comprehensive experimental database consisting of 443 square and rectangular CFST stub column test results was developed. Statistical evaluations were undertaken and the results indicate that the current cross-section slenderness limit for square and rectangular CFSTs is too conservative and can be safely relaxed from $52\sqrt{235/f_y}$ to $68\sqrt{235/f_y}$. The design equation could be extended to incorporate high strength materials. Then, a reliability analysis in accordance with EN 1990 was performed to evaluate the applicability of the proposed design method. It was found that the current partial factors are safe for the design of square and rectangular CFSTs with normal strength materials even after relaxation of the cross-section slenderness limit. However, modification should be applied to the existing design equation for the design of high strength materials to yield an acceptable safety level. Finally, an experimental investigation on 10 stub column tests was carried out to further verify the applicability of the proposed design recommendation. It was found that the proposed design recommendation yields satisfactory predictions when compared to test results.

Keywords: Database; CFST; rectangular or square sections; stub columns; design; reliability analysis; partial factors.

1. Introduction

Concrete-filled steel tubular (CFST) members consist of outer steel tubes and infilled concrete cores. This type of arrangement optimises the use of both steel and concrete materials, making the complimentary interaction between steel tubes and the infilled concrete more efficient when compared to traditional reinforced concrete and structural steel members [1]. The steel tube provides confinement to the infilled concrete, while in turn the infilled concrete can delay the local buckling of steel tube, resulting in a higher load-bearing capacity and ductility [2]. Due to the excellent composite structural behaviours and corresponding shortened construction time, CFST members have been increasingly used around the world as various structural components. For example, CFST members have been used as: (1) columns in high-rise buildings, such as Ruifeng Building in Hangzhou China [2] and Taipei 101 tower [3]; (2) chords, columns or truss girders in bridges [4]; and (3) piles in floodwall structures [5].

Over the past few decades, extensive experimental investigations on CFSTs have been carried out to advance the knowledge of structural behaviours of CFST members, most of which have focussed on stub column tests to investigate the cross-sectional capacity of CFSTs. For instance, experimental tests on different cross-sections have been carried out by Furlong [6], Tomii et al. [7], Tomii and Sakino [8], Schnelder [9], Varma [10], Uy [11], Han [12], Han and Yao [13], Han et al. [14-16], Sakino et al. [17], Lam and Williams [18], Zhang et al. [19], Tao et al. [20], Liu and Gho [21], Liu [22] and among others. Several databases have been developed by Aho [23], Nishiyama et al. [24], Kim [25], Gourley et al. [26], Xiong et al. [27] and Goode [28], respectively. And these tests formed the basis of the design codes worldwide, such as Eurocode EN 1994-1-1 [29], American standard ANSI/AISC 360-16 [30], and Chinese code GB 50936-2014 [31].

In modern society, the application of high strength materials, namely, high strength steel and high strength concrete, is of vital importance, in particular in high-rise buildings. Use of high strength materials can reduce the dimensions of structural members, resulting in larger usable floor area, fewer material consumptions, lighter structural self-weight, lower constructional expenses as well as lower carbon footprints. Meanwhile, due to the rapid development of the material technology, at present, high strength materials, such as steel with nominal yield strength exceeding 460 MPa and concrete with characteristic cylinder strength up to 120 MPa, have become commercially available [27]. However, to date, current design codes are only applicable to limited range of steel and concrete. For instance, EN 1994-1-1 [29] covers the design of composite columns with the concrete strength classes higher than C20/25 and lower than C50/60 (with corresponding characteristic cylinders strength between 20 MPa and 50 MPa) and with the steel grades not higher than S460; ANSI/AISC 360-16 [30] limits steel yield strength no more than 525 MPa and specified concrete cylinder strength within the range from 21 MPa to 69 MPa. High strength concrete with a compressive cylinder strength greater than 50 MPa and high strength steel with a nominal yield strength exceeding 460 MPa (EN 1994-1-1), or 525 MPa (ANSI/AISC 360-16) are not covered. The use of high strength steel and high strength concrete has been continuously increasing in bridges [4] and high-rise buildings [1, 2]. Therefore, the development of design equations for CFST members incorporating high strength materials is of great urgency and study on whether the current design equations and partial factors can be safely extended to the design of high strength materials needs to be conducted. In the last few years, comprehensive experimental tests on either high strength concrete or high strength steel have been carried out by Wei et al. [32], Zhu and Chan [33], Xiong et al. [27], Zhu et al. [34], Du et al. [35, 36], Duarte et al. [37], Aslani et al. [38], Ding et al. [39, 40], Mursi and Uy [41], Sakino et al. [17], Uy [11], Varma [10], and among others. Lai and Varma [1] summarised some of the tests on high strength rectangular CFST members and conducted a reliability analysis according to North American framework. Wang et al. [42] proposed new capacity prediction models based on CFST database which covers high strength materials and a reliability analysis following Australian approach was conducted to calibrate the resistance factor incorporated in Australian code. The design of high strength CFSTs in accordance with European framework based on an extensive database needs to be further verified.

This study, therefore aims to extend the scope of current EN 1994-1-1 for the design of square and rectangular CFSTs in compression to incorporate high strength materials (460 MPa $\leq f_y \leq$ 690 MPa and 50 MPa $\leq f_c' \leq$ 120 MPa) based on an extensive test database. The database on the square and rectangular CFST stub columns in axial compression in different literature from China, Hong Kong, Japan, United Kingdom, United States etc., consisting of 443 test data, is developed. Broad ranges of parameters, for example steel yield strengths, concrete cylinder strengths and cross-sectional slenderness, are covered in this database. The database is then used to evaluate the current EN 1994-1-1 and recommendations on the relaxation of the cross-section slenderness limit and on the design of high strengths CFSTs in compression are subsequently proposed. A reliability analysis is then carried out to examine the applicability of the current partial factors to the design of high strength materials. Finally, an experimental investigation on 10 stub column tests is carried out to further verify the applicability of the proposed design recommendation.

2. Development of the database

2.1 Concrete strength

Countries in different regions around the world use different concrete compressive strengths, such as concrete cylinder strength and concrete cube strength. Moreover, the standard dimensions of cylinders and cubes differ in different regions, for example, 150 mm by 300 mm cylinders (diameter by height) and 100 mm by 200 mm cylinders, and cubes with side lengths of 150 mm are used. To unify the definition of concrete compressive strength is of great significance when developing an experimental database and comparing CFST tests carried out around the world.

Therefore, in this study a unified compressive strength is used to enable consistent comparisons between tests carried out in different regions. According to Eurocode framework, the concrete compressive strength is referred to the strength of a standard cylinder with a diameter of 150 mm and a height of 300 mm, denoted as f_c'. The reported concrete compressive strengths, the cube strengths ($f_{cu,150}$ or $f_{cu,100}$) or non-standard cylinder strengths ($f_{cyl,100}$), in different experimental investigations were firstly converted to the standard cylinder strengths (f_c') for consistency, where $f_{cu,150}$ and $f_{cu,100}$ denote the compressive strengths from cubes with side lengths of 150 mm and 100 mm, respectively, $f_{cyl,100}$ represents the compressive strengths from cylinders with dimensions (diameter by height) of 100 mm × 200 mm. In this research, the conversion factors and equations adopted have followed the procedures used by Reineck et al. [43-45], Mirza and Lacroix [46], and Sakino et al. [17]. The detailed conversion factors and equations are listed in Table 1.

