Experimental study of square and rectangular CFDST sections with stainless steel outer tubes under axial compression

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Abstract:

A comprehensive experimental investigation into the axial compressive response of concrete-filled double skin tubular (CFDST) sections with stainless steel square and rectangular outer tubes is presented. A total of 28 tests was carried out. The experimental setup and procedures are described, and the test observations are fully reported. The test results are employed to assess the applicability of the current European and North American design provisions for composite carbon steel members to the design of the studied CFDST cross-sections. Modifications to the current design codes are also considered—a higher buckling coefficient k of 10.67 to consider the beneficial restraining effect of the concrete on the local buckling of the stainless steel outer tubes and a reduction factor η to account for the effective compressive strength of high strength concrete. Overall, the comparisons revealed that the existing design rules may generally be safely applied to the prediction of the compressive resistance of CFDST cross-sections with stainless steel outer tubes, while the modified design rules offered greater accuracy and consistency.

Keywords: Cold-formed, Composite structures, CFDST, Stainless steel, Experiments, Testing.

INTRODUCTION

Concrete-filled double skin tubular (CFDST) sections consist of two metal tubes—an outer and inner tube—with concrete infilled between the tubes. CFDST sections possess the high strength, stiffness and ductility as other composite sections, and provide good fire resistance since the concrete infill provides protection to the inner tube at elevated temperatures (Lu et al. 2010). CFDST sections share the constructability benefits of concrete filled tubular (CFT) sections, with the steel tubes acting as permanent formwork, but will typically be lighter owing to the absence of the inner core of concrete.

Stainless steel members have been utilized in construction increasingly year on year over the past few decades for their excellent combination of corrosion resistance and mechanical properties (Gardner 2005). There are multiple grades of stainless steel, with the austenitics being the most commonly used in the construction industry, but lean duplex and ferritic stainless steels, which contain less nickel, offer attractive alternatives due to their good mechanical properties along with competitive cost that are appropriate for many applications (Cashell and Baddoo 2014). In the studied form of construction, the metal tubes interact with the sandwiched concrete, which leads to efficient material utilisation, and the presence of the inner tube allows the stainless steel outer tube thickness to be reduced, thus improving the cost-effectiveness of the system. In this study, a novel type of CFDST section is therefore proposed, employing carbon steel for the inner tube and stainless steel for the outer tube.

Previous experimental studies of CFDST members are scarce and most of which have focused on CFDST sections employing circular or square carbon steel tubes and sandwiched concrete grades up to 72 MPa (Wang et al. 2016). Investigations into CFDST members were first carried out at Monash University, where Zhao and Grzebieta (2002) studied the compressive behavior of CFDST members with square inner and outer cross-sections through eight stub column tests. Further work on CFDST stub columns with rectangular inner and outer cross-sections was described by Tao et al. (2004), where design formulae incorporating the confinement effect of the sandwiched concrete were proposed. Tao and Han (2006) conducted two more stub column tests on CFDST sections with rectangular hollow section (RHS) inner and outer tubes and the load-deformation relationship of the composite section was predicted using a theoretical model. The axial compressive behavior of rectangular stainless steel CFT sections was examined by Ellobody and Young (2006), Young and Elloboday (2006), Uy et al. (2011), Lam et al. (2017) and Li (2017); the significant influence of the slenderness of the metal tube on the compressive strength and ductility of the studied CFT stub columns was highlighted in these studies. Uy et al. (2011) found a substantial difference between the structural performance of stainless steel CFT columns and carbon steel CFT columns, owing principally to the rounded stress-stain response of the stainless steel material. Theofanous and Gardner (2009), Afshan and Gardner (2013), Huang and Young (2013, 2014a), and Zhao et al. (2015, 2016) have studied the structural performance of lean duplex and ferritic stainless steel RHS members, and the influence of the particular characteristics of these stainless steel grades has been examined and suitable design recommendations have been proposed. To date, investigations into CFDST sections employing stainless steel as the outer tubes are very limited, while the behavior of CFDST members with lean duplex or ferritic stainless steel outer tubes remains unexplored.

