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Keywords: FRP confined concrete; Steel tube confined concrete; Constitutive model; Confinement mechanism; axial compressive behaviour

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Monotonic axial compressive behaviour and confinement mechanism of square CFRP-steel tube confined concrete

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Abstract
Steel tube confined concrete (STCC) is widely used in the vertical members of high-rise buildings such as columns. The axial load is not directly resisted by the steel tube in STCC, but is resisted via the interfacial frictional stress between steel tube and concrete core, which is different with that of concrete filled steel tube (CFT) members and would effectively suppress the outward local buckling of steel tube at early stage. Recently, fibre-reinforced polymer (FRP) confined STCC presents a potential to enhance the ductility and durability of such vertical elements. This paper presents an experimental study on monotonic axial compressive behaviour of carbon FRP (CFRP) confined STCC (CFRP-STCC) stub column and an analytical study on the confinement mechanism of and the ultimate axial bearing capacity of the elements. A three-stage confinement mechanism involving the different contributions of the steel tube and the CFRP wrap in CFRP-STCC elements was proposed based on the test results. A prediction model of the ultimate axial bearing capacity of CFRP-STCC stub columns was developed subsequently. Results show that the presence of CFRP wrap enhances effectively the load-bearing capacity and the ductility of steel tube confined plain concrete and reinforced concrete elements, and significantly prevents the local buckling of the steel tubes in the
elements. The proposed prediction model of ultimate axial bearing capacity assesses test results with a great agreement.

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1. Introduction
Reinforced concrete (RC) structures still are widely used in most of the earthquake-prone zones of the world. Numerous studies have revealed that a sufficient confinement can significantly enhance the ductility of RC elements subjected to seismic loads. To achieve an effective confinement, various methods and technical provisions have been developed according to a series of experimental laboratorial studies and earthquake field surveys. Among them, an effective and easily implemented method at the early stage of the previous research is using steel stirrups or hoops with a smaller spacing at the hinge zones of RC elements such as RC columns.

In order to further improve the bearing capacity and seismic performance of RC columns, concrete-filled steel tube (CFT) column (Fig.1a) has been developed and widely applied in civil engineering due to the effective confinement of steel tube in such elements [1]. However, the steel tube of CFT must be thick to avoid its potential local buckling [2]. Steel tube confined concrete (STCC) column (Fig. 1b) is an innovative type of composite columns [3-9], in which the main difference with CFT column is that the steel tube is disconnected to both ends of the column (Fig. 1b). There are two main benefits obtained from this difference of STCC columns. One is the construction simplification of beam-column joints because that steel tube does not need to pass through the joint zone, which has been illustrated by the literature [9]. Another is that the potential local buckling of steel tube can be effectively avoided or delayed as STCC elements are under compressive load. This is because that the steel tube in STCC does not resist directly axial load and mainly provides a confinement to concrete core. It means the thickness of steel tube in STCC can be controlled compared with that of CFT in order to archive the same load-bearing capacity. The STCC elements have the potential of wide applications in new construction. It should be noted that, however, the steel tube in STCC still resists certain axial load from compressive load via the interfacial friction between steel tube and concrete core. But the interfacial friction can be reduced by smoothing the inner surface of steel tube (i.e. oil treatment). However, the main concerns of CFT and STCC elements are the durability issues of external steel tube (i.e. its resistance to corrosion) when they are subjected to aggressive environments. The conventional corrosion protection for steel tube is additional coating. However, some small
Defects could occur in the coating process or the use of steel tubes [2] such as cyclic loads or fatigue loads, which then can cause the pitting corrosion of the tube and then result in the subsequently large area corrosion of the steel tube. Therefore, it is desirable to explore alternative corrosion protection for steel tube.

Fig. 1. Schematic diagram of different confined concrete columns.