2.2 Description of square and rectangular CFST stub column database

This section summarises the experimental investigations on square and rectangular CFST stub columns. Results of 443 on stub column tests from 40 different sources [9-22, 27, 33-41, 47-62] were collated to establish a comprehensive database to assess and develop current EN 1994-1-1. In terms of the failure modes of the experimental tests, two typical failure modes have been observed during the data collecting process, namely, local buckling of steel tube followed by concrete crushing [2] and local buckling of steel tube followed by concrete shear failure [33]. The first failure mode is normally observed in specimens infilled with normal strength concrete, while the second failure mode generally occurs in high strength concrete specimens, primarily due to the brittle nature of high strength concrete. To evaluate the composite behaviour of square and rectangular CFST cross-sections, only tests loaded on steel and concrete simultaneously were collated and the steel tube confined concrete tests (concrete-loaded tests) were not included in this study. And this paper mainly focuses on stub columns; hence only tests with non-dimensional relative slenderness $\bar{\lambda} \leq 0.2$ (in accordance with EN 1994-1-1) were collected. If the length/depth ratios of the stub columns were not reported, a value of 3 was assumed as it was a commonly used ratio for stub column tests. Detailed information of the rectangular and square CFST database can be found in Table 2, which summarises the original data sources, the ranges of geometries and material properties of the test specimens, and the number of tests. It can be seen from Table 2 that a wide range of parameters is covered in this database. The outer depth H, outer depth-to-width ratio H/B and outer depth-to-thickness ratio H/t of test specimens vary from 60 to 400 mm, 1.00 to 2.01, and 10.5 to 150.0, respectively. The steel yield strength f_y ranges from 194 to 835 MPa and concrete cylinder strength f_c' ranges from 10.5 to 156.4 MPa and steel contribution ratio δ (please refer to Section 3 for details) varying from 0.132 to 0.923.

3. Design for square and rectangular CFSTs in EN 1994-1-1

The current EN 1994-1-1 sets material strength limitations on both concrete and steel and is only applicable to normal strength concrete and steel. More specifically, in EN 1994-1-1, the design rule covers composite column with the concrete strength classes higher than C20/25 and lower than C50/60 (with corresponding characteristic cylinders strengths between 20 MPa and 50 MPa) and with the steel yield strength not more than 460 MPa. To avoid local buckling of steel tubes, EN 1994-1-1 gives a maximum depth-to-thickness (H/t) value of $52\sqrt{235/f_y}$. EN 1994-1-1 also sets a limitation on the steel contribution ratio δ (as expressed in Eq. (1)) that δ should be within the range $0.2 \leq \delta \leq 0.9$, where δ is defined as the resistance contribution of steel over the cross-section plastic resistance ($N_{pl,Rd}$), as given in Eq. (2). A_s and A_c are areas of steel tube and concrete, f_y and $f_c^{'}$ are steel yield strength and concrete compressive cylinder strength, γ_{M0} and γ_c are partial factors for steel and concrete equal to 1.0 (EN 1993-1-1 [63]) and 1.5 (EN 1992-1-1 [64]), respectively.

$$\delta = \frac{A_s f_y}{N_{pl,Rd}} \tag{1}$$

$$N_{pl,Rd} = A_s f_y / \gamma_{M0} + A_c f_c^{'} / \gamma_c \tag{2}$$

4. Evaluation of the database

4.1 Comparison with EN 1994-1-1

In order to compare the calculated resistance of square and rectangular CFST cross-sections using EN 1994-1-1 with the experimental test results, a model safety factor γ_{mod} was used, which is defined in Eq. (3). N_{Test} is the test result, and N_0 is the calculated value using the design equation. However, all the partial factors γ_c and γ_{M0} for concrete and steel are set unity as expressed in Eq. (4) to make a direct comparison with the design equation in EN 1994-1-1,

$$\gamma_{mod} = N_{Test} / N_0 \tag{3}$$

$$N_0 = A_s f_y + A_c f_c^{'} \tag{4}$$

where A_s and A_c are areas of steel tube and concrete, f_y and $f_c^{'}$ are measured steel yield strength and concrete cylinder strength.

Fig. 1 shows the relationship between the model safety factor γ_{mod} and the steel contribution ratio δ, defined as Eq. (1). 431 data points out of the 443 tests in the database satisfy the requirement for steel contribution ratio in EN 1994-1-1 ($0.2 \leq \delta \leq 0.9$). Based on the steel contribution ratio δ, the database was split into three datasets. The statistical evaluation results of the datasets with respect to the steel contribution ratio are summarised in Table 3, in which $\gamma_{mod,max}$, $\gamma_{mod,min}$, $\gamma_{mod,m}$ are the maximum, minimum and mean values of the model safety factor and s_{mod} is the corresponding standard deviation. The results indicate that EN 1994-1-1 gives good predictions on the cross-section capacities for square and rectangular CFSTs, yielding a mean value of model safety factor γ_{mod} of 1.04 and a corresponding standard deviation s_{mod} of 0.11, when the steel contribution ratio δ meets the codified requirement.

The anticipated trend of reducing values of model safety factors with the increase of cross section slenderness is illustrated in Fig. 2. A fitting curve was determined by a linear regression analysis to best fit the test data. While assessing the experimental test data in Fig. 2, the current value of slenderness limit for square and rectangular CFSTs ($52\sqrt{235/f_y}$) appears too conservative. The slenderness limit calculated from the fitting curve assuming $\gamma_{mod} = 1$ is $68\sqrt{235/f_y}$, which is approximately 1.31 times of the current limit. ANSI/AISC 360-16 limits the slenderness ratio to $2.26\sqrt{E_s/f_y}$ for compact sections, with an equivalent limiting value of $67.6\sqrt{235/f_y}$ (having been converted to the Eurocode format, taking the elastic modulus of steel E_s as 210000 MPa). Therefore, considering the results from regression analysis and recommendation in ANSI/AISC 360-16, the slenderness limit is suggested to be relaxed from $52\sqrt{235/f_y}$ to $68\sqrt{235/f_y}$ for square and rectangular CFST sections. Based on the value of cross section slenderness ratio, the database was split into four datasets. The statistical evaluation results of the test database regarding the cross section slenderness ratio are summarised in Table 4. A total of 80 tests fall into the cross section slenderness range of $52\sqrt{235/f_y} < H/t \leq 68\sqrt{235/f_y}$, with a mean value of model safety factor $\gamma_{mod,m}$ of 1.03 and a corresponding standard deviation s_{mod} of 0.10. Some 358 tests have the cross section slenderness with a range of $H/t \leq 68\sqrt{235/f_y}$. Mean value of model safety factor and corresponding standard deviation of these tests are 1.06 and 0.11, respectively.

In Fig. 3, the model safety factors are plotted with respect to the steel yield strength f_y. Based on the strength limitations on steel in EN 1994-1-1 for normal strength steel and in EN 1993-1-12 [65] for high strength steel, the database was split into five datasets. The statistical evaluation results of the test database with regard to the steel yield strength are summarised in Table 5. 96 tests were included in the dataset for steel yield strength greater than 460 MPa but no more than 690 MPa. The mean value of model safety factor $\gamma_{mod,m}$ is 1.03 and the corresponding standard deviation s_{mod} is 0.09. 368 results have the yield strength ranging between 235 MPa and 690 MPa. A mean value of model safety factor $\gamma_{mod,m}$ of 1.03 from is obtained with a corresponding standard deviation s_{mod} of 0.12, indicating that the current design equation in EN 1994-1-1 could be extended to the design of high strength steel up to 690 MPa.

Fig. 4 presents the relationship between model safety factors and the concrete cylinder strength f_c'. Based on the requirements on concrete cylinder strength in EN 1994-1-1, EN 1992-1-1 and fip Model Code 2010 [66], the database was split into four datasets to investigate the influence of concrete cylinder strength. The statistical evaluation results of the test database regarding the concrete cylinder strength are summarised in Table 6. The dataset for concrete cylinder strength greater than 50 MPa but no more than 120 MPa consists of 236 test results. The mean value of model safety factor $\gamma_{\mathrm{mod,m}}$ is 1.03 and the corresponding standard deviation s_{mod} is 0.12, indicating that the current design equation in EN 1994-1-1 could be extended to the design of high strength concrete with cylinder strengths not greater than 120 MPa.

4.2 Extension of EN 1994-1-1

The aim of developing the database was to collate comprehensive test results that could be used to extend the scope of design equation in current EN 1994-1-1 to larger ranges of parameters, namely ranges of material strengths and geometries. Based on the statistical evaluation in the last section in accordance with Eurocode framework, the current slenderness limit for square and rectangular CFST sections is proposed to be relaxed from $52\sqrt{235/f_{\mathrm{y}}}$ to $68\sqrt{235/f_{\mathrm{y}}}$, the limitations on steel yield strength and concrete cylinder strength can be extended from 460 MPa and 50 MPa to 690 MPa and 120 MPa, respectively.

