Confinement effects for rubberised concrete in tubular steel cross-sections under combined loading

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Received: 5 November 2020 / Revised: 15 February 2021 / Accepted: 20 February 2021 / Published online: 13 March 2021
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Abstract

This paper describes an experimental investigation into confinement effects provided by circular tubular sections to rubberised concrete materials under combined loading. The tests include specimens with 0%, 30% and 60% rubber replacement of mineral aggregates by volume. After describing the experimental arrangements and specimen details, the results of bending and eccentric compression tests are presented, together with complementary axial compression tests on stub-column samples. Tests on hollow steel specimens are also included for comparison purposes. Particular focus is given to assessing the confinement effects in the infill concrete as well as their influence on the axial–bending cross-section strength interaction. The results show that whilst the capacity is reduced with the increase in the rubber replacement ratio, an enhanced confinement action is obtained for high rubber content concrete compared with conventional materials. Test measurements by means of digital image correlation techniques show that the confinement in axial compression and the neutral axis position under combined loading depend on the rubber content. Analytical procedures for determining the capacity of rubberised concrete infilled cross-sections are also considered based on the test results as well as those from a collated database and then compared with available recommendations. Rubber content-dependent modification factors are proposed to provide more realistic representations of the axial and flexural cross-section capacities. The test results and observations are used, in conjunction with a number of analytical assessments, to highlight the main parameters influencing the behaviour and to propose simplified expressions for determining the cross-section strength under combined compression and bending.

Keywords Rubberised concrete · Concrete confinement · Infilled steel tubes · Axial–bending interaction · Inelastic response

List of symbols

Latin uppercase

A_c	Concrete area
A_p	Inelastic energy
A_s	Steel cross-sectional area
A_y	Elastic energy
D	Diameter
E_s	Steel elastic modulus,

E_{rc}	Concrete elastic modulus
L	Specimen length
M, M_i	Bending moment
$M_{max,R}$	Maximum moment
$M_{pl,R}, M_{u,i}$	Bending moment capacity
M_{test}	Test bending moment
$N_{pl,R}, N_{u,i}$	Axial capacity
N_{res}	Sectional analysis predicted strength
N_{test}	Test axial strength
N_{ST}	Cross-sectional capacity hollow steel sections
W_{pl}	Plastic section modulus of
W_{ps}	Steel plastic section modulus,
W_{psn}	Steel plastic section modulus from $2 \times h_n$
W_{pc}	Concrete plastic section modulus,
W_{pcn}	Concrete plastic section modulus from $2 \times h_n$
$\emptyset$	Diameter

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Latin lowercase

e	Eccentricity
e_{test}	Test eccentricity
d	Depth

$d_{g,max}$	Aggregate size
$f_{c,28d}$	Concrete compressive strength
f_{ctr}	Concrete splitting tensile strength
f_{rc}	Rubberised concrete compressive strength
f_{c0}	Reference concrete compressive strength
f_y	0.2% Steel proof yield strength
f_u	Ultimate steel strength,
h	Distance between tension and compression fibres
h_n	The distance between the neutral axis and the cross-section centreline
u	Axial shortening
u_i	Shortening

Greek letters

α	Interaction diagram coefficient
β	Interaction diagram coefficient
δ	Displacement
ϵ	Strains
ϵ_1	Lateral strain
ϵ_2	Longitudinal strain
$\epsilon_{cc,1}$	Conventional concrete axial strain at peak of confined
$\epsilon_{c0,1}$	Reference concrete axial strain at crushing of
ϵ_c	Compression strain
ϵ_{rcc1}	Rubberised concrete confined axial strain
$\epsilon_{rc,1}$	Rubberised concrete unconfined crushing strain
$\epsilon_{rcc,1}$	Rubberised concrete strain at peak
$\epsilon_{rc1,1}$	Concrete axial strain at crushing
$\epsilon_{rc1,2}$	Concrete lateral strain at crushing
ϵ_t	Tension strain
ϵ_u	Ultimate steel strain
η_s	Steel confinement coefficient
η_c	Concrete confinement coefficient
θ_{peak}	Peak rotation
θ_{max}	Maximum rotation
κ_{cc}	Confinement factor
λ	Aggregate size factor
λ_{cc}	Concrete contribution ratio
λ_{rcc}	Confinement effectiveness factor
μ_u	Ductility index
μ_ϕ	Curvature ductility
ξ	Confinement parameter
ρ_{vr}	Rubber content
σ	Stresses
χ_{rcc}	Modification factor
ϕ_y	Yield curvature
ϕ_p	Plastic curvature
ϕ_u	Ultimate curvature

1 Introduction

The potential benefits of rubberised concrete (RuC) compared with conventional concrete materials (CCMs) were highlighted in various recent investigations [1]. These studies indicated that RuC possesses enhanced ductility, energy absorption, durability and sound insulation, as well as lower unit weight in comparison with CCM [2]. In contrast, the partial replacement of mineral aggregates with rubber particles had a detrimental effect on the fresh and mechanical properties, particularly in terms of the reduction in elastic modulus as well as the compressive and tensile strengths [3].

The merits of RuC materials motivated a number of studies on structural members with confining reinforcement such as stirrups and spiral arrangements. Such reinforced rubberised members (RRuC) were shown to develop nearly twofold the ductility levels of their conventional reinforced concrete (RC) counterparts [4]. Moreover, under reverse cycling loading, RRuC exhibited an increase in energy dissipation of up to 150% in comparison with conventional elements [5]. However, the enhancement in ductility and energy dissipation was counterbalanced by a reduction in the loading carrying capacity due to the relatively modest levels of confinement mobilised.

External confinement, such as using fibre-reinforced polymer (FRP), provides a viable approach to recover some or all of the strength loss due to the presence of rubber [6]. FRP laminates were shown in previous studies to improve the capacity of RuC members under combined compression and bending loading and to provide high rotation capacity compared with RRuC members with internal stirrup confinement [7]. The capacity of FRP-confined RuC was between two to four times higher than unconfined RuC when the rubber content was 30% and 60%, respectively [8].

As an alternative to external FRP or internal hoop confinement, steel tubes can offer a favourable confinement solution [9]. In this case, the congestion of reinforcement in the connection zones can be minimised and the tube also provides a permanent formwork for the concrete core. Additionally, the infilled concrete acts as a resistance mechanism to inward local buckling of the steel tube [10]. In circular concrete-filled steel tube (CFST) specimens, higher levels of circumferential stresses occur in the tube walls, allowing higher confinement effects to develop in the concrete core [11]. As a result, CFST develops significantly higher confinement effects when circular cross-sections are used, in comparison with square or rectangular forms [12]. Square and rectangular rubberised concrete filled steel tubes (RuCFSTs) were shown to exhibit a generally ductile behaviour [13], both under monotonic and cyclic loading with some degradation effects occurring under cyclic conditions due to more pronounced local buckling [14].

Axial compression tests on RuCFST stub columns showed that the presence of rubber particles can potentially improve the strength, ductility and energy absorption capacity of such members [15]. For low rubber content ratios, the reduction in strength was counterbalanced by the increase in confinement provided by the circular tube [16]. Displacement ductility ratios and confinement levels for RuCFST were shown to be more than twice those with square or rectangular sections [17]. Eccentric tests on RuCFST columns also indicated improved bending performance and ductility for tubular members compared to conventional concrete specimens [18].

The above-mentioned studies illustrated the potential for using RRuC- and FRP-confined RuC in various structural applications due to their ability to sustain relatively large deformations. The enhancement in performance due to steel tube confinement in compression-dominated CCM members has also been demonstrated in numerous investigations [19]. Although the performance of confined RuCFST has been assessed in several previous studies, this was primarily considering axial compression [15, 18], or under modest levels of axial load combined with bending [13, 14, 16, 20]. Most studies also investigated the overall response of RuCFST without a detailed assessment of cross-sectional behaviour under combined loading and largely focussed on relatively thick tubular sections and concrete with low rubber content. Importantly, the lack of studies on members with relatively high rubber content, of more than 30%, may in part be attributed to the reduction in concrete strength to levels typically below the limits imposed by various codified provisions [21], hence necessitating the modification of existing design provisions.