This paper presents an experimental program conducted on CFDST sections with carbon steel inner tubes, lean duplex or ferritic stainless steel outer tubes, and three grades of concrete infill. The test setup, procedures and observations are fully reported. The test results are employed to evaluate the applicability of the European Code EN 1994-1-1 (CEN 2004a) and two American Specifications AISC 360 (AISC 2016) and ACI 318 (ACI 2014) to the design of the CFDST sections studied herein. Modifications to the design treatment in the areas of local buckling of the outer tubes and the effective compressive strength of the concrete are also considered.

EXPERIMENTAL INVESTIGATION

Test specimens

Typical CFDST sections with cold-formed carbon steel SHS as the inner tubes and cold-formed stainless steel (a) RHS and (b) SHS as the outer tubes are presented in Fig. 1. The stainless steel grades employed in the present study were lean duplex stainless steel, grade EN 1.4062 (2202), which comprises only 2.6% nickel, and ferritic stainless steel, grade EN 1.4003 (410), which contains an even lower nickel content of 0.4%. Lean duplex stainless steel RHS 150×80×3 mm (depth × width × thickness) and SHS 100×100×3 mm or ferritic stainless steel RHS 100×80×4 and 120×80×3 mm were adopted as the outer tubes. The inner tubes were grade S275 (ASTM A 36) carbon steel SHS 40×40×4, 40×40×1.5, 20×20×2.5, and 20×20×1.5 mm. The nominal stub column length (L) was 2.5×H_o, which was deemed appropriately short to prohibit global buckling, yet adequately long to avoid end effects.

The CFDST specimens were prepared by first precisely locating the inner tubes and outer tubes concentrically, and then welding steel strips (10 mm deep and 2 mm thick) to the tubes near both ends of the stub columns to fix their relative positions, as detailed in Fig. 2. Together, the outer and inner tubes were wire cut flat and straight before casting concrete. The concrete was compacted to reduce the volume of air voids. Strain visualization grids were painted onto the specimen surfaces. Geometric measurements were carefully taken: the width and depth of the cross-sections were measured using a Mitutoyo digital caliper; a Mitutoyo digital micrometer was employed for measuring the thickness and the corner radii were measured using Moore Wright radius gauges. The average measured values are presented in Table 1, where B, H and t are the metal tube dimensions—width, height and thickness, which are differentiated by subscripts (o for outer and i for inner) in the symbols, r_{int} and r_{ext} are the internal and external corner radii, and A_i, A_o and A_c correspond to the calculated cross-sectional areas of the carbon steel inner tube, stainless steel outer tube and sandwiched concrete.

The CFDST test specimens were labelled such that the material, shape and dimensions of the outer and inner tubes, as well as the grade of the concrete infill can be identified. For example, the label LS100×3-NS40×4-C40R defines the following specimen: the first letter “L” refers to lean duplex stainless steel (“F” is used for ferritic stainless steel); the second letter “S” means SHS (“R” is used for an RHS); this is followed by the nominal dimensions of the SHS or RHS outer tube –100×3 mm ($H_o \times t_o$); for RHS, $H_o \times t_o$ is used. The hyphens in the label separate the information of the outer tube, inner tube and concrete grade, so in this case the notation “NS40×4” refers to the inner tube, where the letter “N” represents normal strength carbon steel and the letter “S” indicates the SHS shape with the nominal dimensions of 40×4 mm. The term after the second hyphen describes the sandwiched concrete, where the letter “C” followed by the value of the concrete strength in MPa (40 MPa) designates the nominal concrete grade. For repeated tests, the letter “R” is added as a suffix to the label.