A total of 357 tests, denoted as dataset EXT standing for extension, out of the total 443 tests satisfy the extended scopes of parameters, namely $0.2 \leq \delta \leq 0.9$, $H/t \leq 68\sqrt{235/f_{\mathrm{y}}}$, 235 MPa $\leq f_{\mathrm{y}} \leq$ 690 MPa and 20 MPa $\leq f_{\mathrm{c}}' \leq$ 120 MPa. To distinguish the influence of the material strengths, the dataset of the 357 tests (EXT) was split into two sub-sets with respect to the material strengths, namely, NS (normal strength) and HS (high strength). Dataset NS consists of 90 tests, which are within the current material limitations of EN 1994-1-1, whilst HS includes 267 test data with either steel or concrete strength exceeding the specified material limits. Table 7 summarises the statistical evaluation results of the datasets, in which $\gamma_{\mathrm{mod,5\%}}$ and $\gamma_{\mathrm{mod,95\%}}$ are the 5% fractile value of the model safety factor of the datasets as defined in Eq. (5) and the 95% fractile value of the model safety factor of the datasets as expressed in Eq. (6). These fractile values are commonly used to account for the effect of extremely low or extremely high values on the distribution of the model safety factors. In addition, the characteristic material strength in Eurocode is defined related 5% fractile value (or 95% fractile value) [67].

$$\gamma_{\mathrm{mod,5\%}} = \gamma_{\mathrm{mod,m}} - 1.64 s_{\mathrm{mod}} \tag{5}$$

$$\gamma_{\mathrm{mod,95\%}} = \gamma_{\mathrm{mod,m}} + 1.64 s_{\mathrm{mod}} \tag{6}$$

Distributions of the model safety factor γ_{mod} of the datasets EXT, NS and HS are presented in Fig. 5. Referring to Fig. 5 and Table 7, it can be concluded that the design equation in EN 1994-1-1 can precisely predict the cross-section capacities of square and rectangular CFSTs giving mean values of model safety factors of 1.06, 1.03 and 1.07 for data sets EXT, NS and HS with corresponding standard deviations of 0.11, 0.12 and 0.11. Moreover, the distribution of the model safety factor can meet the requirements for the 5% and 95% fractile values as the percentages of the tests in corresponding data sets are smaller or marginal to 5%, as reflected by Fig. 5.

5. Reliability analysis

5.1 General

According to EN 1990 [67], the reliability class (RC) for typical building structures falls into RC2 and hence the target value of reliability index β for a reference period of 50 years corresponding to RC2 for structural members is 3.8 for ultimate limit state design. EN 1990 adopts a semi-probabilistic approach to decouple the effects of the actions and resistances by multiplying weight factors α_E and α_R, respectively. And the weight factor on resistance side α_R may be taken as 0.8 given that $0.16 < \alpha_E/\alpha_R < 7.6$. Therefore, the design fractile on resistance side is taken as $k_d = \alpha_R\beta = 0.8 \times 3.8 = 3.04$ for ultimate limit states design, leading to a probability of failure of around 0.1%.

A standard procedure to obtain the design values for a resistance model through a statistical evaluation of experimental data is provided in Annex D of EN 1990. First of all, the theoretical resistance values r_t obtained from the resistance model ($r_t = g_{rt}(X)$) using the measured material and geometric properties (the same as the calculation of N_0 in Eq. (4)), with the experimental test results $r_{e,}$. A linear regression is performed through a plot of r_e against r_t values. An example based on the dataset EXT is presented in Fig. 6.

Then, an error term $\delta_i = r_{e,i}/br_{t,i}$, is calculated for each data pair ($r_{t,i}$, $r_{e,i}$), where b is the correction factor determined as the slope of the least squares best fit of the r_e versus r_t plot. The coefficient of variation of this error term V_δ is determined as an index of the variabilities associated with the predictions from the resistance model, which can be obtained from Eqs. (7-9), where Δ_i is the logarithm of the error term and $s_\Delta{}^2$ is the corresponding variance.

$$V_\delta^2 = \sqrt{\exp\left(s_\Delta^2\right) - 1} \tag{7}$$

$$s_\Delta^2 = \frac{1}{\text{n-1}}\sum_{i=1}^{n}\left(\Delta_i - \frac{1}{n}\sum_{i=1}^{n}\Delta_i\right)^2 \tag{8}$$

$$\Delta_i = LN\left(\delta_i\right) \tag{9}$$

The basic variables in the resistance model $g_{rt}(X)$, including material and geometric properties (A_s, A_c, f_y and f_c'), are scattered and the variances of these variables are accounted for through their coefficient of variation (V_{As}, V_{Ac}, V_{fy}, $V_{fc'}$), where A_s and A_c are cross-section areas of steel tube and infilled concrete, f_y and f_c' are the yield strength of steel and cylinder strength of concrete, V_{As}, V_{Ac}, V_{fy} and $V_{fc'}$ are the corresponding coefficients of variation of cross-section areas of steel tube and infilled concrete, steel yield strength and concrete cylinder strength, respectively. The coefficient of variation of the resistance model in this paper, V_{rt}, can be obtained from Eq. (10), where $g_{rt}(X)$ is the resistance model evaluated from the mean values of the basic variables, $\partial g_{rt}/\partial x_i \cdot \sigma_i$ is the partial derivative for the variable X_i multiplied by its standard deviation σ_i, The coefficients of variation of the basic variables (V_{As}, V_{Ac}, V_{fy}, $V_{fc'}$) are generally determined on the basis of prior knowledge which will be detailed in next section.

$$\begin{aligned} V_{rt}^2 &= \frac{VAR\left[g_{rt}\left(\underline{X}\right)\right]}{g_{rt}^2\left(\underline{X}_m\right)} \cong \frac{1}{g_{rt}^2\left(\underline{X}_m\right)} \times \sum_{i=1}^{j}\left(\frac{\partial g_{rt}}{\partial X_i}\sigma_i\right)^2 \\ &= \frac{1}{g_{rt}^2\left(\underline{X}_m\right)} \times \left[f_y^2 A_s^2\left(V_{A_s}^2 + V_{f_y}^2\right) + f_c'^2 A_c^2\left(V_{A_c}^2 + V_{f_c'}^2\right)\right] \end{aligned} \tag{10}$$

Finally, the design resistance value r_d is obtained from Eq. (11), where b is the mean value correction factor, $g_{rt}(X_m)$ is the design resistance evaluated from the mean values of the basic variables, $k_{d,n}$ is the design fractile factor for the population of tests n for ultimate limit states design and $k_{d,\infty}$ is the design fractile factor for the population of the tests n tending to infinity ($k_{d,\infty} = 3.04$), parameters α_{rt} and α_δ are weighting factors for Q_{rt} and Q_δ, while the parameters Q_{rt}, Q_δ and Q are defined by Eqs. (14-16).

$$r_d = b g_{rt}\left(\underline{X}_m\right)\exp\left(-k_{d,\infty}\alpha_{rt}Q_{rt} - k_{d,n}\alpha_\delta Q_\delta - 0.5Q^2\right) \tag{11}$$

$$\alpha_{rt} = Q_{rt} / Q \tag{12}$$

$$\alpha_\delta = Q_\delta / Q \tag{13}$$

$$Q_{rt} = \sqrt{\ln\left(V_{rt}^2 + 1\right)} \tag{14}$$

$$Q_\delta = \sqrt{\ln\left(V_\delta^2 + 1\right)} \tag{15}$$

$$Q = \sqrt{\ln\left(V_r^2 + 1\right)}, \text{ with } V_r^2 = V_\delta^2 + V_{rt}^2 \tag{16}$$

If a large number of tests (say n ≥ 100) is available, the design value r_d may be simply obtained from Eq. (17).

$$r_d = b g_{rt}\left(\underline{X}_m\right)\exp\left(-k_{d,\infty}Q - 0.5Q^2\right) \tag{17}$$

5.2 Calibration of partial factors

In order to verify the reliability of the resistance model for predicting the cross-section capacities of square and rectangular CFSTs, a reliability analysis in accordance with EN 1990 was performed. To distinguish the influence of the material strengths, the data set of the 357 tests (EXT) was split into two sub-sets with respect to the material strengths, namely, NS (normal strength) and HS (high strength). Details about the datasets can be found in Section 4.2.