This paper examines the confinement effects of rubberised concrete within circular CFST and RuCFST cross-sections, through axial, eccentric compression and three-point bending tests. Particular focus is given to relatively high rubber content of up to 60% and comparatively thin tubes with diameter-to-thickness ratio larger than 50, for which experimental data are largely lacking. Although circular sections are typically employed for compression elements in practice, bending and low axial load cases are considered in the tests in order to validate the full cross-section strength interaction for generalised loading conditions. Detailed measurements using digital image correlation techniques are employed to assess the surface deformation characteristics and the cross-sectional response of RuCFST under combined loading. Based on the test results, supported by analytical assessments using nonlinear sectional analysis procedures, simplified expressions to determine the axial–moment cross-section strength interaction are proposed, with due account for the influence of key geometric and material properties including the rubber content.

2 Experimental programme

The concrete-infilled specimens included 0%, 30% or 60% rubber content, as a replacement of both fine and coarse mineral aggregates by volume, representing a volumetric rubber ratio (ρ_{vr}) of 0, 0.3 and 0.6, referred to herein as R00, R30 and R60 concrete, respectively. The confinement effects provided by circular tubular sections to rubberised concrete materials under axial compression were assessed by means of $6 \times \emptyset 150 \text{ mm} \times 300 \text{ mm}$ stub columns samples, $2 \times \emptyset 150 \text{ mm} \times 300 \text{ mm}$ hollow steel elements and $9 \times \emptyset 150 \text{ mm} \times 300 \text{ mm}$ cylinders. All cylindrical samples had relatively low aspect ratio ($L/D = 2.0$) and are considered stub-column material tests, which provide useful information on the interaction between the confined infill concrete and the confining steel tube on the cross-section level. In addition to axial compression tests on stub-column samples, $44 \times \emptyset 100 \text{ mm} \times 200$ cylinders and 36 cubes of $100 \times 100 \times 100 \text{ mm}$ were also tested to assess the main material properties of the rubberised and conventional concrete. Tests on six coupons were also carried out in order to assess the uniaxial properties of the steel tube materials.

It should be noted, however, that reliable assessment of axially loaded CFST members, for practical application, would necessitate further experimental assessment on specimens with higher aspect ratios ($L/D \geq 3.0$) [22]. Stub columns with $L/D \approx 3.0$ would be adequate as they would not fail in global buckling and have a representative aspect ratio to reflect the stress and strain distributions accurately [23]. Conversely, specimens with aspect ratios $L/D < 3.0$ may develop elephant foot-type buckling, which can result from end effects [24]. To complement the axial tests from this paper, a database of the results of axial tests on specimens with aspect ratios $L/D \geq 3.0$ was collated [25–28], in order to validate the axial–bending cross-section interaction diagrams described later on in the paper. The number of specimens (n), specimen aspect ratio (L/D), diameter-to-thickness ratio (D/t), steel yield strengths (f_y) and concrete compressive strengths in each investigation are: (1) $n = 15$, $L/D = 3.5$, $D/t = 60$ – 220 , $f_y = 185$ – 363 MPa , $f_c = 50$ – 80 MPa [25], (2) $n = 12$, $L/D = 3.0$, $D/t = 33$ – 67 , $f_y = 303 \text{ MPa}$, $f_c = 46.8 \text{ MPa}$ [26], (3) $n = 14$, $L/D = 3.5$, $D/t = 165$, $f_y = 200 \text{ MPa}$, $f_c = 95$ – 110 MPa [27]; (4) $n = 12$, $L/D = 3.0$, $D/t = 55$ – 88 , $f_y = 276$ – 590 MPa , $f_c = 59.8 \text{ MPa}$ [28]. In addition to the above ranges, a set of 25 tests with $L/D = 2.0$, $D/t = 21$ – 34 , $f_y = 365 \text{ MPa}$ and $f_c = 29.1$ – 75.2 MPa were considered [29].

In order to obtain a detailed insight into the confinement effects for cross-sections under combined loading, eccentric compression tests, representing combined compression and bending, on $9 \times \emptyset 150 \text{ mm} \times 600 \text{ mm}$ CFST specimens,

were carried out. Bending tests on $3 \times \text{Ø}150 \text{ mm} \times 600 \text{ mm}$ CFST and $1 \times \text{Ø}150 \text{ mm} \times 600 \text{ mm}$ hollow steel specimens, under three-point loading, were also undertaken in order to validate the full cross-section strength interaction for generalised loading conditions. All the eccentric and bending specimens had an aspect ratio $L/D = 4.0$, a range at which the end effects due to friction between the steel tube and loading platens [30] should not be significant. Additionally, such aspect ratios ensure minimum influence from the specimen slenderness on the global behaviour [31].

2.1 Concrete materials

Concrete mixes were produced with high strength cement CEM I 52,5, EN 450-1 fineness category S fly-ash, rubber particles, mineral aggregates, polycarboxylate superplasticiser admixtures and water. The rubber particles consisted of six size ranges: 0–0.5 mm, 0.5–0.8 mm, 1.0–2.5 mm, 2–4 mm, 4–10 mm and 10–20 mm. These represented 5%, 15%, 20%, 10% and 45% of the total added rubber blend, respectively, and were chosen following a detailed mix optimisation procedure in which a balance between workability and strength loss was sought [2]. The specific gravity of rubber was 1.1, and the fineness modulus of the rubber blend was 4.90. All rubber particles are reported to have 25% content of carbon black and polymers in the range of 40–55%, whereas the remaining constituents are

softeners and fillers. Fine ($< 5 \text{ mm}$) and coarse aggregates (5–10 mm) had a specific gravity of 2.65, whilst the rubber particles had a specific gravity of 0.78. The fineness modulus was 2.35 for fine aggregates and 5.88 for coarse aggregates.

Using the above constituents, three types of concrete materials were prepared with the proportions shown in Table 1. The target concrete strength for the conventional mix was 60 MPa. The mixing procedure followed primarily EN 12390-2 guidelines [32] and was adapted to RuC materials used in previous studies by the authors [1]. The procedure included adding the dry constituents in a rotary mixer until a homogenous dry mix was obtained. After adding half of the water, it was mixed together for about 180 s. In the last stage, the remaining water and admixtures were poured into the container and all constituents were mixed until a homogenous and workable mix was obtained. The fresh concrete was removed from the mixer and placed in plastic forms or steel tubes. Concrete materials placed in cylindrical plastic forms were compacted on a vibrating table, whilst those in steel tubes were compacted using a concrete vibrator. Finally, all samples were covered with plastic sheets and left under standard laboratory conditions. All samples were kept and tested at ambient temperature of $23 \pm 2 \text{ }^\circ\text{C}$ and relative humidity $\text{RH} = 30\text{--}50\%$.

To obtain the material properties of concrete, cylinders of $\text{Ø}100 \text{ mm} \times 200 \text{ mm}$ (diameter $\times$ length) for compressive and splitting strength, and concrete cubes of 100 mm for compressive strength, were tested. The cylindrical rubberised concrete samples were provided with jubilee clips at the sample ends and then capped at the top surfaces by mortar to avoid local crushing and potential end failure during the compressive strength test. For each batch of concrete, at least three cubes and cylinders were tested at 28 days and at the day of testing. A total of eighty (80) concrete samples were tested, including 44 cylinders and 36 cubes. The cylinder test procedures were undertaken according to EN 12390-2 [32]. The average measured material properties at 28 days are summarised in Table 2. These are the compressive strength $f_{c,28d}$, splitting tensile strength f_{ctr} , axial strain at crushing $\epsilon_{rc1,1}$, lateral strain at crushing $\epsilon_{rc1,2}$ and the elastic modulus E_{rc} .

Table 1 Mix proportions per cubic metre

	R00	R30	R60
Cement (kg)	340	340	340
Fly ash (kg)	85	85	85
Aggregates (kg)			
0–5 mm	820	574	328
5–10 mm	1001	700.7	400.4
Rubber (kg)			
0–4.0 mm	–	74.3	148.5
4.0–10.0 mm	–	16.5	33
10–20.0 mm	–	74.25	148.5
Admixture (l)			
(PL + SPL)	7.6	8.33	7.6
w/c	0.35	0.44	0.35

Table 2 Material properties

Unconfined concrete	$f_{c,28d}$ (MPa)	f_{ctr} (MPa)	ϵ_{rc1} (%)	ϵ_{rc2} (%)	E_{rc} (GPa)	Slump (mm)
R00	54.1	3.97	–	–	35.7	125
R30	21.2	2.21	0.13	0.09	20.1	115
R60	6.4	1.08	0.15	0.10	8.92	50
Steel tube	D/t (mm/mm)	$f_{y0.2\%}$ (MPa)	f_u (Mpa)	ϵ_u (%)	E_s (GPa)	
ST	152/2.8	290	421	0.14	200	

2.2 Steel coupon tests

Circular steel tubes of 152 mm outer diameter and 2.8 mm wall thickness were used in the experimental assessment. To determine the steel material properties, a total of six coupons were extracted from $2 \times \varnothing 150 \text{ mm} \times 300 \text{ mm}$ steel tubes in accordance with EN 10002-1 [33]. Strain values were obtained both through a video extensometer by assigning a gauge length of 230 mm, and from strain gauges that were positioned at mid-length of each sample following its symmetry centreline. The key material properties of steel tubes obtained from the tensile coupon tests are also summarised in Table 2, where E_s is the elastic modulus, f_y is the 0.2% steel proof yield strength, f_u is the ultimate steel strength and ϵ_u is the ultimate strain.