Fibre reinforced polymer (FRP) has been widely applied in civil engineering due to its high strength, light weight, good fatigue resistance, and especially excellent durability [10-17]. FRP confined concrete (FCC) column (Fig. 1c) is one of important applications of FRP material in civil engineering to improve the bearing capacity and ductility of concrete core [18-19]. FRP material provides a new choice for steel tube to resist corrosion by wrapping FRP layer on the outside of steel tube. To improve the durability of the outer steel tube of CFT and STCC elements under aggressive environments, and to avoid or delay the early age local buckling of steel tube of CFT elements, several researchers proposed using FRP wrap to confine CFT (FRP-CFT, Fig. 1d) [20-28] or STCC (FRP-STCC, Fig. 1e) [29] elements. FRP-CFT and FRP-STCC elements are two innovative composite elements, which benefit the advantages of both CFT and STCC. The outer FRP wrap/confining can effectively prevent the potential corrosion problem of outer steel tube under aggressive environments and enhance the bearing capacity of CFT/STCC. This means that the same bearing capacity still can be reached in the composite elements when the thickness of steel tube is reduced, which can reduce the manufacturing difficulty of thick steel tube. Meanwhile, it also can delay or even avoid the cracking of the welding seam of the steel tube because of the effective confinement of the outer FRP wrap. It should be admitted that the brittle fracture of FRP material at its ultimate state may lead to a sudden failure of FRP-STCC elements, however, the FRP wrap can provide the STCC higher confinement which could significantly improve the bearing capacity and the peak strain of the STCC.
elements. Due to the large difference of thermal expansivity between FRP and steel, large temperature
difference is considered as a challenge for the interface adhesive in FRP-CFT and FRP-STCC
elements. This environment may cause the degradation of structural performance of the elements, thus
endangers the service life span of the structures. Therefore, high toughness adhesives are suggested to
fabricate the FRP wrap in FRP-CFT and FRP-STCC elements to delay the deterioration of their
structural behaviours caused by a large temperature difference. Moreover, the balance between the
toughness of the adhesives and their glass transition temperatures should be considered, to avoid the
serviceability problems of the elements at higher service temperatures due to low glass transition
temperature. On the other hand, the aging problem of external FRP wrap due to sunlight (mainly
Ultraviolet light) [30], temperature, and humidity is the main concern of the durability of FRP-
confined or -strengthened structures. To fix this issue, a surface treatment such as coating of FRP wrap
is suggested in practical application. As new corrosion protection of steel, the cost of FRP wrap in
FRP-STCC elements is more expensive than those of the conventional corrosion protections of steel,
due to the high price of FRP materials and additional coating materials to resist the aging problems of
FRP. However, FRP wrap is also expected to improve the structural performance (the bearing capacity, peak strain and local buckling, etc.) of STCC elements with the benefits of the material advantages.

Compared to STCC and FCC elements, limited studies were conducted [2,29,31] to understand the
structural behaviour of FRP-STCC elements such as the effectiveness of FRP wrap to prevent the
failure provoked by local damage of steel tube. Lin [29] studied the structural behaviour of circular
glass FRP (GFRP) confined STCC (GFRP-STCC) columns to investigate the effects of the type of and
the number of layers of FRP wrap, stirrup ratio, and loading type. It was reported that FRP wrap, steel
tube, and reinforcements in STCC elements all can enhance significantly the axial load-carrying
capacity and the ductility of the elements [28]. Huang [31] experimentally investigated the cyclic
constitutive behaviour of circular GFRP-STCC columns and proposed a design model to predict the
compressive behaviour of the confined concrete. Xu et al [2] tested circular carbon FRP (CFRP)
confined STCC (CFRP-STCC) stub columns to investigate their eccentric compressive behaviour and
presented $N$-$M$ interaction relationship by a plastic stress distribution method. However, up to now,
only a few parameters were studied to understand their effects of FRP wrap on the constitutive
behaviour of confined concrete [28,31] and no research was reported about square FRP-STCCs.
However, both constitutive behaviour and confinement mechanism are considered very important to
the structural analysis of FRP-STCC structures. To develop a more reliable analysis constitutive model,
more test studies on square FRP-STCC elements are needed to establish the stress-strain law of square
FRP-STCCs.
The main objectives of the paper are to study the monotonic axial compressive behaviour of square CFRP-STCCs and to analyse the confinement mechanism of square steel tube and CFRP wrap in the confined concrete stub columns. Although CFRP materials are more expensive and have a small fracture strain and may cause potential galvanic corrosion issues, however, as a start of the study on the confined STCC elements, CFRP was first selected among commonly used FRP materials (i.e. CFRP, GFRP, aramid FRP, and basalt FRP). The main reasons are: (1) The elastic modulus of CFRP materials is close to that of steel materials, which means it is easier to work together with the steel tube, compared with the other FRP materials. (2) CFRP materials have a higher strength-weight ratio, which means it has a high potential to effectively improve the confinement of the inside concrete in STCC elements. (3) The basic research conclusions of CFRP-STCC are also applicable to those of the STCC confined by other FRP materials due to the inherent linear elastic response of FRP materials. Based on the experimental study, a calculation model was proposed to assess the axial bearing capacity of CFRP-STCC stub columns. The investigation mainly includes failure modes, load-deformation behaviour, the influence of main parameters (the number of layers of CFRP wrap, width-to-thickness ratio of steel tube, corner radius at sectional corner), and confining stress analysis of CFRP-STCCs.