In the reliability analysis, the mean to nominal yield strength ratio $f_{y,mean}/f_{y,nom}$, also termed as the material over strength, is of vital importance. It is observed that different values have been adopted in analyses, for example 1.16 from Byfield and Nethercot [68] and 1.26 from Tankova et al. [69] for normal strength steel and 1.135 from Wang et al. [70] for high strength steel. Shi et al. [71] collected over 10000 tensile coupon test results of 500 MPa, 550 MPa and 690 MPa steel products and found the mean to nominal yield strength ratio varied from 1.107 to 1.312 for these steel grades. In this paper, the mean to nominal yield strength ratios for normal strength steel and high strength steel were taken as 1.16 and 1.135, and the corresponding coefficients of variation of normal strength steel and high strength steel were 0.05 [68] and 0.055 [70]. And the coefficient of variation of geometric properties was taken as 0.02 [68]. The characteristic cylinder strength of concrete was calculated compling with EN 1992-1-1 and fip Model Code 2010, in which a scatter Δf is deducted from measured mean strength to take account of material variability the as expressed in Eq. (18).

$$f_c^{'} = f_{ck} + \Delta f \tag{18}$$

In EN 1992-1-1 and fip Model Code 2010, the scatter Δf is taken as 8 MPa to consider the variability and unfavourable conditions on construction site. This value appears too high for the tests carried out in the laboratories which are under good quality control. Therefore, a smaller scatter of 4 MPa was chosen for later comparisons and evaluations Reineck et al. [43-45]. According to EN 1990, the characteristic value is defined as the 5% fractile value. Therefore, the coefficient of variance for concrete cylinder strength can be determined by Eq. (19).

$$V_{f_c^{'}} = 4 / 1.64 f_c^{'} \tag{19}$$

To evaluate the applicability of extending the current partial factors (1.0 for steel and 1.5 for concrete) to the design of high strength materials, a corrected partial factor $\gamma_M{*}$ is defined as the slope of the least squares best fit of the r_n versus r_d plot, as expressed in Eq. (20), where the r_d and r_n can be obtained from Eqs. (17) and (21), respectively. The r_e versus r_t diagrams to determine the mean value correction factor b and $r_{n,i}$ versus $r_{d,i}$ diagrams to obtain the corrected partial factor $\gamma_M{*}$ for the two sub datasets are presented in Figs. 7 and 8, respectively. It should be noted that the current partial material factors (1.0 for steel and 1.5 for concrete) were incorporated in the calculation of r_n.

$$\gamma_M{*} = \frac{\sum_{i=1}^{n} r_n r_d}{\sum_{i=1}^{n} r_d^2} \tag{20}$$

$$r_n = N_{pl,Rd} = A_s f_y / \gamma_{M0} + A_c f_{ck} / \gamma_c \tag{21}$$

The results of the reliability analyses and a summary of the key statistical parameters are listed in Table 8, in which $k_{d,n}$ is the design fractile factor for n tests (taking as 3.04 because of a large number of n), where n is the population of test data under consideration, b is the mean value correction, V_δ coefficient of variation of the tests associated with the resistance model, V_r is the combined coefficient of variation incorporating both model and basic variable uncertainties, and γ_M* is the corrected partial factor. Under the required reliability level, a value of γ_M* equal or less than unity means the current partial factors for concrete and steel could be safely extended to larger ranges of parameters, while a value higher than 1.0 means greater partial factors should be incorporated.

The results in Table 8 indicates that the current design equation can be safely use for the design of normal strength CFSTs with an extended cross-sectional slenderness limit as the corrected partial factor is very marginal to 1.0. However, the corrected partial factor is greater than 1.0 for the data set HS (high strength), indicating the current partial factors (1.0 for steel and 1.5 for concrete) could be used for normal strength materials within the proposed cross-section slenderness range but are not safe for the design of high strength materials. It was found that higher partial factors, nearly 1.05 times of current partial factors, should be used for square and rectangular CFST sections incorporating high strength materials. To be consistent with the current partial factors γ_{M0} and γ_c (1.0 and 1.5, respectively), the following expression (Eq. (22)) is proposed for the design of square and rectangular CFST sections incorporating high strength materials (460 MPa $\leq f_y \leq$ 690 MPa and 50 MPa $\leq f_c^{'} \leq$ 120 MPa) to yield an acceptable safety level.

$$N_{\mathrm{pl,Rd}} = \frac{1}{1.05}\left(A_s f_y / \gamma_{M0} + A_c f_c^{'} / \gamma_c\right) \tag{22}$$

6. Experimental investigation

To further verify the applicability of the proposed design recommendations on square and rectangular CFST in compression, an experimental investigation on 10 stub column tests were carried out. The square and rectangular hollow tubes were manufactured though a combination of press-braking and welding. It is worth noting that strengths of steel and concrete used in this study exceed the material limits specified in current EN 1994-1-1 (f_y > 460 MPa and $f_c^{'}$>50 MPa).

6.1 Specimens

A total of 10 CFST stub columns with square and rectangular cross-sections were prepared. The measured cross-sectional dimensions are reported in Table 9. Definition of symbols is depicted in Fig. 9 where H and B are overall depth and overall width of the square or rectangular sections and t is thickness of steel tubes. L is length of specimens and is equal to 3(H+B)/2. It can be seen from Table 9 that a wide range of cross-sectional slenderness $H/(t\sqrt{235/f_y})$ was covered. Two batches of steel plates with nominal thickness of 3 mm and 6 mm were employed in this study. Two grades of concrete with target cylinder strengths of 50 MPa (C50) and 90 MPa (C90) were used to fill the hollow tubes. A specimen label system is adopted throughout this study, which firstly specifies the cross-sectional shape of the specimen: “S” for square and “R” for rectangular. Following the shape, cross-sectional dimensions are given by $H \times B \times t$. Finally, the target concrete cylinder strength is given after a hyphen. For example, the label “S150×150×6-90” defines a square CFST specimen with the nominal dimension of 150×150×6 infilled with 90 MPa (cylinder strength) concrete. Symbol “#” in the suffix indicates a repeated test.

6.2 Material tests

Three coupons were extracted from each batch of steel. The coupon tests were conducted in accordance with EN ISO 6892-1:2019 [72]. The mean measured material properties are summarised in Table 10, in which E_s is elastic modulus of steel, v is Poisson’s ratio, f_y is yield strength, f_u is ultimate strength, ε_{sh} is strain-hardening strain (the strain at which strain-hardening initiates), ε_u is strain at the ultimate strength, and ε_f is proportional elongation at fracture based on an original gauge length of $5.65\sqrt{A_0}$, (A_0 is cross-sectional area within the original gauge length of the coupon). Typical stress-strain curves obtained from tensile coupon tests are depicted in Fig. 10(a).

Three standard concrete cylinders with a diameter of 150 mm and a height of 300mm were prepared during the concrete infilling of specimens for each batch of concrete. The cylinders were tightly wrapped with cling film and sealed with tape to keep the moisture content and to simulate the curing conditions of the concrete that were infilled in the tubes. The cylinders were tested at the test day to obtain the exact concrete strengths at the test day. The concrete cylinders were tested under displacement-control at a rate of 0.15 mm/min and the testing procedure complied with EN 12390-3 [73]. The concrete compression test results, the compressive cylinder strengths (f_c'), are summarised in Table 9. Typical stress-strain curves of concrete cylinder tests are depicted in Fig. 10(b).

6.3 Stub column tests

All stub column tests were conducted at The Hong Kong Polytechnic University. Strain gauges were attached at the mid-height of specimens to capture axial strain of the column under compression. Strains at corner and flat portions were recorded. A total of 4 linear variable displacement transducers (LVDTs) were used around the specimens to record the end-shortening. Fig. 11 shows the schematic view and experimental view of test arrangements of stub column tests. The axial load was applied through a displacement controlled mechanism at a rate of 0.05%L mm/min (equivalent to the rate of concrete cylinder tests). The obtained maximum axial loads of each specimen are reported in Table 11. Axial load versus end shortening curves of the specimens are presented in Fig. 12.