2.3 Stub-column tests

As indicated in Table 3, the sample reference follows the format XXRyyEzz in which XX represents the member type (CF stands for concrete filled tubes and ST stands for steel hollow sections), Ryy is the rubber content in percentage (R00, R30 and R60 for 0%, 30% and 60%, respectively) and Ezz represents the eccentricity level (E00, 0 mm for axial tests). The sample reference ends with A or B, denoting the testing sequence. For example, “CFR30E00A” represents the first rubberised concrete filled steel tube sample, with 30% rubber content in uniaxial compression.

As shown in Fig. 1a, short stub-columns of $\varnothing 150 \text{ mm}$ and 300 mm length were tested under axial loading in a 3500-kN capacity Instron machine. These included nine

Fig. 1 Testing arrangement: **a** axial tests, **b** eccentric tests, **c** bending tests

concrete cylinders and six circular concrete filled steel tubes (CFST), using R0, R30 and R60 concrete, as well as two circular hollow steel (CHS) tubes. As shown in the figure, to ensure loading concentricity and reliable boundary conditions, two thick rigid loading plates with dimensions of $200 \text{ mm} \times 200 \text{ mm}$ were used at the top and bottom of each sample. Another $25 \text{ mm} \times 300 \text{ mm} \times 300 \text{ mm}$ plate was placed concentrically between the rigid plates and a 3D hinge. A displacement rate of 0.25 mm/min was used and was increased to 1.0 mm/min in the post-peak range up to failure. The same loading procedure was applied for the eccentric and bending tests described below.

Table 3 Results of axial stub-column tests

Type of test (–)	Type of specimen (–)	Specimen ID	Size ($D \times L$)/ t (mm $\times$ mm/mm)	ρ_{vr}	e_{test} (mm)	N_{test} or P_{test} (kN)
Axial	C	R00E00 ^a	150 $\times$ 300/2.8	0.0	0	903
	C	R30E00 ^a	150 $\times$ 300/2.8	0.3	0	375
	C	R60E00 ^a	150 $\times$ 300/2.8	0.6	0	113
	ST	ST-E00A	152 $\times$ 300/2.8	–	0	424
	ST	ST-E00B	152 $\times$ 300/2.8	–	0	428
	CF	CFR00E00A	152 $\times$ 300/2.8	0.0	0	1688
	CF	CFR00E00B	152 $\times$ 300/2.8	0.0	0	1637
	CF	CFR30E00A	152 $\times$ 300/2.8	0.3	0	941
	CF	CFR30E00B	152 $\times$ 300/2.8	0.3	0	944
	CF	CFR60E00A	152 $\times$ 300/2.8	0.6	0	618
	CF	CFR60E00B	152 $\times$ 300/2.8	0.6	0	628

The notations for the type of specimens refer to: C—concrete cylinder, ST—circular hollow steel tube, CF—circular concrete-filled steel tube. P_{test} corresponds to the peak axial load N_{test} for concentric compression tests

^a Average values of three tests

2.4 Eccentric and bending tests

Using the same notation described above, the specimen reference follows the format $XXR_{yy}E_{zz}$. In the eccentric and bending tests, the eccentricity levels E_{zz} are as follows: E10, E25 and E50 for 10 mm, 25 mm and 50 mm, respectively, and E ∞ refers to the case of bending only. For example, “CFR30E10” represents a rubberised concrete filled steel tube, with 30% rubber content, subjected to eccentric compression with 10 mm eccentricity.

With reference to Fig. 1b, an additional $9 \times$ CFST specimens of $\varnothing 150$ mm diameter and 600 mm length were tested. For each eccentricity level, specimens with R0, R30 and R60 concrete were tested in a 2000-kN capacity Instron-T200 servo-controlled hydraulic testing machine. Two steel rings of 50 mm depth were used to fix the ends of the specimen and acted as clamping devices to the machine plates which ensured pinned–pinned boundary conditions through a set of knife edges. After the specimens were positioned in the testing frame, their position was adjusted to ensure the application of the required eccentricity level. Testing was undertaken in displacement control with the displacement applied incrementally, using the same rates noted above, until the maximum rotation of the end plates was reached.

Another four specimens, including $3 \times$ CFST of $\varnothing 150$ mm diameter and 600 mm length as well as one circular hollow section (CHS) specimens, were tested under three-point bending in a 750-kN Mayes testing machine, as shown in Fig. 1c. The clear span between the supports was 500 mm, whilst the load was applied at the specimen mid-length through a loading ring. Half-ring supports were used at the ends. The same displacement control of 0.25 mm/min was used as before, increasing to 1.0 mm/min at deformations above 20 mm. The tests continued until the support rotation limit was reached. After each test, the outer tube was removed and the failure pattern of the concrete core was examined.

2.5 Instrumentation

For the axial compression tests, the 25-mm plate provided a reference to the three displacement transducers (LDVTs) that were placed around the cylindrical samples to record their axial shortening. The lateral displacement of the CFST specimens under eccentric loading was measured by three displacement transducers (LDVTs), and the axial shortening was obtained from the testing machine. Moreover, the end rotations at the loading and base plates were obtained from two inclinometers as shown in the figure. For the bending tests, in-plane displacements were measured at three locations using three transducers (LVDTs), and for comparison purposes, strain gauges were also placed at mid-span.

Besides the direct load, displacement and rotation measurements, a digital image correlation (DIC) system was used to record the detailed deformations and strains. For this, the specimen surface was provided with a fine speckle pattern. In the first stage, a black matte layer was applied to avoid the reflections and then a white speckle pattern was sprayed to obtain randomly distributed dots of 0.5–2.0 mm diameter (i.e. white polka dots on black background). This is necessary to ensure that the DIC system can track the movement of a specimen surface under applied load. Two DIC cameras with 35 mmf/2D lenses were used with a focal distance of about 2.0 m during the test, as illustrated in Fig. 2a. Prior to testing, a calibration procedure was undertaken. A recording rate of 0.2 Hz was used as it represents a good balance between the number of images and characteristic deformation response.

The processed DIC recordings were transformed into strains using assigned virtual gauges that tracked the specimen surface during the tests. To obtain deformations and strains, the programme processes the deformation of the surface within discretised subsets or facet elements of the recorded image. The maximum correlation in each subset corresponds to the displacement, giving vector lengths and directions. These are then differentiated to obtain strain tensors across the specimen surface [2, 34]. As shown in Fig. 2b, for the short axial specimens, 100-mm vertical and 50-mm horizontal gauges were considered in order to describe the axial/longitudinal (ϵ_1) and lateral (ϵ_2) specimen response, respectively. For the bending and eccentric compression tests, 100-mm virtual gauges were placed parallel to the longitudinal specimen axis at the extreme fibres (Fig. 2c, d).

3 Test results

3.1 Stub-column tests

3.1.1 Load shortening response

Axial compression tests on short stub-columns were carried out to examine the influence of rubber content (ρ_{vr}) on the confinement levels developed within the infilled concrete material and, in turn, on the overall strength of the cross-section for the case of pure axial load. For this purpose, it should be noted that these tests were carried out on relatively short elements, with L/D of 2.0. These are therefore considered as stub-column tests, which provide useful information on the interaction between the confined infill concrete and the confining steel tube on the cross-sectional level. However, reliable assessment of axially loaded CFST members, for practical application, would necessitate further dedicated examination and validation for more practical ranges with

Fig. 2 **a** Set-up of digital image correlation (DIC) system; **b** virtual gauge locations for: **b** axial tests, **c** eccentric tests, **d** bending tests

higher L/D ratios, as the latter can have a notable influence on behaviour.