2 Test investigation

2.1 Test specimens

In this study, total 23 specimens were prepared and tested, including 11 square CFRP-steel tube confined plain concrete (CFRP-STCC) stub columns, 3 square steel tube confined plain concrete (STCC) stub columns, 6 square CFRP-steel tube confined reinforced concrete (CFRP-STCRC) stub columns and 3 square steel tube confined reinforced concrete (STCRC) stub columns. The height-to-width ratio \( H/B_0 \) of all specimens is 3.0. Fig. 2 gives the details of the test specimens. The volumetric ratios of the longitudinal reinforcement (4Φ12) and steel stirrup (Φ6@200) of confined RC specimens were 2.0% and 0.4%, respectively. The steel stirrups in the related specimens were only used to fix the longitudinal reinforcements, and the hoop confinement of them to the concrete core was ignored in the later analysis. In order to ensure that applied axial load was transferred uniformly to the internal longitudinal reinforcement in the specimens, both ends of each longitudinal rebar were welded to the bottom and top steel plates of each specimen (see Fig. 2b), respectively. In order to guarantee that the steel tube does not directly bear axial load in each specimen, a ring with a length of 10 mm was cut after casting from both ends of steel tube (40 mm from the ends), forming two girth gaps in each specimen shown in Fig.2. A wet lay-up process was used to conduct CFRP wrap to steel tubes in the
Before CFRP was wrapped, the floating rust and impurities on the surface of the steel tubes were removed with a fine sandpaper and using an alcohol treatment. CFRP sheets with the same height as that of the steel tube were then uniformly and tightly wrapped on the outer surface of the steel tube with an epoxy adhesive. The overlapping length of CFRP sheets was 120 mm according to the Chinese Code (GB 50608-2010) [32], which was arranged to cover one of the welding seams of steel tube (seen Fig. 3). The details of each specimen are listed in Table 1. The studied corner radiiuses of the steel tubes were 10 mm, 20 mm, 30 mm, as PC-D-2-2(10), PC-B-2-2 and PC-D-2-2(30) specimens listed in the table, respectively.

Table 1. Details of test specimens

<table>
<thead>
<tr>
<th>Types</th>
<th>Specimen no.</th>
<th>Steel tube</th>
<th>CFRP</th>
<th>R /mm</th>
<th>Cross section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>t/mm</td>
<td>B/t</td>
<td>n</td>
<td>t_{frp} /mm</td>
</tr>
<tr>
<td>Confined plain concrete (PC)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PC-A-1-0</td>
<td></td>
<td>1</td>
<td>152</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>PC-A-1-1</td>
<td></td>
<td>1</td>
<td>152</td>
<td>1</td>
<td>0.167</td>
</tr>
<tr>
<td>PC-A-1-2</td>
<td></td>
<td>1</td>
<td>152</td>
<td>2</td>
<td>0.334</td>
</tr>
<tr>
<td>PC-A-1-3</td>
<td></td>
<td>1</td>
<td>152</td>
<td>3</td>
<td>0.501</td>
</tr>
<tr>
<td>PC-B-2-0</td>
<td></td>
<td>2</td>
<td>77</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>PC-B-2-1</td>
<td></td>
<td>2</td>
<td>77</td>
<td>1</td>
<td>0.167</td>
</tr>
<tr>
<td>PC-B-2-2</td>
<td></td>
<td>2</td>
<td>77</td>
<td>2</td>
<td>0.334</td>
</tr>
<tr>
<td>PC-B-2-3</td>
<td></td>
<td>2</td>
<td>77</td>
<td>3</td>
<td>0.501</td>
</tr>
<tr>
<td>PC-C-3-0</td>
<td></td>
<td>3</td>
<td>52</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>PC-C-3-1</td>
<td></td>
<td>3</td>
<td>52</td>
<td>1</td>
<td>0.167</td>
</tr>
<tr>
<td>PC-C-3-2</td>
<td></td>
<td>3</td>
<td>52</td>
<td>2</td>
<td>0.334</td>
</tr>
<tr>
<td>PC-C-3-3</td>
<td></td>
<td>3</td>
<td>52</td>
<td>3</td>
<td>0.501</td>
</tr>
<tr>
<td>PC-D-2-2(10)</td>
<td></td>
<td>2</td>
<td>77</td>
<td>2</td>
<td>0.334</td>
</tr>
<tr>
<td>PC-D-2-2(30)</td>
<td></td>
<td>2</td>
<td>77</td>
<td>2</td>
<td>0.334</td>
</tr>
</tbody>
</table>
Note: $B/t$ is the width-to-thickness ratio of steel tube; $t$ and $t_{frp}$ are the thickness of steel tube and CFRP wrap, respectively; $n$ is the number of layers of CFRP; $R$ is the corner radius of steel tube.