6.4 Comparison with proposed design recommendations

Test results were firstly compared with calculations from the current Eurocode equation N_1 (Eq. (2)). For direct comparisons, the partial factors γ_{M0} and γ_c are taken as unity. Referring to Table 11, a mean N_{test}/N_1 value of 0.97 was obtained with a corresponding CoV (coefficient of variance) of 0.07 if the 10 tests were considered. 7 out of the 10 specimens satisfy the cross-sectional slenderness limit $H/t \leq 68\sqrt{235/f_y}$. If the 7 tests were considered, the mean value of N_{test}/N_1 was 0.99 with a CoV of 0.05. As the strengths of steel and concrete used in this study both exceed the material limits specified in current EN 1994-1-1 (f_y > 460 MPa and f_c'>50 MPa), the tests are classified as HS (high strength category) according to Section 4.2. The proposed equation given in Eq. (22) was also used to calculate the cross-sectional capacity (N_2). If all tests were considered, the mean N_{test}/N_2 value is 1.03 and the corresponding CoV is 0.03. If the tests satisfying the cross-sectional slenderness limit $H/t \leq 68\sqrt{235/f_y}$ were considered, the mean N_{test}/N_2 value of 1.05 with a CoV of 0.05 was obtained, indicating that the proposed design recommendations yielded satisfactory predictions. Therefore, the proposed design equation - Eq. (22) is applicable to the design of square and rectangular CFSTs using high strength materials.

7. Conclusion

This paper presented the results of a systematic approach used to address the advancement of EN 1994-1-1 for the design of square and rectangular CFSTs incorporating high strength materials. A comprehensive experimental database consisting of 443 square and rectangular CFST stub column tests was compiled, followed by a thorough statistical evaluation on the possibility of extending the current design equation in EN 1994-1-1 to the design of high strength square and rectangular CFST stub columns. Results of the statistical evaluations indicate that: firstly, the cross-section slenderness limit can be safely relaxed from $52\sqrt{235/f_y}$ to $68\sqrt{235/f_y}$; secondly, the design equation could be extended to the design of square and rectangular CFST stub columns incorporating high strength materials (460 MPa $\leq f_y \leq$ 690 MPa and 50 MPa $\leq f_c' \leq$120 MPa). Subsequently, a reliability analysis was performed to calibrate the partial factors for the design of high strength square and rectangular CFST stub columns. The current partial factors (1.0 for structural steel and 1.5 for concrete) recommended by EN 1994-1-1 were found to be adequate for the normal strength materials, even after the relaxation of the cross-section slenderness limit. However, the factors are unsafe for high strength materials. Therefore, a new design equation - Eq. (22) was recommended to the design of high strength square and rectangular CFSTs. To verify the applicability of the proposed design recommendations, a total of 10 square and rectangular CFST sub columns were tested. It was found that the proposed design recommendations yielded satisfactory predictions when compared with test results.

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Table 1. Conversion factors and equations for concrete compressive strengths

Strengths	Conversion factors and equations	Ref.
$f_{cu,150}$	$f_c' = f_{cu,150}\left[0.76 + 0.2\log_{10}\left(\frac{f_{cu,150}}{19.6}\right)\right]$	[46]
$f_{cu,100}$	$f_{cu,150} = 0.9 f_{cu,100}$	[43]
$f_{cyl,100}$	$f_c' = 1.67 \times 150^{-0.112} f_{cyl,100} = 0.953 f_{cyl,100}$	[17]

Table 2. Experimental tests on square and rectangular CFST stub columns.

References	H (mm)	H/B	H/t	f_y (MPa)	f_c' (MPa)	δ	No. of tests
Zhu and Chan, 2018 [33]	200	1	33.3	485	38.0~112.1	0.363~0.627	7
Xiong et al., 2017 [27]	150	1.00	12.0~18.8	446.0~779.0	140.1~156.4	0.557~0.692	15
Zhu et al., 2017 [34]	197~201	1.00	19.5~32.5	381.7~437.9	19.1~20.9	0.741~0.820	6
Du et al., 2016 [35]	150~203	1.00~2.01	18.1~24.5	488.4	35.8~47.0	0.731~0.805	6
Du et al., 2016 [36]	120~180	1.17~1.50	15.6~31.9	423.2~514.5	35.8~47.0	0.660~0.804	8
Duarte et al., 2016 [37]	100~220	1.00~2.00	32.8~72.7	284.0~455.5	41.6	0.323~0.630	8
Aslani et al., 2015 [38]	70~190	1.00	14.0~38.0	701.0	21.0~54.5	0.595~0.923	12
Ding et al., 2015 [39]	300	1.00	77.5~81.1	311.0	35.5~54.4	0.228~0.310	4
Yamamoto et al., 2015 [47]	100	1.00	47.4	353.0	77.1~90.6	0.260~0.292	2
Ding et al., 2014 [40]	250~251	1.00	66.9~67.5	327.7	33.2	0.381	3
Guler et al., 2014 [48]	100.2~101.2	1.00	20.0~33.1	300.0~310.0	115.0	0.257~0.388	6
Yamamoto et al., 2013 [49]	400	1.00	17.9~44.0	346.0~412.0	32.3~56.1	0.376~0.774	6
Tao et al., 2009 [50]	250	1.00	100.0	338.0	20~43.8	0.244~0.411	4
Yu et al., 2008 [51]	100	1.00	52.6	404.0	111.7	0.226	4
Tao et al., 2008 [52]	190~250	1.00	76.0~100.0	270.0~342.0	49.8~58.1	0.183~0.244	6
Han et al., 2008 [16]	177	1.00	70.8	362.9	65.1	0.248	2
Yao and Gao, 2006 [53]	100~300	1.00~1.50	23.5~60.7	245.8~254.2	18.3	0.539~0.716	4
Liu, 2005 [22]	120~190	1.00~2.00	26.5~47.5	495.0	60.0~89.0	0.409~0.551	22
Han et al., 2005 [14]	60~250	1.00	32.1~133.7	282.0~404.0	42.9~71.5	0.132~0.582	24
Liu and Gho, 2005 [21]	120~220	1.00~2.00	20.7~47.5	300.0~495.0	55.0~106.0	0.315~0.572	26
Tao et al., 2005 [20]	129~250	1.01	51.6~100.2	343.7	50.1~54.8	0.162~0.261	2
Zhang et al., 2005 [19]	101~195	1.00~1.61	20.5~56.6	255.1~347.3	50.5~62.5	0.272~0.612	50
Han and Yao, 2004 [13]	200	1.00	66.7	260.5	50	0.276	6
Lam and Williams, 2004 [18]	100~101	1.00	10.5~24.4	289.0~400	24.6~89.1	0.411~0.895	10
Mursi and Uy, 2004 [41]	100~260	1.00	22.0~52.0	761.0	20.3	0.754~0.887	4
Sakino et al., 2004 [17]	110~319	1.00	18.5~73.7	262~835	24.2~86.8	0.162~0.865	48
Zhang et al., 2004 [54]	141~142	1.00	69.8~70.7	305.1	51.1	0.262	3

Han and Yao, 2003 [55]	135~240	1.50~2.00	50.9~90.6	340.1	17.9	0.570~0.668	7
Liu et al., 2003 [56]	100~200	1.00~2.02	24.0~47.9	550.0	61.7~72.6	0.429~0.632	21
Han, 2002 [12]	90~160	1.00~1.75	21.1~52.4	194.0~228.0	50.8	0.278~0.503	24
Huang et al., 2002 [57]	200~300	1.00	40.0~150.0	265.8~341.7	27.2~31.2	0.255~0.343	3
Han et al., 2001 [14]	120~200	1.00	20.5~36.5	321.0~330.0	10.5~41.3	0.488~0.851	20
Uy 2001 [11]	110	1.00	22.0	784.2	49.2	0.770	2
Varma, 2000 [10]	305	1.00	34.3~52.6	259.0~660.0	110.0	0.225~0.394	4
Yamamoto et al., 2000 [58]	100~300	1.00	45.9~49.3	300.0~395.0	24.5~60.7	0.325~0.575	8
Lv et al., 1999 [59]	200~300	1.00	40.0~60.0	227.0	29.3~38.4	0.293~0.456	4
Schnelder, 1998 [9]	127~153	1.00~2.00	17.0~50.8	312.0~413.0	23.8~30.5	0.555~0.815	11
Miyaki et al., 1996 [60]	151	1.00	17.7~34.0	353.0~542.0	76.2~84.0	0.374~0.636	2
Kato, 1995 [61]	200~250	1.00	21.0~55.6	316.8~767.9	27.3~78.6	0.319~0.860	26
Shakir-Khali and Mouli, 1990 [62]	120~150	1.25~1.50	24.0~30.0	340.0~362.5	35.7~40.5	0.592~0.713	13
Total	***60~400***	***1.00~2.02***	***10.5~150.0***	***194.0~835.0***	***10.5~156.4***	***0.132~0.923***	***443***

Table 3. Statistical evaluation of the database with respect to δ.