Material properties

Longitudinal tensile coupon tests were carried out to obtain the material properties of the metal tubes. Since cold-formed metal tubes undergo strength enhancement due to cold-working during production, which is particularly pronounced in the corner areas of sections, coupons were extracted from both the flat and corner regions of the tested tubes. The flat and corner coupons were taken from the positions shown in Fig. 3(a) and (b) for the outer and inner tubes. Each flat coupon was prepared with a 12.5 mm parallel width and a 50 mm gauge length, while each corner coupon had a 4 mm parallel width and a 25 mm gauge length. For the corner coupons, two 10.5 mm diameter holes were drilled and reamed at 17 mm from each end. The flat coupons were gripped using a set of end-clamps, while a pair of steel rods was inserted into the drilled holes of the corner coupons, through which the tensile force was applied. A contact extensometer was attached to the coupons and a strain gauge was affixed to each side of the coupons at mid-length. All the longitudinal tensile coupon tests were displacement controlled and conducted in an MTS 50 kN testing machine. A constant displacement rate of 0.05 mm/min was used in the elastic range of the stress–strain curves, while a higher rate of 0.4 mm/min was used in the inelastic range; in the post-ultimate range, a rate of 0.8 mm/min was adopted, as recommended in Huang and Young (2014b).

The static 0.2% proof stress $\sigma_{0.2}$, the static ultimate tensile stress σ_u, the Young's modulus E, the elongation at fracture ε_f, and the strain hardening exponents n and m, used in the compound Ramberg-Osgood (R-O) material model (Mirambell and Real, 2000; Rasmussen, 2003; Arrayago et al., 2015; Gardner and Yun, 2018), as determined from the coupon tests are provided in Table 2. It can be observed that the process of cold-forming has resulted in a moderate enhancement in both $\sigma_{0.2}$ and σ_u in the corner regions, though this is accompanied by a ductility reduction. The full stress-strain curves are presented in Fig. 4(a) and (b) for the outer and inner tubes, respectively.

Concrete cylinder tests were performed to obtain the material properties of the concrete. Three concrete grades—C40, C80, and C120 MPa—were produced in the laboratory using commercially available materials. Their mix proportions are presented in Table 3. For each batch of concrete, concrete cylinders were cast and cured together with the CFDST test specimens. Two concrete cylinders were tested after 28 days of casting and the remainder were tested at the days of the respective CFDST specimen tests. Table 4 summarizes the mean measured strengths and the test number for each concrete grade.

Stub Column Tests

A total of 28 tests on the CFDST stub columns was performed, three for each of the eight series and four repeated tests. Axial compressive force was applied to the CFDST stub columns in an INSTRON 5000 kN capacity testing machine. Two reinforcing frames (see Fig. 5) were clamped near the ends of the specimens to prevent localized failure due to end effects. The top surface of the specimens was uneven due to concrete shrinkage; a thin layer (< 1 mm) of plaster was thus utilised to fill the small gap. The plaster was then left to harden under an approximately 2 kN applied load. This ensured that the inner tube, the outer tube and the sandwiched concrete were loaded simultaneously during the tests. Three 50 mm range transducers (LVDTs) were placed between the testing machine platens to measure the axial shortening of the tested specimens; the layout of the LVDTs is illustrated in Fig. 6. A constant 0.4 mm/min displacement rate was used to drive the bottom end platen upwards in order to apply load to the stub columns. All the stub column tests were stopped at a similar maximum axial strain of approximately 0.05.

Localized strains were monitored in seven of the stub columns. These seven specimens cover a variety of the key parameters, including the outer and inner tube slenderness, as well as the concrete strength. For each of these specimens, 12 strain gauges were mounted to the outer tube at 1/4, 1/2 and 3/4 points along the stub column lengths, in order to monitor the plate deformations and strain development histories, as presented in Fig. 6. Of the 12 strain gauges, three pairs of longitudinal and transverse strain gauges were affixed to the outer face of each cross-section adjacent to the weld, while the other three pairs were positioned in the corner region.