### 2.2 Material properties

The elastic modulus, the yield load, and the ultimate tensile strength of the used steel tubes were measured according to the Chinese Code, GB/T 228-2002 [33]. The test results are shown in Table 2. The longitudinal rebars were HRB 335 rebars with a diameter of 12 mm, a measured yield strength of 378 MPa and an ultimate tensile strength of 540 MPa. A standard commercial concrete with a maximum coarse aggregate size of 10.0 mm was used in all specimens which was supplied by a local company. Three cylinders of $\varnothing150 \times 300$ mm were tested under axial compression to define the compressive strength of used concrete. The average compressive strength of unconfined concrete was 55.4 MPa. The related material properties of CFRP sheet (surface density: 300 g/m$^2$, provided by Toray Co., Ltd, Japan), and of epoxy adhesive (provided by Dalian Kaihua New Technology Engineering Co., Ltd, China), were provided by manufacturers and listed in Table 2. In order to avoid potential galvanic corrosion between CFRP wrap and steel tube in practical application, a thin insulating layer (i.e. Glass FRP) must be wrapped firstly before wrapping CFRP sheet on steel tube. However, the insulating layer was not applied in the study considering the test is short-term without such galvanic corrosion issue. Although the CFRP-STCC elements proposed in this paper are relative complex, consisting of steel rebars, concrete, steel tube, GFRP, CFRP, epoxy layers, and an additional protection layer, it is one of the ways to effectively solve the corrosion problem of steel tube. And if CFRP is replaced by GFRP in the elements, the additional insulating layer is not needed. Moreover, to resist the steel corrosion, similar technologies using FRP wrap on steel tube had already been applied in the structures with steel piles located in several harbours in China [31]. These projects preliminarily proved the effectiveness of the FRP wrap to resist steel corrosion of the structures. Therefore, as one of the treatments of durability and effective confinement methods, the proposed FRP-STCC elements present the potential of wide applications in practical projects to address the corrosion problem of steel.
tube and improve the structural performance of the elements. In addition, to simplify the analysis, the 
axial compressive behaviour contributed from the thin GFRP insulating layer can be omitted due to 
the layer can be very thin in the practical application of CFRP-STCC elements.

Table.2 Material properties of steel tube, CFRP sheet and epoxy adhesive

<table>
<thead>
<tr>
<th>Materials</th>
<th>Nominal thickness /mm</th>
<th>Elastic modulus /GPa</th>
<th>Yield tensile strength /MPa</th>
<th>Ultimate tensile strength /MPa</th>
<th>Elongation /%</th>
</tr>
</thead>
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<tr>
<td>Steel #1</td>
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<td>188</td>
<td>330</td>
<td>-</td>
</tr>
<tr>
<td>Steel #2</td>
<td>2.0</td>
<td>204</td>
<td>192</td>
<td>345</td>
<td>-</td>
</tr>
<tr>
<td>Steel #3</td>
<td>3.0</td>
<td>205</td>
<td>200</td>
<td>323</td>
<td>-</td>
</tr>
<tr>
<td>CFRP</td>
<td>0.167</td>
<td>245</td>
<td>-</td>
<td>4077</td>
<td>1.51</td>
</tr>
<tr>
<td>Epoxy</td>
<td>-</td>
<td>&gt;2.5</td>
<td>-</td>
<td>&gt;40</td>
<td>&gt;1.80</td>
</tr>
</tbody>
</table>