Criterion	No. of tests	Model safety factor			
		$\gamma_{mod,max}$	$\gamma_{mod,min}$	$\gamma_{mod,m}$	s_{mod}
$\delta < 0.2$	11	0.96	0.83	0.92	0.04
$\boldsymbol{0.2 \leq \delta \leq 0.9}$	**431**	**1.37**	**0.74**	**1.04**	**0.11**
$\delta > 0.9$	1	-	-	1.39	-

Table 4. Statistical evaluation of the database regarding the $H/(t\sqrt{235/f_y})$.

Criterion	No. of tests	Model safety factor			
		$\gamma_{mod,max}$	$\gamma_{mod,min}$	$\gamma_{mod,m}$	s_{mod}
$H/(t\sqrt{235/f_y}) \leq 52$	278	1.39	0.74	1.06	0.11
$52 < H/(t\sqrt{235/f_y}) \leq 68$	80	1.31	0.84	1.03	0.10
$\boldsymbol{H/(t\sqrt{235/f_y}) \leq 68}$	**358**	**1.39**	**0.74**	**1.06**	**0.11**
$H/(t\sqrt{235/f_y}) > 68$	85	1.12	0.77	0.93	0.08

Table 5. Statistical evaluation of the database regarding the steel yield strength.

Criterion	No. of tests	Model safety factor			
		$\gamma_{mod,max}$	$\gamma_{mod,min}$	$\gamma_{mod,m}$	s_{mod}
$f_y < 235$	30	1.30	0.90	1.09	0.08
$235 \leq f_y \leq 460$	272	1.37	0.74	1.03	0.12
$460 < f_y \leq 690$	96	1.36	0.79	1.02	0.09
$\boldsymbol{235 \leq f_y \leq 690}$	**368**	**1.37**	**0.74**	**1.03**	**0.12**
$f_y > 690$	45	1.39	0.77	1.03	0.12

Table 6. Statistical evaluation of the database regarding the concrete cylinder strength.

Criterion	No. of tests	Model safety factor			
		$\gamma_{mod,max}$	$\gamma_{mod,min}$	$\gamma_{mod,m}$	s_{mod}
$f_c' < 20$	23	1.22	0.86	1.05	0.09
$20 \leq f_c' \leq 50$	169	1.39	0.77	1.02	0.12
$50 < f_c' \leq 120$	236	1.31	0.74	1.03	0.12
$\boldsymbol{20 \leq f_c' \leq 120}$	**405**	**1.39**	**0.74**	**1.03**	**0.12**
$f_c' > 120$	15	1.23	1.06	1.14	0.04

Table 7. Statistical evaluation of the data set EXT.

Data set	No. of tests	Model safety factor						Percentage	
		$\gamma_{mod,max}$	$\gamma_{mod,min}$	$\gamma_{mod,m}$	s_{mod}	$\gamma_{mod,5\%}$	$\gamma_{mod,95\%}$	$\gamma_{mod} < \gamma_{mod,5\%}$	$\gamma_{mod} > \gamma_{mod,95\%}$
EXT	357	1.37	0.74	1.06	0.11	0.87	1.24	3.4%	5.3%
NS	90	1.37	0.80	1.03	0.12	0.84	1.22	4.4%	5.6%
HS	267	1.36	0.74	1.07	0.11	0.88	1.25	3.0%	4.9%

Table 8. Summary of statistical analysis parameters for EN 1990.

Data set	No. of tests	$k_{d,n}$	b	V_δ	V_r	γ_M*
NS	90	3.05	1.05	0.112	0.121	1.01
HS	267	3.04	1.02	0.107	0.114	1.05

Table 9. Detailed information of the test specimens.

Specimen	Width H, mm	Depth B, mm	Thickness t, mm	Length L, mm	$H/(t\sqrt{235/f_y})$	f_y MPa	f_c' MPa
S150×150×6-50	163.74	158.61	5.80	450	44.4	580.7	51.3
S150×150×6-90	162.84	158.29	5.82	447	44.0	580.7	91.3
S150×150×6-90#	161.32	160.20	5.84	447	43.4	580.7	91.3
S200×200×6-90	212.12	211.60	5.82	595	57.3	580.7	88.4
R150×100×3-50	157.64	106.33	3.06	372	78.7	546.5	51.3
R150×100×6-50	164.17	108.24	5.84	375	44.2	580.7	51.3
R150×100×6-90	163.45	109.20	5.82	373	44.2	580.7	91.3
R200×100×3-50	206.03	107.73	3.06	451	102.7	546.5	51.3
R200×100×3-90	207.99	106.36	3.04	448	104.4	546.5	91.3
R200×100×6-90	214.50	108.04	5.81	448	58.0	580.7	91.3

Note: # indicates a repeated test.

Table 10. Test results of tensile coupons.

Steel	E_s GPa	ν -	f_y MPa	f_u MPa	ε_{sh} %	ε_u %	ε_f %
3 mm	209.5	0.28	546.5	625.8	2.2	10.9	26.0
6 mm	213.3	0.28	580.7	666.1	2.3	10.1	25.4

Table 11. Comparison with Eurocode EN 1994-1-1.

Specimen	$H/(t\sqrt{235/f_y})$ -	f_y MPa	f_c MPa	N_{test} kN	N_1 kN	N_{test}/N_1 -	N_2 kN	N_{test}/N_2 -
S150×150×6-50	44.4	580.7	51.3	3346	3204	1.04	3052	1.10
S150×150×6-90	44.0	580.7	91.3	4008	4079	0.98	3885	1.03
S150×150×6-90#	43.4	580.7	91.3	4122	4092	1.01	3897	1.06
S200×200×6-90	57.3	580.7	88.4	6231	6291	0.99	5991	1.04
R150×100×3-50	78.7	546.5	51.3	1664	1630	1.02	1552	1.07
R150×100×6-50	44.2	580.7	51.3	2659	2486	1.07	2368	1.12
R150×100×6-90	44.2	580.7	91.3	3065	3079	1.00	2932	1.05
R200×100×3-50	102.7	546.5	51.3	1763	2061	0.86	1963	0.90
R200×100×3-90	104.4	546.5	91.3	2555	2857	0.89	2721	0.94
R200×100×6-90	58.0	580.7	91.3	3520	3847	0.91	3664	0.96
Mean						0.97 (0.99)		1.03 (1.05)
CoV						0.07 (0.05)		0.03 (0.05)

Note: # indicates a repeated test; values in the brackets are the results without considering specimens R150×100×3-50, R200×100×3-50 and R200×100×3-90 ($H/t > 68\sqrt{235 / f_y}$)

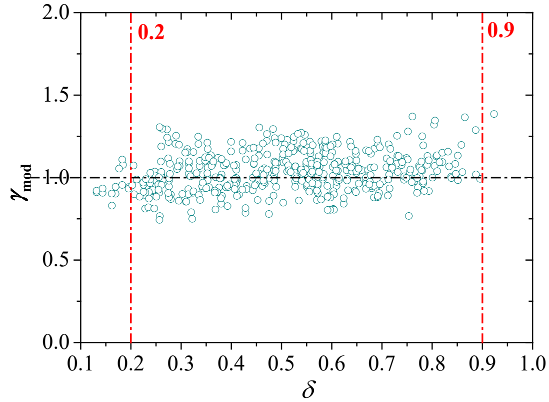


Fig. 1. Model safety factor γ_{mod} versus steel contribution ratio δ diagram.