TEST OBSERVATIONS

Failure modes

The failure modes of the tested CFDST stub columns featured outward local buckling of the stainless steel outer tube, crushing of the infill concrete, as well as local buckling of the carbon steel inner tube. Photographs of typical failure modes are displayed in Fig. 7. The buckling modes of both tubes were influenced by concrete shear failure, as shown for specimen FR100×4-NS20×1.5-C40 and LS100×3-NS40×1.5-C40 in Fig. 7. Outward only local buckling of the outer tubes was observed for all the tested specimens, as presented in Fig. 7 (a) and (c), but, different failure modes were detected for the inner tubes, i.e. inward and outward local buckling and inward only local buckling, as shown in Fig. 7 (b) and (d), respectively. The outward only local buckling mode of the outer tube is due to the presence of the concrete, which inhibits inward deformations, with concrete dilation under compression promoting positive contact between the concrete and the outer tube. Local buckling of the outer tube is also relatively insensitive to loss of support due to concrete failure since the cracking is very localized in comparison to the local buckling half-wavelength of the plates. For the inner tube, there was a trend of inward only local buckling for the inner tubes with high plate slenderness, whereas both inward and outward local buckling was detected for the more compact inner tubes. For the tested range of properties, neither the steel type nor the concrete compressive strength appeared to have any significant influence on the failure mode.

Load versus axial deformation relationships

The load (P) versus average axial strain (ε) curves for all the stub column specimens are plotted in Fig. 8, where P is the applied load recorded by the load actuator and ε is the average axial strain, defined as the average axial shortening (Δ), calculated from the LVDT readings, divided by the original specimen length (L). The experimental peak loads P_{exp} are presented in Table 1. In general, it may be observed that the concrete strength significantly influences the ductility of the stub columns and their cross-sectional strengths. The ductility of the CFDST stub columns was assessed through the ductility index (DI) given by Eq. (1), as proposed in Tao et al. (2004), and widely adopted for concrete-filled tubular members in (Yang et al. 2008; Jamaluddin et al. 2013; McCann et al. 2015).

$$DI = \frac{\Delta_{85\%}}{\Delta_u} \qquad (1)$$

where $\Delta_{85\%}$ is the axial displacement when the load decreases to 85% of the ultimate load and Δ_u is the axial displacement at ultimate load. Values of the ductility index obtained from each of the stub column tests are presented in Table 1. A low DI value indicates that the load drops off quickly beyond the peak load, whereas a high value indicates an ability to maintain at least 85% of P_{exp} with a considerable associated deformation. Values of DI for each test series are plotted against the measured concrete cylinder strength in Fig. 9. Overall, it may be seen that higher concrete strengths result in increased compressive resistance, but lower ductility. The exception in Fig.9 (b) is caused by the shapes of the load versus average axial strain curves for the C80 and C120 specimens, which resulted in higher DI values for the adopted definition of ductility. However, as observed in Fig8 (c) and (d), the specimens with higher concrete strengths (C80 and C120) still showed reduced ductility relative to their C40 counterparts. The effect of the slenderness of the outer tube on ductility is also assessed through comparisons among specimens with the same inner tubes and concrete grades but varying h_o/t_o ratios, as shown in Fig. 10. It may be observed that specimens with a more compact outer tube displayed greater ductility, owing to the reduced susceptibility to local buckling and the improved confinement afforded to the concrete.