2.3 Loading and measurement

The measurement and setup of the test are presented in Figs. 3 and 4. A monotonic axial compressive 
loading was applied on each specimen by a 5000 kN hydraulic compressive machine (see Fig. 4), 
which was controlled by vertical displacement with a rate of 0.5mm per minute referring to the 
literature [1]. The axial compressive load was measured by a load cell placed on the top of the 
specimens. Two linear variable displacement transducers (LVDTs) with a measurement range of 
50 mm were arranged symmetrically on the diagonal direction of the test specimens to measure the 
vertical displacement of stub columns, as shown in Figs. 3 and 4. Twelve strain gauges with a gauge 
length of 20 mm were installed on CFRP wrap to measure the axial and hoop strains of CFRP wrap 
and steel tube at the mid-height of the test specimens, as shown in Fig. 3. Since CFRP wraps were well 
bonded to steel tubes with epoxy adhesive, the inner steel tube was considered to work together with 
the outer CFRP wrap without interfacial slippage. Therefore, the strains of the inner steel tube were 
assumed to be the same as those of the outer CFRP wrap. The strain and load information were 
collected synchronously at an acquisition frequency of 1.0 Hz.
3 Test observations and analyses

3.1 Failure modes

The damage and failure modes of the steel tube confined concrete specimens and the CFRP-steel tube confined concrete specimens are shown in Fig. 5. In the steel tube confined concrete columns, the concrete cover at the ends of steel tube experienced sporadic crushing or spalling when approaching the peak loads of the columns. When the axial load dropped to around 70% of their peak load, the steel tube near the middle section suffered a significant outward local buckling. After removing the steel tubes, several obvious shear damages were observed in the steel tube confined plain concrete specimens, as shown in Fig. 5 (a), (b) and (c). On contrast, the shear failure was not pronounced in the steel tube confined RC specimens instead of evenly distributed cracks, as shown in Fig. 5 (f), (j) and (h), indicating that the installation of longitudinal reinforcements improved the axial compressive
behaviour of steel tube confined concrete.

For the CFRP-steel confined concrete specimens, their ultimate failure was dominated by the hoop rupture of CFRP wrap (see Fig. 5 (d), (e), (i) and (j)). After the fracture of CFRP wrap, the local

Fig. 5. Failure models of several representative confined concrete stub columns.
buckling of steel tube near specimens’ mid-height section was observed and then the whole specimen failed. After removing the steel tubes, diagonal shear cracks still were observed in the surface of the concrete core in the specimens, shown in Fig. 5 (d) and (e). However, the shear failure was avoided in the CFRP-steel tube confined RC specimens (Fig. 5i and j), which confirms that the addition of longitudinal reinforcement can play a beneficial effect on the axial compressive behaviour of CFRP-steel tube confined concrete columns.

3.2 Axial load-strain behaviour

Figs. 6 and 7 depict the axial load-strain curves for several representative CFRP-steel tube confined plain concrete specimens. In this study, the nominal axial strain was calculated as a ratio of the axial shortening to the initial height of specimens, while the hoop strain was the average measured strain by four hoop strain gauges installed on the corners or middle sections.

Fig. 6. Axial load-strain curves of confined plain concrete specimens.
Results show that all confined plain concrete and confined RC specimens deformed elastically at the early stage. The axial deformation increased approximately linearly, and its increasing rate was much greater than that of the lateral deformation. With the increasing of axial deformation, the lateral deformation at the corners ($\varepsilon_{h,c}$) was smaller than the deformation at the middle of steel tube side at the middle section ($\varepsilon_{h,p}$). This indicates that the concrete deformation at the corners of the steel tubes was restrained well while the other deformations at the middle section are not well confined. The bearing capacity of steel tube confined concrete specimens rapidly decreased after the specimens reached their peak loads, and the axial load tended to stabilize when the peak load was reduced to a certain load ranging from 50% to 90% of corresponding peak load.

![Axial load-strain curves of confined reinforced concrete specimens.](image)

For both CFRP-steel confined plain concrete and confined RC specimens, their load carrying capacity started to decrease after the specimens reached their first peak load. The lower the number of layers of CFRP was, the larger the decrease of the bearing capacity was. When the curves decreased to
a certain extent, the hoop strain of the confined concrete started to increase and the curves began to
slightly rise. The greater the number of layers of CFRP wrap used in the specimens, the higher the
increase rate of the bearing capacity was. The softening phenomenon indicates that the confinement
effectiveness of FRP-steel tube in square section concrete specimens was relatively weak. The
softening phenomenon also occurred in CFRP-steel tube confined RC columns. However, the peak
load of the curves in the second rising section was generally larger than that of the confined plain
cement specimens, e.g., PC-B-2-3 and RC-B-2-3 specimens. It shows that the deformability of
confined concrete specimens was improved after reinforcing rebars were added to the columns. This
improvement was more conducive to the development of the confinement effectiveness of the FRP-
steel composite tube so that the load carrying capacity of the columns increased.