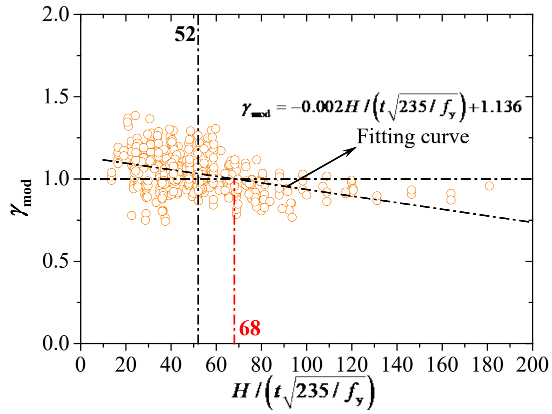


Fig. 2. Model safety factor γ_{mod} versus cross-section slenderness $H/(t\sqrt{(235/f_y)})$ diagram.

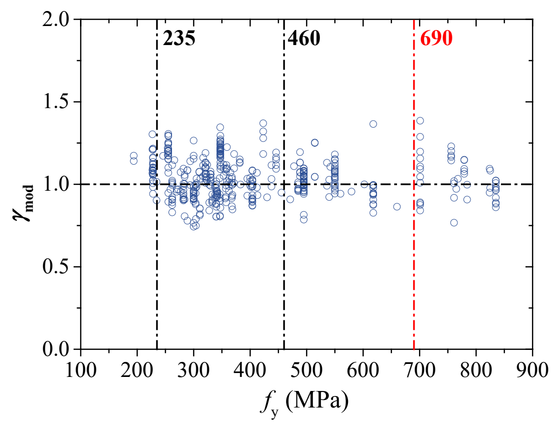


Fig. 3. Model safety factor γ_{mod} versus steel yield strength f_y diagram.

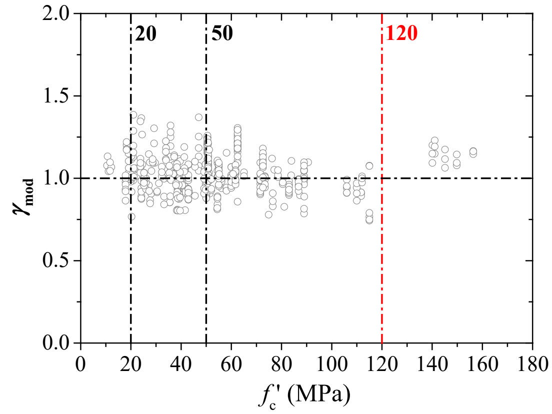
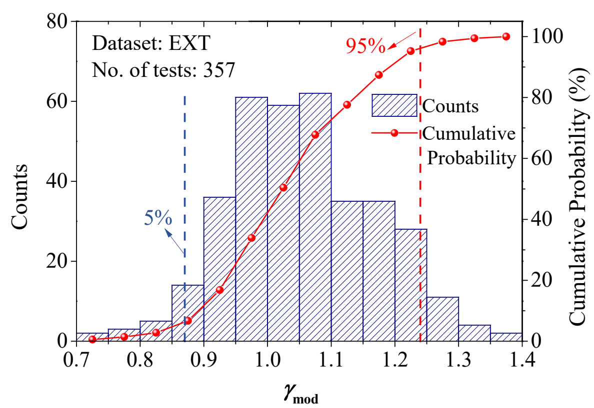
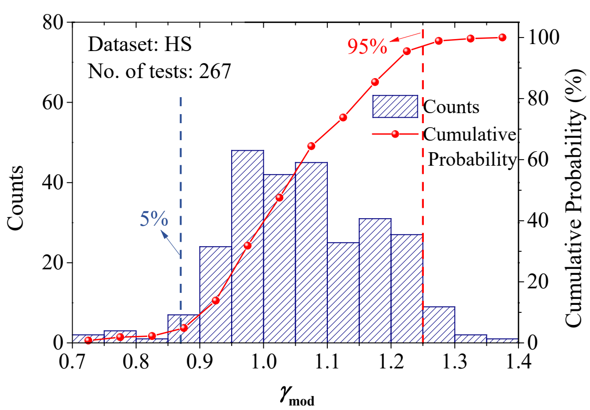


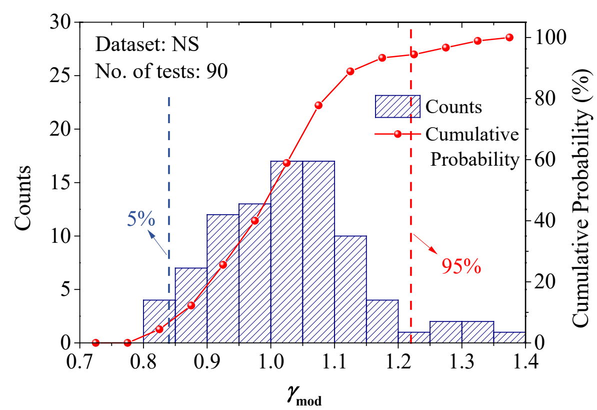
Fig. 4. Model safety factor γ_{mod} versus concrete cylinder strength f_c' diagram.



(a) Dataset EXT



(b) Dataset NS



(c) Dataset HS

Fig. 5. Distributions of model safety factor γ_{mod}.

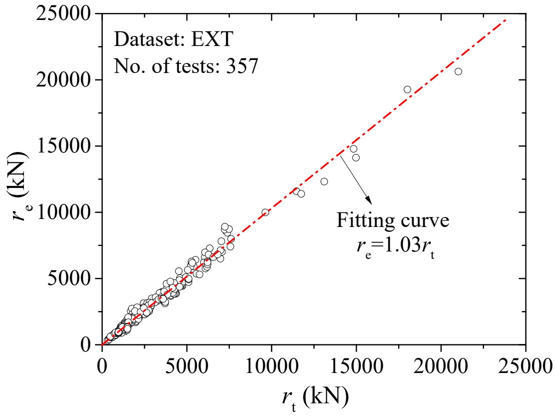
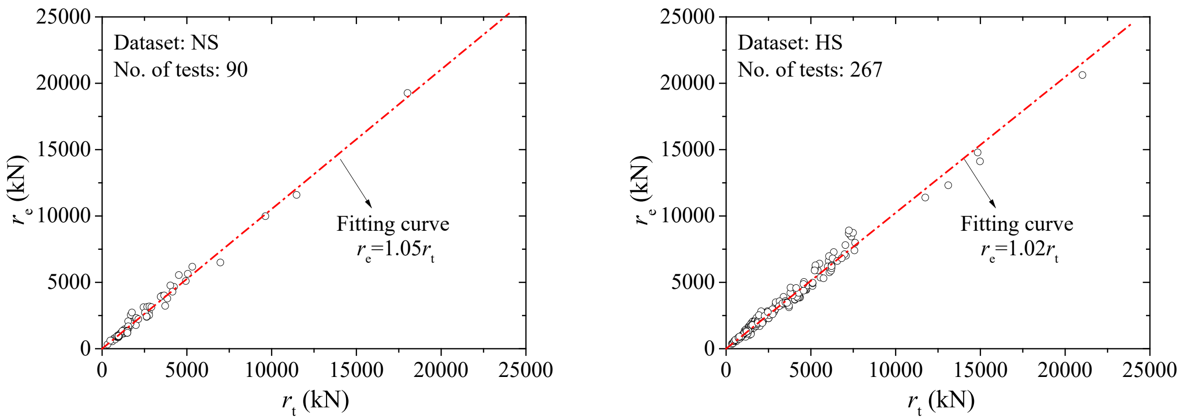
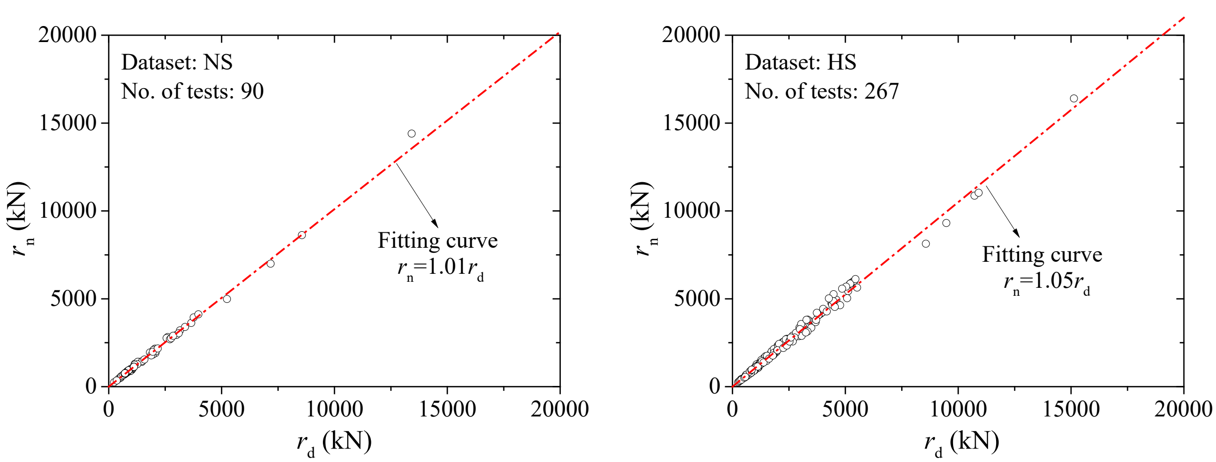