Transverse to longitudinal strain ratios

The transverse to longitudinal strain ratios in the outer steel tubes of the tested specimens can be used to assess the degree of confinement provided to the concrete (Uy et al. 2011); typical examples are plotted against the normalised axial load in Fig. 11, where the strain ratios may be seen to be approximately 0.3 in the early stages of loading. The ratios increase gradually until the loads reach around 0.8 P_{exp}, and then grow sharply as the loads approach P_{exp}. This can be explained with reference to the development of confinement in the CFDST stub columns (Chan et al. 2015). The Poisson's ratio of concrete (typically equal to about 0.2) is lower than that of stainless steel (approximately 0.3) in the early (elastic) stages of loading, during which the confinement afforded by the outer tube to the concrete core is negligible. As the load increases, the concrete enters the plastic regime, and the effective Poisson's ratio increases; this causes greater lateral expansion of the concrete, increasing the contact pressure against the outer tube, leading to increased confinement and enhanced transverse strains. Thus, increasing ratios of transverse to longitudinal strain correspond to increasing levels of confinement to the concrete core.

DISCUSSION AND ASSESSMENT OF CURRENT DESIGN CODES

General

In this section, the applicability of current design rules to the design of the studied CFDST cross-sections is appraised. The experimental ultimate loads are compared with the resistance predictions determined from the current European Code EN 1994-1-1 (EC4) (CEN 2004a) and North American design provisions —AISC 360 (AISC 2016) and ACI 318 (ACI 2014) for composite carbon steel members, as shown in Tables 5-7. For the slender cross-sections, the effective width concept was employed to consider the effect of local buckling of the outer tubes; note that the inner tubes were all fully effective in this study. Modifications to the existing rules are also considered. In the comparisons presented, the measured material properties and geometric dimensions of the test specimens have been employed, and all partial safety factors have been taken to be equal to unity. The code limitations on steel strength and concrete strength are often exceeded, but comparisons are still presented in order that possible extension of the range of applicability of the codes can be assessed.

EN 1994-1-1 (EC4)

The compressive design resistance of rectangular or square carbon CFT sections in EC4 (CEN 2004b) is a simple summation of the steel tube and concrete contributions. Account is taken of the higher resistance of the concrete, caused by confinement from the outer steel tube, by adopting a concrete coefficient of 1.0, rather than 0.85 (CEN 2004b). The cross-section capacity (P_{EC4}) of a concrete-filled rectangular CFDST compression member is thus given by Eq. (2).

$$P_{\mathrm{EC4}} = A_o \sigma_{0.2,o} + A_c f_c + A_i \sigma_{0.2,i} \tag{2}$$

where $\sigma_{0.2,o}$ and $\sigma_{0.2,i}$ correspond to the outer tube and inner tube 0.2% proof stresses, while f_c is the concrete cylinder compressive strength measured on the day of the corresponding stub column tests.

A slenderness limit of $H_o/t_o \leq 52(235/f_y)^{0.5}$ for the outer tube is also specified in EC4 (CEN 2004b), beyond which local buckling needs to be explicitly accounted for. In this study, the limit has been modified for stainless steel to reflect the differences in material yield strength and Young's modulus, as given by Eq. (3),

$$\frac{H_o}{t_o} \leq 52\left(\frac{235}{\sigma_{0.2,o}}\frac{E_o}{210000}\right)^{0.5} \tag{3}$$

It is worth noting that when the presence of the concrete is ignored in the classification of the cross-section, the outer tubes of the LS100×3 series are class 4 (slender). However, when the beneficial influence of the concrete infill in inhibiting local buckling is considered, i.e. assessing the slenderness of the outer tube against the limit given by Eq. (3), these cross-sections are deemed to be non-slender. The outer tubes of the specimens in the LR150×3 and FR120×3 series exceed the limit of Eq. (3) and are hence deemed to be slender, despite the influence of the concrete, as shown in Table 5. In these cases, the effective width equations provided in EN 1993-1-4 (CEN 2006a; Gardner and Theofanous 2008), as given by Eqs. (4) and (5), are adopted for calculating the effective area of the outer tube:

$$\rho = \frac{0.772}{\bar{\lambda}_p} - \frac{0.079}{\bar{\lambda}_p^{\,2}} \tag{4}$$