The load-carrying capacity of the samples in compression was found to be a function of the ρ_{vr} and confinement. This is highlighted through the axial load versus shortening ($N-u$) relationships shown in Fig. 3 and listed in Table 3. The maximum axial force obtained from the tests (N_{test}) is referred to as the experimental axial capacity. The N_{test} corresponds to the peak load for samples that developed post-peak softening response, whilst N_{test} was chosen at an axial shortening of 2% for cases exhibiting post-peak hardening. The average N_{test} of CFR00E00, CFR30E00 and CFR60E00 were 1663 kN, 943 kN and 623 kN, respectively.

As shown in Fig. 3a, the $N-u$ curve of the conventional CFR00E00 sample is characterised by initial post-peak softening, whilst for CFR30E00 with 30% rubber the $N-u$ exhibits post-peak hardening with a small region of softening just after the peak. In contrast, the response of CFR60E00 samples with 60% rubber exhibited a smooth transition from the elastic to inelastic regimes and gradual post-peak hardening. For example, at a displacement of $u_1 = 25$ mm, corresponding to about 8% axial shortening (Fig. 3a), the reduction in strength for CFR00E00 element was around 25%. In contrast, at the same level of axial shortening, CFR30E00 had a similar strength as at peak, whilst CFR30E00 showed an increase in strength of more than 30% above first yield corresponding to deviation from elastic behaviour.

The gradual shift in behaviour from post-peak softening to hardening indicates different interaction properties

Fig. 3 Stub-column tests **a** load-axial shortening curves, **b** deformed shape of CFR30E00B at key stages (I) yield, (II) visible onset of local buckling, and (III) 10% axial shortening; **c** failure of the stub-column samples

between the concrete infill and the steel tube. On the one hand, as the concrete strength decreases with increased rubber content (e.g. from R00 to R60), the confining pressure required to enhance the post-peak response is reduced. For relatively high-strength CCM, similar to that used herein, minor improvements can occur in the post-peak region, whilst for relatively low-strength concrete such as RuC, only a small amount of confinement pressure is needed to enhance the response.

The infilled samples behaved in a relatively ductile manner under all loading conditions. This is shown in Fig. 3b through the load–displacement (N – δ) curves and surface deformation characteristics at key displacement levels for selected samples. The characteristic displacement levels denoted with (I), (II) and (III) correspond to yield, visible onset of local buckling and termination of the test, respectively. An assessment of the critical strain ϵ_{cr} , using a widely used model [35] and representing initiation of local buckling of a hollow circular section [36] with the geometrical and material properties in Table 2, showed that the critical displacement δ_{cr} was between stages (I) and (II) for all tests. This indicates that local buckling always occurred after initiation of yielding (i.e. after stage (I) in Fig. 3b).

3.1.2 Overall behaviour

As shown in Fig. 3b, c, for short stub-columns, radial expansion and outward folding failure modes were observed. This is a typical response of short stub-columns in which for relatively thin tubes, local buckling can occur due to end effects [24]. After localised buckling occurred at the CFR30E00B sample top as depicted in Fig. 3b, local folds and axial shortening increased quickly. The ultimate behaviour depended on the rubber content (ρ_{vr}), as discussed in Sect. 3.4, being observed with the increase in ρ_{vr} . After unloading and after removing the load transfer plates were removed, the rubberised concrete sprang back indicating a high amount of elastic energy accumulated within the rubberised concrete. As shown in Fig. 3c, this effect increased with the amount of rubber.

It should be noted that the failure mode illustrated in Fig. 3c has the purpose to only show the spring-back response of rubberised concrete in comparison with normal concrete. The photographs were taken after removal from the test rig at 50% compression (i.e. compressive displacement of 150 mm corresponding to half of the sample length). Deformations above 3–5% would not be achievable in practical application within building structures due to imposed rotation limits of connecting members. An axial shortening of about 5% corresponds to 15-mm axial deformation of the investigated stub members. This point corresponds to Notation E00-I in Fig. 3b.

3.1.3 Confinement effects

As described in Sect. 2.2, a DIC system was employed to record surface deformations, from which strains (ϵ) can be obtained from assigned virtual gauges. For the axial compression cases, the stresses (σ) can be readily determined and used to obtain the stress–strain (σ – ϵ) curves for each test and then averaged for each set. The σ – ϵ relationships under axial compression for the unconfined concrete and CFST members are shown in Fig. 4 in order to evaluate the relative confinement effect provided by the tubular steel wall.

The crushing of the R00 high-strength concrete infill was brittle and relatively sudden, and the confinement was activated instantaneously (CFR00E00) as shown in Fig. 4a. In contrast, R30 and R60 concrete materials were relatively ductile with soft crushing behaviour and a flatter post-peak σ – ϵ response as shown in Fig. 4b, c (CFR30E00, CFR60E00). It is shown that rubberised concrete exhibits a more gradual crushing response, and there is significant elastic energy accumulated within the rubber granules; hence, the interaction with the steel tube is also progressive. The ultimate state was characterised by radial expansion and outward folding of the external tube. As discussed below, the presence of rubber in concrete enhanced the confinement action leading to beneficial effects in terms of both strength and deformation capacity. Based on the measured strains, it was observed that the external confinement enhanced the cross-section capacity of both conventional and rubberised CFST samples under compression in comparison with the corresponding unconfined conventional concrete or rubberised concrete cylinders. As indicated in Table 3 and Fig. 4, the confined-to-unconfined (CFRxxE00/RxxE00) strength ratios were 1.84, 2.51 and 5.51 for samples incorporating R00, R30 and R60 concrete, respectively.

For the conventional CFST samples (CFR00E00), the axial strain at peak ($\epsilon_{cc,1}$) was higher by about 40% compared to the axial strain at crushing ($\epsilon_{c0,1}$) of the corresponding unconfined CCM element (R00E00) (Fig. 4a). As shown in Fig. 4b, the average axial strain at peak ($\epsilon_{rc,1}$) of CFR30 was about 20% higher than the unconfined crushing strain ($\epsilon_{rc,1}$) of its RuC non-filled counterpart (R30E00). Enhancement of more than 50 times in strain at peak ($\epsilon_{rc,1}$), which, due to strain-hardening response, corresponds to ultimate ($\epsilon_{rc,u}$), was obtained for CFR60 compared to $\epsilon_{rc,1}$ of an unconfined RuC sample (R60E00) (see Fig. 4c). As shown in Fig. 4, the lateral strains were also influenced by the presence of rubber and confinement, with improved energy dissipation levels occurring for relatively high ρ_{vr} . The above observations suggest that confinement effectiveness increases significantly with the increase in ρ_{vr} ; hence, a highly beneficial effect in terms of general behaviour is obtained for RuCFST in comparison with CFST specimens.

Fig. 4 Comparative stress–strain curves for CFST and unconfined specimens: **a** R00, **b** R30, **c** R60

3.2 Eccentric and bending tests

3.2.1 Load–rotation response

In addition to the axial stub–column tests described above, eccentric and bending tests enable an evaluation of the axial–bending (N – M) interaction curves for CFST

cross-sections with CCM and RuC. Figure 5 depicts the applied load (N) versus deflection angle (θ) relationships for eccentrically loaded specimens and those subjected to bending, whilst a summary of the main test results is given in Table 4. Despite having different aspect ratios than eccentric and bending specimens and for comparative purposes only, the axial capacity of the corresponding short specimens

Fig. 5 Axial load–rotation (N – θ) curves for tested specimens under axial–bending loading conditions for: **a** CFR00Ezz, **b** CFR30Ezz, **c** CFR60Ezz

Table 4 Results of eccentric and bending tests

Type of test (–)	Type of specimen (–)	Specimen ID	Size ($D \times L$)/ t (mm $\times$ mm/mm)	ρ_{vr}	e_{test} (mm)	N_{test} or P_{test} (kN)	M_{test} (kNm)	$\theta_{peak}/\theta_{max}$ (mrad)
Eccentric	CF	CFR00-E10	152 $\times$ 600/2.8	0.0	12	1330	15.9	14/45
	CF	CFR00-E25	152 $\times$ 600/2.8	0.0	24	986	23.7	19/58
	CF	CFR00-E50	152 $\times$ 600/2.8	0.0	53.5	592	31.7	23/73
	CF	CFR30-E10	152 $\times$ 600/2.8	0.3	12	859	10.3	7/46
	CF	CFR30-E25	152 $\times$ 600/2.8	0.3	23	679	15.6	17/67
	CF	CFR30-E50	152 $\times$ 600/2.8	0.3	50.5	448	22.3	25/115
	CF	CFR60-E10	152 $\times$ 600/2.8	0.6	12	525	6.3	34/67
	CF	CFR60-E25	152 $\times$ 600/2.8	0.6	25	415	10.3	37/112
	CF	CFR60-E50	152 $\times$ 600/2.8	0.6	53.5	307	16.5	35/115
Bending	ST	ST-E ∞	152 $\times$ 600/2.8	–	–	140	15.5	> 110
	CF	CFR00-E ∞	152 $\times$ 600/2.8	0.0	–	217	24.9	> 115
	CF	CFR30-E ∞	152 $\times$ 600/2.8	0.3	–	219	24.7	> 115
	CF	CFR60-E ∞	152 $\times$ 600/2.8	0.6	–	187	21.0	> 115