Fig. 6. r_e versus r_t diagram for dataset EXT.



(a) Dataset NS (b) Dataset HS

Fig. 7. r_e versus r_t diagrams for two sub datasets.



(a) Dataset NS (b) Dataset HS

Fig. 8. r_n versus r_d diagrams for two sub datasets.

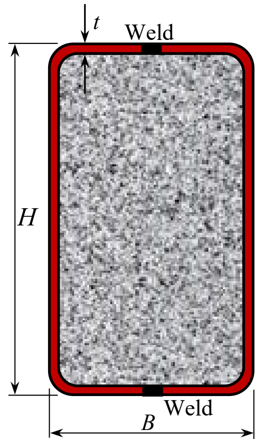


Fig. 9. Definition of symbols.

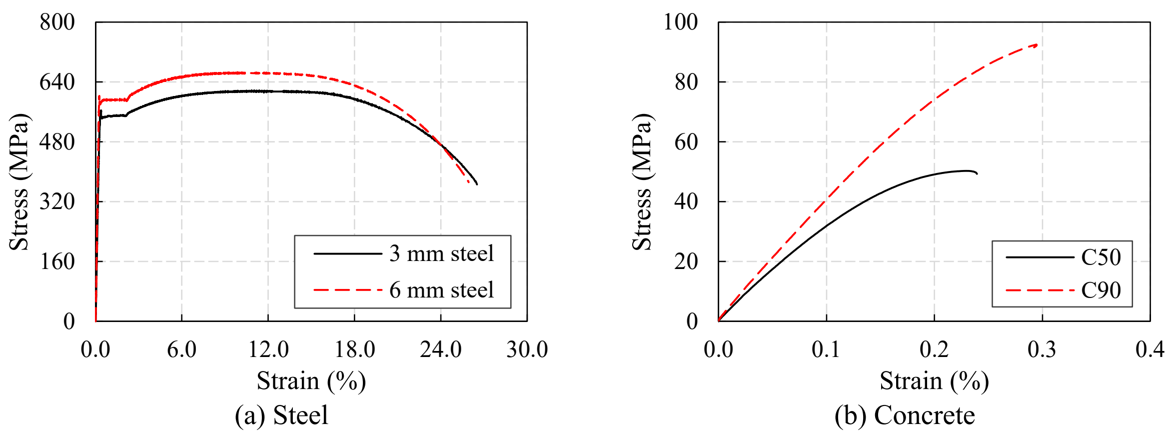
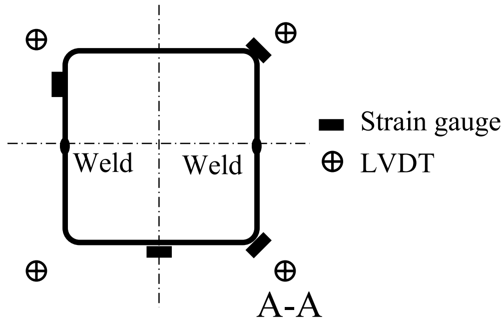
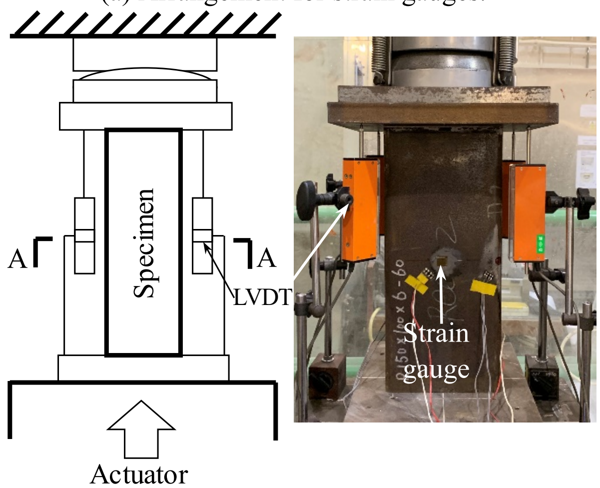


Fig. 10. Typical stress-strain curves of steel coupon and concrete cylinder tests.

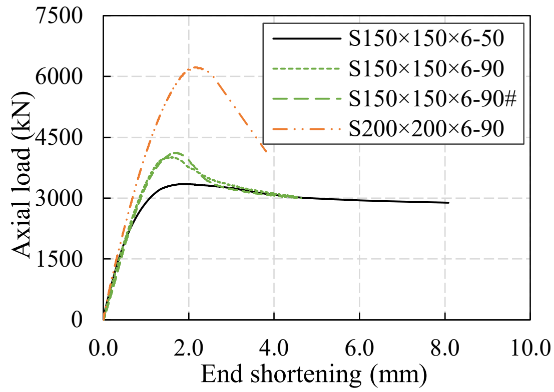


(a) Arrangement for strain gauges.

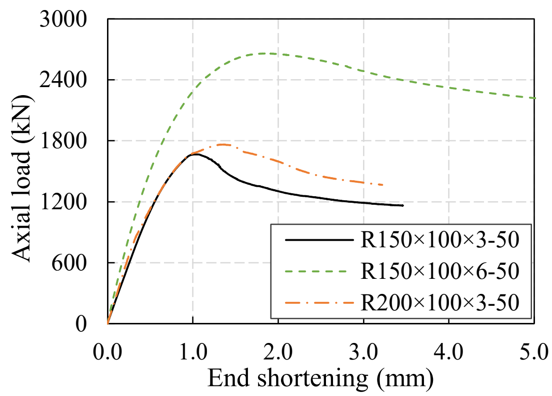


(b) Arrangement for stub column tests.

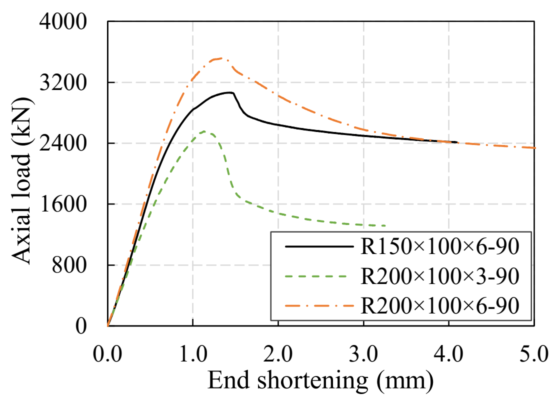
Fig. 11. Schematic view and experimental view of stub column tests.



(a)



(b)



(c)

Fig. 12. Axial load versus end shortening curves of stub column tests.