$$\bar{\lambda}_p = \sqrt{\frac{\sigma_{0.2,o}}{\sigma_{cr}}} = \sqrt{\frac{12(1-\upsilon^2)\sigma_{0.2,o}}{k\pi^2 E_o}}\,(h_o/t_o) \tag{5}$$

where ρ is the reduction factor for local buckling, $\bar{\lambda}_p$ is the element slenderness, v is the Poisson's ratio equal to 0.3, h_o is the flat element height of the outer tube (replaced by b_o for the flat element width), k is the buckling coefficient, taken equal to 4 for plates with simply supported boundary conditions in pure compression (CEN 2006b) and E_o is the outer tube Young's modulus. Table 5 presents the reduction factors to the width (ρ_b) and height (ρ_h) of the stainless steel outer tubes for the LR150×3 and FR120×3 series, as well as the overall effective areas.

AISC 360

The American Specification AISC 360 (AISC 2016) for the design of concrete-filled composite members is also assessed herein. In AISC 360, concrete-filled composite cross-sections are categorised into compact, noncompact and slender sections according to the width-to-thickness ratio of the outer tube. The resulting classification influences the calculation of the axial compressive strength. A compact section is able to reach the yield strength of the steel tube and develop a concrete compressive strength of $0.85f_c$. A noncompact section confines the concrete to a lesser extent, with $0.70f_c$ being used in the design calculation, after which it is assumed that the concrete volumetric dilation cannot be confined adequately since the noncompact steel tube undergoes local buckling (Chen and Han 2007). A slender section can neither reach the yield strength of the steel tube nor confine the concrete beyond 0.70 f_c (Lai et al. 2014).

The limiting width-to-thickness ratios, i.e. λ_p for compact/noncompact and λ_r for noncompact/slender, are tabulated in Table 6 for the outer tubes of all the tested sections. In this study, all tested CFDST sections are classified as compact. Note that a local buckling coefficient k = 10.67 is employed in AISC 360 to reflect the influence of the concrete in restraining plate buckling (Uy and Bradford 1996). The cross-section capacities (P_{AISC}) can be thus predicted using Eq. (I2-9b) of AISC 360 (AISC 2016). It should be noted that the term for the reinforcing bars is replaced by the inner tube. The structural behavior of the inner tube is however different from that of the reinforcing bars. Reinforcing bars have little or no axial resistance upon crushing of the concrete, whereas the inner tube still continues to sustain load and can thus be treated as an independent term in the resistance function. Therefore, the compressive cross-section strengths (P_{AISC}) of the tested CFDST stub columns are calculated from Eq. (6).

$$P_{\mathrm{AISC}} = A_o \sigma_{0.2,o} + 0.85 A_c f_c + A_i \sigma_{0.2,i} \tag{6}$$

ACI 318

The American Concrete Institute design guidelines ACI 318 (ACI 2014) for CFT sections are also assessed herein. According to ACI 318 (ACI 2014), the cross-section resistance (P_{ACI}) is determined from Eq. (7).

$$P_{\mathrm{ACI}} = A_o \sigma_{0.2,o} + 0.85 A_c f_c + A_i \sigma_{0.2,i} \tag{7}$$

It should be noted that the gross area of the outer tube can only be used provided that the thickness of the outer tube satisfies $t_o \geq H_o(\sigma_{0.2,o}/3E_o)^{0.5}$, as specified in Section 10.3.1.6 of ACI 318 (ACI 2014). The test specimens in the LR150×3 and FR120×3 series exceed the above limit, as shown in Table 7. The compressive design resistance of the sections is therefore not explicitly covered by ACI 318 but, in order to enable comparisons to be made, the effective width expressions in the American Specification SEI/ASCE-8-02 (ASCE 2002) were utilised in the calculations. The effective areas of the outer tubes were determined using the local buckling reduction factors ρ, obtained from Eqs. (8)-(9),