The notations for the type of specimens refer to: C—concrete cylinder, ST—circular hollow steel tube, CF—circular concrete-filled steel tube. P_{test} corresponds to the peak axial load N_{test} for eccentric compression tests, whilst P_{test} is the peak load applied at mid-span in three-point bending tests; symbol “>” indicates that θ_{max} was not achieved and the test was halted due to test rig constraints; $\theta_{peak} = \theta_{max}$ in bending tests due to bi-linear hardening response

described above is also shown in the figure. The chord rotation θ was assessed from the central deflection δ of the member at its half-length and considering the position of end supports. The value of θ_{peak} in Table 4 corresponds to the test peak strength N_{test} , whilst the maximum rotation θ_{max} corresponds to a 15% reduction in capacity for the eccentric tests or when rig support limits are reached in the bending tests.

The eccentric compression tests focussed on nominal eccentricities up to $e = 50$ mm, which were feasible within the experimental constraints. The test curves are characterised by a post-peak softening response, as shown in Fig. 5a–c, after the peak load was reached. In contrast, specimens subjected to bending only had a largely bi-linear response with hardening. A gradual reduction in peak load N_{test} is observed with an increase in bending moment as a function of the applied eccentricity (see Table 4). In contrast, θ_{max} gradually increased with the eccentricity. Additionally, for the same level of eccentricity, a higher θ_{max} was obtained with an increase in ρ_{vr} . For example, for a nominal $e = 25$ mm, the specimen with 60% rubber (CFR60E25) had a higher θ_{max} by a factor of 1.93 compared to the specimen without rubber (CFR00E25), whilst for CFR30E25 (with 30% rubber), the increase was about 15%. On the other hand, the specimen in bending (CFRyyE ∞) reached θ_{max} over 115 mrad and was limited by the testing arrangement.

3.2.2 Overall behaviour

As indicated from Figs. 6 and 7, for specimens subjected to eccentric loading, a behaviour governed by crushing of the concrete infill and buckling of the steel tube in compression

occurred. Failure of these specimens was by outward folding failure modes of the compression face around their mid-height but no fracture occurred in tension. This is shown in Fig. 6b for Specimen CFR30E25 at various stages of deformation and in Fig. 7a after the specimens were removed from the rig. A similar folding failure mechanism without tensile fracture was observed for all members (filled tubes and hollow tubes) subjected to three-point bending. As illustrated in Fig. 6c for Specimen CFR30E ∞ , local buckling was observed around the loading ring on the compressive side of the specimens. Moreover, flexural cracks in the concrete were clearly visible on the tension side at mid-span for concrete-filled specimens after half the steel tube was removed (Fig. 7b).

In the eccentric tests, an increase in replacement ratio indicated a larger compression zone promoted by a gradual reduction in stress after the peak crushing strain was reached. This is compensated by a shorted tension zone, yet with more and finer cracks. The latter occur due to the lower tensile strength of rubberised concrete in comparison with the conventional concrete. This behaviour is described below in Sect. 3.4 through a detailed sectional analysis of eccentric tests. These comments agree with other studies in the literature, which showed that reinforced rubberised concrete members would develop finer cracks at a shorter spacing [37] and in CFST tests with rubberised concrete up to 30% replacement ratios [17].

As noted in the introduction, this paper examines the interaction between confined rubberised concrete and the confining steel tube for the purpose of assessing the cross-section performance. Although microstructural

Fig. 6 **a** Comparative applied load–displacement curves for selected CFR30Ezz specimens and key response characteristics at (I) yield, (II) visible onset of local buckling, and (III) termination of the test; deformed shape of specimens: **b** CFR30E25, **c** CFR30E ∞

Fig. 7 Failure modes of tested CFST specimens for: **a** axial tests, **b** eccentric tests, **c** bending tests

investigations would provide some information regarding the interface transmission zone, this is beyond the scope of the current study. Microstructural investigations have already been carried out on similar RuC mixes in a wider research project [3]. It was shown that rubber and mineral aggregates were well distributed within the investigated surface, yet large gaps were evident between the rubber particles and the surrounding cement paste. Such gaps occur due to the lack of bonding and limited cement hydration at the rubber–cement paste interface zone, or by rubber detachment during specimen preparation, as also shown in other studies [38].

3.2.3 Confinement effects

In addition to the confined-to-unconfined strength ratios for which the strength of the unconfined concrete was used as a benchmark, the performance of CFST specimens subjected to both concentric and eccentric compression is assessed as

a reference to the strength of the hollow steel section. The concrete contribution ratio (λ_{cc}) is evaluated as the ratio of the test ultimate capacity of infilled specimens to the cross-sectional theoretical capacity of the hollow steel section subjected to concentric or eccentric loading ($N_{CFST, test}/N_{ST}$) [39]. It should be noted that N_{ST} can be determined using codified guidance and depends on the yield strength f_y , cross-sectional area A_s and plastic section modulus W_{pl} of the steel tube, as well as on the eccentricity level (Eq. 1a) [40].

The λ_{cc} parameter indicates the contribution of concrete infill to the capacity as well as the synergy between the steel tube and concrete core. Generally, the ultimate capacity and post-peak behaviour are related to the confinement level, and a higher confinement factor would improve ductility [41]. The concrete contribution ratio (λ_{cc}) is plotted against the confinement factor (ξ) for each specimen in Fig. 8. The parameter ξ in Eq. (1b) is the ratio between the product of yield strength and the cross-sectional area of the steel tube

Fig. 8 Relationship between λ_{cc} and ξ

$(f_y \times A_s)$, and the product of concrete compressive strength and the cross-sectional area of the concrete core ($f_c \times A_c$).

$$N_{ST} = \frac{f_y}{\left(\frac{1}{A_s} + \frac{e}{w_{pl}}\right)} \quad (1a)$$

$$\xi = \frac{f_y A_s}{f_c A_c} \quad (1b)$$

As shown in Fig. 8, λ_{cc} decreases with ρ_{vr} , whilst ξ increases with the rubber content. As the confinement effects occur due to the lateral expansion of the concrete core at ultimate [42] and the RuC exhibits larger lateral dilation than CCM, an enhanced composite action is obtained for RuCFST in comparison with CFST. This effect is captured by the trend in Fig. 8. For an axially loaded circular CFST in compression, the confinement is uniform around the circumference of the tube. In contrast, in specimens subjected to bending only this is non-uniform since the strain varies across the section and is limited to the area of concrete in compression [43]. Unsurprisingly, for R00 CCM the highest λ_{cc} value in Fig. 8 is for axial loading (CFR00E0) and decreases with eccentricity. However, as ρ_{vr} increases, λ_{cc} becomes relatively constant for R30 and increases for R60 proportionally with the eccentricity level. This effect occurs due to the relatively soft crushing behaviour of RuC compared to CCM, which is manifested by a larger area in compression, as discussed further below.

Figure 9 shows selected strain profiles (ϵ) across the depth for characteristic bending moment levels. As expected, the neutral axis (n.a.), marked with dashed horizontal lines, moves downwards with a decrease in eccentricity, indicating an extension of the compression region. On the other hand, a comparative assessment of neutral axis position as a function of ρ_{vr} shows a proportional reduction in the tension zone with an increase in ρ_{vr} . For the same level of

Fig. 9 Representative strain profiles for specimens: **a** CFRyyE50, **b** CFRyyE25, CFRyyE10

eccentricity (e.g. 50 mm), the tension zone of the RuCFST specimen with 60% rubber (CFR60E50) settles at about 31% of the member depth, whilst for the conventional CFST (CFR00E50) this is around 52% of its section diameter. For comparative purposes only, as shown in Fig. 10, for the members tested bending only, the inelastic stage was characterised by hardening. For consistency, the moment capacity of these members is considered as test bending moment at a longitudinal flexural strain of 1.0% [44].