$$\rho = \frac{1 - 0.22/\bar{\lambda}_p}{\bar{\lambda}_p} \tag{8}$$

$$\bar{\lambda}_p = \left(\frac{1.052}{\sqrt{k}}\right)\frac{h_o}{t_o}\left(\sqrt{\frac{F_n}{E_o}}\right) \tag{9}$$

where $\bar{\lambda}_p$ is the local slenderness, termed λ in SEI/ASCE-8-02 (ASCE 2002), F_n is the column buckling stress, calculated using the iterative tangent modulus design approach, and the other symbols are as previously defined. Taking k equal to 4 according to SEI/ASCE-8-02 (ASCE 2002), F_n equal to $\sigma_{0.2,o}$ due to the short length of the stub columns and ν = 0.3, the local slenderness calculated using Eq. (9) is the same as that obtained from Eq. (5), and hence the same symbol has been adopted herein. Table 7 presents the reduction factors and effective areas of the stainless steel outer tubes for the LR150×3 and FR120×3 series.

Discussion and Design Recommendations

The ratio of test-to-design strength ($P_{exp}/P_{\mathrm{code}}$) for each of the tested CFDST specimens is presented in Tables 5-7. The comparisons indicate that the unmodified design models (leading to the capacity predictions P_{EC4}, P_{AISC} and P_{ACI}), which were developed for carbon steel CFT sections, offer generally conservative strength predictions for the studied CFDST sections. The predicted axial capacities determined according to EC4 (P_{EC4}) show the best agreement with the test strengths, with the mean value of $P_{exp}/P_{\mathrm{EC4}} = 1.1$, owing mainly to the higher adopted concrete coefficient. Of the North American provisions, AISC 360 (AISC 2016) yielded slightly more accurate axial capacity predictions (P_{AISC}) than ACI 318 (ACI 2014), due to consideration of the beneficial effect of the concrete infill on the local buckling behavior of the outer tube; this is discussed further below.

The ratios of test-to-predicted ultimate loads ($P_{exp}/P_{\mathrm{code}}$) are plotted against the concrete cylinder strength in Fig. 12. The comparisons reveal that EC4 (CEN 2004b) provides less conservative predictions for specimens with high strength concrete (C80 and C120) than their counterparts with normal strength concrete (C40), particularly for sections within the specified code slenderness limits. This observation has previously been made for concrete-filled tubes; to remedy this, an effective compressive strength, as defined in EN 1992-1-1 (CEN 2004a), is used for concrete strengths greater than 50 MPa and below 90 MPa. The effective strength is determined by multiplying the concrete strength by a reduction factor η, as given by Eq. (10). For concrete strengths beyond 90 MPa, a constant reduction factor η of 0.8, as proposed by Liew et al. (2016), is employed herein to determine the effective compressive strength for sections falling within the specified code slenderness limits. The values of η, as calculated from Eq. (10), are shown in Table 5 for the specimens tested in the present study.

$$\eta = \begin{cases} 1.0-(f_c-50)/200 & 50\ \text{MPa} < f_c \le 90\ \text{MPa} \\ 0.8 & f_c > 90\ \text{MPa} \end{cases} \tag{10}$$

The resulting ratios of test-to-modified predicted strengths ($P_{exp}/P_{\text{code*}}$) are presented in Tables 5-7. The inclusion of η leads to more consistent resistance predictions across the different concrete strengths for EC4 and ACI 318, as highlighted in Fig. 12. However, it was found that the design rules incorporating the effective compressive strength of concrete results in more conservative and scattered predictions for AISC 360, particularly for the specimens with LR150×3 and FR120×3 sections as the outer tubes. This is due to the influence of the different concrete grades on the axial capacity of these sections not being as significant as that on the specimens with the more compact sections, i.e., those with LS100×3 and FR100×4 sections as the outer tubes.

The structural behavior of concrete-filled tubular members and hollow tubular members is fundamentally different. As observed in the experiments, the presence of the concrete infill alters the local buckling mode of the outer steel tube by restricting it from buckling inwards (Lai et al. 2014). Uy and Bradford (1996) conducted a semi-analytical investigation using the finite strip method into the elastic local buckling of steel plates in composite steel-concrete members. It was shown that the buckling coefficient k increases from 4 for conventional (two-way) local buckling of simply-supported plates to 10.30 for outward only buckling. A further theoretical study (Bradford et al. 1998) using the Rayleigh-Ritz method, indicated k to be 10.67, which is about 2.67 times that of the unfilled case.

Modification to the current design rules in EC4 and ACI 318, taking the buckling coefficient k as 10.67, rather than 4, in calculating the effective areas of the outer tubes is therefore considered herein. In AISC 360 (AISC 2016), this beneficial effect of the presence of the concrete infill is already considered in the cross-section slenderness limits. Concrete-filled columns with compact sections reach their yield load before the development of local buckling in the outer tube and thus the outer tube is fully effective. Theoretically the noncompact section limit of $1.40(E/F_y)^{0.5}$ given in AISC 360 (AISC 2016) for hollow section could be increased by $\sqrt{2.67}$ times, to $2.29(E/F_y)^{0.5}$. Based on available experimental data and other theoretical studies, the constant has been increased to 2.26 in AISC 360 (Leon et al. 2007).

The modified axial capacity predictions, with k = 10.67, from EC4 and ACI 318 for the tested specimens are shown in Tables 5 and 7, respectively. The local buckling reduction factors ρ are now almost equal to unity for all the series, which indicates that the areas of the outer tubes are essentially fully effective. The comparisons show that the mean ratios of test-to-modified design strengths ($P_{exp}/P_{\mathrm{code}^\wedge}$) are equal to 1.07 and 1.15, with coefficients of variation (COVs) of 0.062 and 0.049 for EC4 and ACI 318, respectively. This illustrates that the modified design rules yield improved consistency and accuracy in the prediction of the compressive resistance of CFDST members.

Some unconservative predictions again arise for the higher concrete strengths (C80 and C120) though, as shown in Fig. 13. Therefore, the reduction factor η is employed as before. The resulting ratios of test-to-predicted modified design strengths ($P_{exp}/P_{\mathrm{code}^\wedge*}$) are plotted against the measured concrete cylinder strength in Fig. 13 and the overall mean ratios of $P_{exp}/P_{\mathrm{code}^\wedge*}$ are shown in Tables 5-7. The results reveal that the modified design rules incorporating both the effective compressive strength of concrete and k = 10.67 provide safe-sided predictions with very good consistency for CFDST with concrete grades extending to C120.

CONCLUSIONS

An experimental investigation into the behavior of concrete-filled double skin tubular (CFDST) stub columns has been presented in this paper. The stub column test results, together with the measured material properties and geometric properties, have been reported. The test strengths were compared with the capacity predictions for conventional concrete-filled carbon steel tubular columns given in the European Code EN 1994-1-1 and two American

Specifications (AISC 360 and ACI 318). Overall, it has been found that EC4 provides good resistance predictions for the tested specimens, which featured steel sections of relatively stocky proportions, while the predictions from the American Specifications AISC 360 (AISC 2016) and ACI 318 (ACI 2014) were on the safe side but rather conservative. The effect of modifications to EC4 and ACI 318 to consider outward only local buckling of the outer tube, by taking the buckling coefficient k as 10.67 instead of 4, was assessed. Concrete strengths were also adjusted, by applying a reduction factor η to high strength material. Comparisons revealed that modifying the existing design rules by incorporating the revised buckling coefficient results in more accurate and consistent capacity predictions, while the approach of using effective concrete strengths allows the rules in EC4 and ACI 318 to be safely extended to the design of CFDST stub columns with concrete compressive cylinder strengths up to 120 MPa.

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