As noted above, for relatively large replacement ratios (CFR60E0), there is a significant decrease in compressive

Fig. 10 Moment (M) versus strain (ϵ) at mid-span at the extreme tensile fibre

Fig. 11 Schematic representation of simplified and complete N – M diagrams

nominal strength compared to the conventional case (CFR00E0). The reduction from R00 to R30 is by about 43% and from R00 to R60 is by about 62%. However, in normalised terms, the confined-to-unconfined strength ratios are 1.84, 2.51 and 5.51 for specimens incorporating R00, R30 and R60 concrete, respectively. The optimum point between all these parameters could possibly be somewhere between R00 and R30 and would require dedicated assessment for specific design situations for which the relative importance of these parameters may differ. To better understand the influence of all these parameters, nonlinear numerical simulations focusing on parameters outside of the tested ranges are currently underway.

4 Cross-section strength interaction

Axial–bending (N – M) strength interaction diagrams are typically adopted to characterise the cross-sectional strength under combined loading. A normalised representation of the N – M diagram can be achieved as illustrated in Fig. 11. The

grey curve represents the actual interaction under combined loading, whilst the black curve is a simplified version. The latter is characterised by three regions that are bounded by four characteristic pairs $(M_i/M_{pl,R}, N_i/N_{pl,R})$. Points A, B, C and D represent the maximum axial capacity, the nominal flexural strength, an intermediate case with both axial and bending, and the balance point corresponding to the maximum moment, respectively.

In this section, the test results presented in Sect. 3 are compared with predictions obtained from the application of existing guidelines and a series of nonlinear sectional analyses employing models with fibre elements [45]. Sensitivity studies, focusing on a wide variation of geometrical and material configurations outside of the tested ranges, are also conducted in order to support the development of unified N – M interactions including for RuCFST with relatively high rubber content. In addition to the results from axial stub-column tests with relatively low aspect ratios, a database with axial compression test results with L/D in the range of three and above was collated [25–29] and used for validation of the unified N – M diagram.

4.1 Comparative assessments

To evaluate the axial–bending (N – M) interaction diagrams for CFST and RuCFST cross-sections, the axial and moment capacities are obtained from existing provisions as well as from sectional analysis considering the material and geometrical properties from Sect. 2.1 [21, 40, 45–47]. It should be noted that for codified assessment based on Eurocode 4 [21], the compressive design resistance of the CFST section is only valid for compressive concrete strength within a specified range ($20 \text{ MPa} \leq f_c \leq 60 \text{ MPa}$) [21]. As shown in Sect. 2.1, R60 concrete is outside this range, whilst R30 concrete is just above the lower bound. Notwithstanding this, an assessment is made herein for all cases, although the specimens with high ρ_{vr} are not explicitly covered by the code.

The axial capacity of a composite cross-section ($N_{pl,R}$) is a function of coefficients representing the confinement effect for steel and concrete (η_s and η_c), respectively, yield strength of the steel tube f_y , its cross-sectional area A_s , thickness t and diameter D , as well as the cross-sectional area of the concrete core A_c and concrete core strength f_c (Eq. 2a). The term in brackets at the right-hand side of Eq. (2a) is expected to capture the effects of confinement on the axial capacity of the composite section. According to Eurocode 4 [21], both η_s and η_c are a function of the tube slenderness. For a perfectly plastic behaviour and an eccentricity-to-diameter ratio $e/D=0$, the two factors are $\eta_s=0.75$ and $\eta_c=4.90$. Reduced steel contribution and concrete enhancement occur due to the development of the hoop stress and confinement effect of the concrete, respectively [48]. For eccentricities below 10% of the diameter, η_s and η_c are reduced proportionally

by a factor depending on e/D ratio. Outside these ranges, for $e/D > 0.1$, the factors $\eta_s = 1.0$ and $\eta_c = 0$ imply no contribution from confinement. It is also worth pointing out that the circular hollow sections of the tested specimens are class 2 in bending and/or compression in accordance with Eurocode 3 classification [40].

At the other end of the N – M interaction, the moment capacity of normal CFST composite section under bending only ($M_{pl,R}$) can be determined from Eq. (2b) [49]. The main parameters required for $M_{pl,R}$ are W_{ps} , W_{psn} , f_y , W_{pc} , W_{pcn} , f_c , which represent the steel plastic section modulus, steel plastic section modulus from $2 \times h_n$, yield strength of steel tube, concrete plastic section modulus, concrete plastic section modulus from $2 \times h_n$, and strength of concrete core, respectively [21, 49]. The parameter h_n is defined as the distance between the neutral axis and the cross-section centreline.

$$N_{pl,R} = \eta_s f_y A_s + \left(1 + \eta_c \frac{t}{D} \frac{f_y}{f_c}\right) f_c A_c \quad (2a)$$

$$M_{pl,R} = (W_{ps} - W_{psn}) f_y + 0.5 (W_{pc} - W_{pcn}) f_c \quad (2b)$$

For comparison purposes, only equations to assess the N – M interaction using AISC360 [46] and GB 50936 [47] provisions are shown below. A limitation of the North American code is that it does not cover high-strength materials but allows for the design of slender sections. It is worth noting that the tubular section used for the tests in this paper is compact considering this guideline. Equation (2c) is used to assess the axial capacity ($N_{u,AISC}$) of a CFST configuration, in which C_2 is a parameter depending on the section type ($C_2 = 0.95$ for circular CFST sections). To assess the complete N – M interaction curve, the code permits the use of two methods. The first method is based on a bi-linear interaction model as for steel sections, whilst the second method is based on a four-point envelope such as that illustrated in Fig. 11 [46, 50]. For compact sections, the moment capacity $M_{u,AISC}$ can be assessed using a plastic stress distribution (Eq. 2b) [46, 49]. For consistency with the Eurocode 4 provisions, the second AISC360 method is used to assess the strength interaction properties.

The GB 50936 design model considers the confinement effects in circular CFST cross-sections when assessing axial load capacity ($N_{u,GB}$) (Eqs. 2e–2g) [51]. The axial capacity is the product of the cross-sectional area A_{sc} of the CFST member and the compressive design strength of the confining concrete f_{sc} in CFST column [50] (Eq. 2f). The latter factor f_{sc} is a function of the confinement factor ξ (Eq. 2g). As pointed out in available literature, the concrete core is under biaxial confinement due to the steel tube having an enhanced compressive strength [52]. The moment capacity ($M_{u,GB}$) considers an empirical coefficient, dependent on ξ , the elastic section modulus $W_{sc} = \pi D^3/32$, and the

compressive strength f_{sc} (Eq. 2h). Note that Eq. (2h) is a condensed form of the unified GB 50936 models for CFST and circular concrete-filled double-steel tubular (CFDST) in which $\chi = 1.0$ [53]. Equations (2e) and (2h) are then used to construct interaction diagrams using an established model [54].

$$N_{u,AISC} = A_s f_y + C_2 A_c f_c \quad (2c)$$

$$M_{u,AISC} = M_{pl,R} \quad (2d)$$

$$N_{u,GB} = A_{sc} f_{sc} \quad (2e)$$

$$f_{sc} = \left[1.212 + \left(\frac{0.176}{f_y} + 0.974 \right) \xi + \left(0.031 - \frac{0.104 f_c}{14.4} \right) \xi^2 \right] f_c \quad (2f)$$

$$\xi = (A_s f_y) / (A_c f_c) \quad (2g)$$

$$M_{u,GB} = (0.963 \sqrt{\xi} - 0.214 \xi) W_{sc} f_{sc} \quad (2h)$$

The (M_{test} , N_{test}) pairs of the tested CFST and RuCFST specimens, and the N – M interaction diagram obtained using Eurocode 4 provisions for which $N_{pl,R}$ and $M_{pl,R}$ were determined using Eqs. (2a, 2b), are shown in Fig. 12. The comparisons between the tests and code results indicate generally good agreement for the CFST specimens. However, the estimated compression capacities $N_{pl,R}$ show unconservative values for RuCFST, below the ultimate capacity measured in the tests N_{test} , noting that the concrete strength of R30 and R60 is below the scope imposed in the code. It is worth noting that Eurocode 4 provides limited information on the design of circular CSFT beam columns incorporating low-strength concrete. On the other hand, assessments of $M_{pl,R}$

Fig. 12 Comparative assessment between test results and codified procedures [21, 46, 47]

indicate relatively conservative predictions of the flexural capacity for circular RuCFST members in bending only. The estimated interaction curves using AISC360 and GB 50936 provisions show conservative values. In all cases, the predicted curves are below the test range and Eurocode 4 estimates. It can be noted that the level of conservatism increases with rubber content, which is largely attributed to the relatively low concrete strengths, which typically would lie outside of the test databases used for validating the design models. As indicated in Fig. 12, Eurocode 4 largely provides more reliable estimates, and it is later modified to better predict the response of RuCFST.

4.2 Modified axial and bending capacities

As mentioned above, the axial load-carrying capacity is affected by confinement effects, and as shown in Fig. 8, there is a significant influence from ρ_{vr} on the confinement enhancement level. The results from the comparative evaluations in the previous section indicate that $N_{pl,R}$ is over-predicted compared to the test axial capacity N_{test} . To achieve more reliable estimates, a regression analysis is carried out and a ρ_{vr} -dependent confinement effectiveness factor (λ_{rcc}) is proposed (Eq. 3a). The λ_{rcc} factor is used to modify the concrete contribution (i.e. term in brackets at the right-hand side of Eq. 2a). The modification can be observed in Eq. (3b). However, as noted before, it should be recalled that the axial compression tests in this study were carried out on relatively short specimens, with L/D of 2.0. These are therefore considered as stub-column tests, which provide useful information on the interaction between the confined infill concrete and the confining steel tube on the cross-section level. However, reliable assessment of axially loaded CFST members, for practical application, would necessitate further dedicated examination and validation for more practical ranges with higher L/D ratios, as the latter can have a notable influence on behaviour.

On the other hand, the assessment of the neutral axis position for specimens subjected to bending and axial force (see

Fig. 9) shows that as ρ_{vr} increases, there is a proportional reduction in the tension zone length. This effect is more visible for bending-only cases and can be directly correlated to estimates of $M_{pl,R}$ using existing codified provisions, which become less conservative with the increase in ρ_{vr} . In order to obtain more reliable estimates of moment capacity of RuCFST under bending only, a ρ_{vr} -dependent factor (χ_{rcc}) is considered for modifying Eq. (2b) that considers a plastic stress distribution within the composite section. A linear relationship between ρ_{vr} and χ_{rcc} is used as depicted in Eq. (4a). The χ_{rcc} parameter is used to modify the codified equation for $M_{pl,R}$ as shown in Eq. (4b).

$$\lambda_{rcc} = (1 - 0.40\rho_{vr}) \quad (3a)$$

$$N_{pl,R} = \eta_s f_y A_s + \lambda_{rcc} \left(1 + \eta_c \frac{t}{D} \frac{f_y}{f_c} \right) f_c A_c \quad (3b)$$

$$\chi_{rcc} = 1 - 0.05\rho_{vr} \quad (4a)$$

$$M_{pl,R} = \chi_{rcc} [(W_{ps} - W_{psn})f_y + 0.5(W_{pc} - W_{pcn})f_c] \quad (4b)$$

The resulting predicted-to-test strength ratios ($N_{test}/N_{pl,R}$) with/without the modifying factor (λ_{rcc}) are plotted against the volumetric rubber ratio (ρ_{vr}) in Fig. 13a. Overall, applying the proposed λ_{rcc} factor to existing code formulations (i.e. Eqs. 4a, 4b with $\lambda_{rcc} = 1.0$) provides improved capacity predictions under axial compression. This is shown by the dashed grey line in Fig. 13a that is parallel to $N_{test}/N_{pl,R} = 1.0$ in which the steel tube contribution was considered as defined by Eurocode 4 ($\eta_s = 0.75$). Note that by considering a full contribution of the steel tube to $N_{pl,Rd}$ (i.e. $\eta_s = 1.00$), the predictions are closest to the $N_{test}/N_{pl,R} = 1.0$ line. Improved predictions are obtained when Eq. (4b), that accounts for the χ_{rcc} parameter, is used to assess $M_{pl,R}$. This is shown in Fig. 13b in which the estimated $M_{pl,R}$ values with and without χ_{rcc} are presented. As in the case of λ_{rcc} -modified $N_{pl,R}$, the χ_{rcc} -dependent $M_{pl,R}$ values in Fig. 13b are more

Fig. 13 Predicted to test strength ratios for: **a** λ_{rcc} , **b** χ_{rcc}

consistently arranged with respect to the $M_{\text{test}}/M_{\text{pl,R}} = 1.0$ line for all ranges of ρ_{vr} .

4.3 Nonlinear sectional response

As mentioned before, besides comparison with codified provisions, nonlinear sectional analyses employing models with fibre elements [45] were conducted to assess the N – M interaction curves for tested configurations as well as to undertake sensitivity studies. In these analyses, the cross-section was discretised into fibre elements, and perfect bond was assumed. The circular cross-section of the concrete infill was discretised into ten fibre elements. In order to represent the steel part, closely spaced discrete rebars, having the same total cross-sectional area as the tube, were considered. An initial assessment indicated that the sectional analysis does not capture implicitly confinement effects, offering overly conservative N – M interaction curves, especially in the cases with low bending moments. To overcome this limitation, confinement effects were explicitly incorporated for CFST sections by using the Eurocode 4 confinement factor denoted here as κ_{cc} , as an input in the programme [45]. The parameter κ_{cc} is the term in brackets at the right-hand side of Eq. (2b) (i.e. $\kappa_{\text{cc}} = [1 + \eta_c(t/D)(f_y/f_c)]$). In the sectional assessments, the input stress–strain relationships and the actual concrete contribution were taken into account by adopting the ρ_{vr} -dependent confinement coefficient λ_{rcc} (i.e. $\kappa_{\text{cc}} = \lambda_{\text{rcc}} \times [1 + \eta_c(t/D)(f_y/f_c)]$).

To validate the above approach, besides the tests from this paper, a total of 40 circular CFST tests in axial compression of compact sections were collated in a database [25–29], and the tests results were compared to those from the analysis. The resulting test-to-sectional analysis-predicted strength ratios ($N_{\text{test}}/N_{\text{res}}$) with and without the adopted confinement factor are shown in Fig. 14. These results reveal that by adopting a confinement factor κ_{cc} in the sectional analysis programme, much closer agreement

Fig. 14 Comparison between test results and nonlinear section analysis for specimens in compression

is obtained. Accordingly, these assumptions were used to undertake sensitivity studies to obtain a detailed insight into the N – M interaction diagram for a wide range of structural parameters.

An orthogonal array method was adopted in the parametric assessment, with three array factors of ρ_{vr} , D/t and f_{cr}/f_y for 18 CFST and RuCFST specimens. The ranges of parameters investigated were $\rho_{\text{vr}} = 0, 0.3, 0.6$ and D/t ratios ranging from 29.5 to 71.4 (in D increments of 12 mm from 152 to 200 mm; t increments of 0.4 mm from 2.8 mm to 8 mm). The f_{cr}/f_y ratio was limited to material properties obtained from tests. Note that f_c denotes the compressive strengths of both conventional and rubberised concretes. A total of 192 models were generated using the sectional analysis procedure and considering the assumptions described above. The main objective was to examine the effect of D/t , ρ_{vr} and f_{cr}/f_y on the N – M interaction of RuCFST sections.

4.4 Simplified axial–bending relationships

As mentioned above, axial–bending (N – M) interaction diagrams for RuCFST cross-sections can be developed using sectional analysis procedures or alternatively, for practical design procedures, through more simplified methods. The latter can be represented by three regions that are bounded by four characteristic pairs ($M_i/M_{\text{pl,R}}$, $N_i/N_{\text{pl,R}}$). Points A, B, C and D from Fig. 11 represent the maximum axial capacity, the nominal flexural strength, an intermediate case with both axial and bending and the balance point corresponding to the maximum moment, respectively.

Two coefficients (α , β) are introduced to represent the N – M values at Points C and D. Considering the unknowns α and β , Eqs. (5a)–(5c) are proposed, in which N_i , M_i , $N_{\text{pl,R}}$ and $M_{\text{pl,R}}$ are the axial load capacity and plastic moment capacity in a particular range, the axial load capacity under axial compression from Eqs. (3a), (3b) and the plastic moment capacity under bending only from Eqs. (4a), (4b), respectively.

$$\frac{N_i}{N_{\text{pl,R}}} + (1 - 2\alpha) \frac{M_i}{M_{\text{pl,R}}} - 1 = 0 \rightarrow 2\alpha \geq \frac{N_i}{N_{\text{pl,R}}} > 1.0 \quad (5a)$$

$$\frac{N_i}{N_{\text{pl,R}}} + \left(\frac{\alpha}{\beta - 1} \right) \frac{M_i}{M_{\text{pl,R}}} - \left(2\alpha + \frac{\alpha}{\beta - 1} \right) = 0 \rightarrow \alpha \geq \frac{N_i}{N_{\text{pl,R}}} > 2\alpha \quad (5b)$$

$$\left(\frac{1 - \beta}{\alpha} \right) \frac{N_i}{N_{\text{pl,R}}} + \frac{M_i}{M_{\text{pl,R}}} - 1 = 0 \rightarrow 0 \geq \frac{N_i}{N_{\text{pl,R}}} > \alpha \quad (5c)$$

where $M_{\text{pl,R}} \leq M_{\text{max,R}}$.

As noted before, the performance of RuCFST cross-sections is strongly affected by the presence of rubber, which has direct implications on the confinement levels

achieved. The application of Eqs. (2a), (2b) in conjunction with Eqs. (5a)–(5c) would provide reliable estimates for conventional CFST. However, based on the experimental observations in Sect. 3 and the comparative assessments from Sect. 4.1, these would be unconservative for relatively high levels of ρ_{vr} . The performance of CFST sections is also influenced by other geometric and material parameters such as the tube diameter-to-thickness ratio, D/t , concrete strength f_c and steel strength f_y . As mentioned above, the influence of these parameters on the N – M interaction of CFST and RuCFST cross-sections was investigated by means of nonlinear sectional analyses.

To enable the development of a ρ_{vr} -dependent simplified N – M assessment method, the normalised maximum moment (β) and the normalised axial load (α) are assessed from the nonlinear sensitivity studies described in Sect. 4.3 by varying the parameters D/t , ρ_{vr} and f_c/f_y . A linear regression method was employed to quantify the influence of each factor on α and β . Both ρ_{vr} and f_c/f_y showed significant influence on Points B, C, D and B with various geometric ratios (D/t). Variations of α and β against key parameters are plotted in Fig. 15. As indicated, both α and β increase as D/t and f_c/f_y increase, but decrease proportionally with ρ_{vr} . These assessments enabled the development of two expressions to evaluate α and β (Eqs. 6a, 6b). Note that only sections that are representative of class 1 or class 2 under bending and/or compression are considered [40].

$$\alpha = (1 - 0.05\rho_{vr}) \left(1.30 \frac{f_{rc}}{f_y} + 0.045 \right) \left[1 - 0.25 \left(\frac{D}{t} \right)^{-1} \right] \quad (6a)$$

$$\beta = (1 - 0.10\rho_{vr}) \left(1.25 \frac{f_{rc}}{f_y} + 1 \right) \left[1 - 0.90 \left(\frac{D}{t} \right)^{-1} \right] \quad (6b)$$

$$f_{rc} = \frac{1}{1 + 2 \left(\frac{3\lambda\rho_{vr}}{2} \right)^{3/2}} f_{c0}, \quad (6c)$$

where λ is function of the replaced mineral aggregate size

$$\lambda = \begin{cases} 2.43 \rightarrow d_{g, repl} \in (0, 5) \\ 2.90 \rightarrow d_{g, repl} \in (0, d_{g, max}) \\ 2.08 \rightarrow d_{g, repl} \in (5, d_{g, max}) \end{cases} \quad (6d)$$

Figure 16 shows predictions for the N – M diagram, obtained using the proposed expressions which are compared with the test results. It can be observed that the predictions of Eqs. (3a, 3b)–(5a, 5b, 5c) using the characteristic pairs (β , α) from Eq. (6a), (6b) offer more reliable estimates for the complete range of N – M interactions in comparison with estimates of existing codified provisions [21, 46, 47]. This modified approach is therefore suitable for capturing the influence of relatively high rubber content on the resistance of CFST configurations with class 1 or class 2 steel cross-sections.

As noted before, it should be recalled that the axial compression tests in this study were carried out on relatively short specimens, with L/D of 2.0. To overcome this limitation, a database of specimens with $L/D > 3.0$ was collated and employed for validation of the modified N – M interaction model described in Sect. 4.2. However, as noted in the literature, axial tests with relatively low aspect ratios can be considered as stub-columns, which offer useful information

Fig. 15 Variation of α and β with respect to parameters ρ_{vr} , f_c/f_y and D/t

Fig. 16 Comparison between test results, estimates of Eqs. (3a, 3b–6a, 6b, 6c, 6d) and codified provisions [21, 47, 486] for specimens: **a** CFR00Ezz, **b** CFR30Ezz, **c** CFR60Ezz

on the material confinement effects provided by the steel tube to the infilled concrete. For practical application at member level, further dedicated examination and validation for higher L/D ratios would be required, as the latter can have a notable influence on behaviour. On the other hand, the eccentric and bending tests aspect ratios ($L/D = 4.0$) are within typical ranges in which both end effects and influence from slenderness are minimised or eliminated [30, 31].

5 Concluding remarks

Experimental and analytical investigations into the response of rubberised concrete within circular CFST and RuCFST cross-sections were described in this paper, with particular focus on relatively high rubber content of up to 60%. The tests included a total of 30 specimens subjected to compression, eccentric compression or bending. A detailed account of the test results and observations were presented, with emphasis on the cross-section strength and confinement. Comparative assessments between the test results and existing recommendations were also undertaken alongside sensitivity studies to assess the influence of key geometric and material parameters, leading to proposed simplified expressions for representing the axial–bending cross-section interaction diagrams. The main conclusions drawn are outlined below.

Experimental observations showed that radial expansion and outward folding failure modes were observed in the axial tests on short specimens, which were highly dependent on the proportion of rubber in the concrete infill. A high

amount of elastic energy is accumulated within the RuCFST specimens, as illustrated by the significant spring-back effect of the rubberised infill which increased with the amount of rubber. In the eccentric tests, a combination of axial and flexural buckling was observed without steel fracture in tension. A similar folding failure mechanism was observed for all specimens (filled tubes and hollow tubes) subjected to three-point bending. Crack patterns observed on the concrete core after removal of the steel tube showed flexural-type failure modes without influence from shear.

The response of RuCFST specimens typically exhibited enhanced confinement characteristics with the confined-to-unconfined strength ratios increasing proportionally with the rubber content, yet at the expense of lower nominal load-carrying capacities. This was manifested by a transition from bi-linear post-peak softening response for 0% rubber to post-peak hardening for 60% rubber. Detailed test measurements by means of digital image correlation techniques showed that for specimens under combined loading subjected to the same level of eccentricity, the compression zone extended proportionally with the rubber content. This behaviour indicated improved confinement even for cases of combined loading with significant bending.

Axial–bending interaction curves for RuCFST cross-sections were compared with predictions from existing codified procedures as well as those resulting from nonlinear sectional analyses, that were validated against the tests results from this paper and a collated database on specimens with a range of aspect ratios. It was shown that existing procedures are unable to predict in a reliable manner the strength of RuCFST cross-sections under axial loading or bending-only,

particularly for cases with relatively high rubber content. Accordingly, rubber content-dependent factors were suggested for use in conjunction with code expressions to obtain improved estimates of the axial and bending cross-section capacity, noting that further dedicated experimental assessments and validations for axially loaded rubberised concrete-infilled tubular members of various length-to-diameter ratios are required. Additional nonlinear numerical sectional assessments were also carried out to assess the influence of key parameters on the strength interaction properties. The tests results, added to those from the numerical studies, enabled the development of more reliable simplified expressions for determining the cross-section strength interaction under combined loading conditions.

Acknowledgements The first author wishes to acknowledge the scholarship funding provided by the Turkish Ministry of National Education. The authors would additionally like to thank the technical staff of the Structures Laboratories at Imperial College London, particularly Mr T Stickland, for their assistance with the tests. The support of Adria Abruzzo, Breedon Group, Elkem and Sika through material provision is also gratefully acknowledged.

Author contribution AM was involved in conceptualisation, methodology, investigation, formal analysis and writing—original draft; DVB performed conceptualisation, methodology, investigation, writing—original draft, supervision; AYE contributed to conceptualisation, methodology, writing—review and editing, funding acquisition and supervision.

Declaration

Conflict of interest The authors declare that they have no conflict of interest.

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