3.3 Stress-strain relationship of steel tube

The confinement of steel tube to concrete core can be understood by analysing the longitudinal and
transverse stress of the steel tube. Referring to the literature [34], the stress of steel tube during loading
was determined based on the hoop and axial strain in the middle of the specimen. This brings a better
understanding of the confinement effectiveness of the steel tubes in the composite elements. Due to a
thin-walled steel tube was used in this study, the force perpendicular to the wall of steel tubes is small
and can be neglected. For this, the steel tube can be considered under the state of plane-stress [35]. Fig.
8 demonstrates the main calculation method of stress analysis of the steel tube at three stages. At the
elastic stage, the stress-strain relationship was assumed to obey the Hooke’s law. An elastic increment
theory [34] was used to determine the stress of steel tube at the elastic-plastic stage (AB). The Von-
Mises yield criterion and the Prandtl-Reuss flow rule were adopted to analyse the behaviour of steel
tube at the plastic hardening stage (BC) [36]. In Fig. 8, \( \sigma_h \) and \( \varepsilon_h \) are the hoop stress and strain of steel
tube, \( \sigma_z \) and \( \varepsilon_z \) are the axial stress and strain of steel tube, \( \sigma_z \) is the equivalent stress of steel tube, \( \mu_e \) is
Poisson’s ratio of steel in the elastic stage, \( E^E_\sigma \) and \( \mu_{sp} \) are the tangent modulus and Poisson’s ratio of
the steel in the elastoplastic stage, \( \sigma_{h}^' \), \( \sigma_{v}^' \) and \( \sigma_{cp} \) are the hoop and axial deviatoric stress of steel and
its mean stress, \( G \) is shear modulus of the steel, \( f_y \) and \( f_p \) are the steel yield strength and proportional
limit (0.8f_y), \( \varepsilon_p \) and \( \varepsilon_y \) are the equivalent strain of steel corresponding to \( f_p \) and \( f_y \), respectively. \( p, H' \)
and \( Q \) are defined parameters for the calculation [34].

It should be noted that the transverse and axial strains used for the stress analysis of steel tubes are
the strains at the middle of the mid-section of the steel tube. Fig. 9 shows the relationship between the
axial load and the stress of steel tube developed in several specimens. The tensile stress was
considered to have a negative sign in the stress analysis of steel tube. It was found that the axial stress
increased more quickly than the hoop stress at the early stage, and the growth rate gradually increased with the increase of axial load. The yielding of steel tubes of the specimens was confirmed around their first peak loads. After that point, the hoop stress of the steel tubes increased slowly, but in some cases, a negative evolution was observed such as PC-B-2-1 and PC-D-2-2 (10). In these specimens, the axial load decreased sharply too. This leads to the fact that the confinement of steel tube to concrete core was effectively confined anymore after the significant expansion of concrete, which then affected the bearing capacity of the specimens. In the CFRP-steel tube confined concrete specimens, the hoop stress of the steel tube increased after the first peak load, and the load carrying capacity of the specimens decreased slowly or increased slightly such as Specimen RC-C-3-3. This implies that the FRP wrap can not only confine the concrete core, but can also confine the steel tube, which increases the confinement effect of the steel tube on concrete core.

\[
E_i = \frac{(f_i - \sigma_i)}{(f_i - f_p)} f_p, \quad \rho = \frac{2H'}{9E_s} \sigma_z^2, \quad \sigma_z = \sqrt{\sigma_z^2 + \sigma_v^2 - \sigma_s \sigma_v}, \quad \sigma_w = \frac{1}{3}(\sigma_s + \sigma_v) \quad \sigma'_w = \sigma_s - \sigma_v
\]

\[
Q = \sigma_s^2 + \sigma_v^2 + 2\mu_s \sigma_v^2 + \frac{2H'(1-\mu_s)^2 \sigma_v^2}{9G}, \quad \mu_s = 0.167 \frac{f_s - f_p}{f_s}, \quad \sigma_v = \sigma_s - \sigma_w, \quad H' = 3 \times 10^3 E_s
\]

**Fig. 8.** Stress analysis of steel tube [34].

Besides, a similar test observation to that of the confined concrete specimens was confirmed in the confined RC specimens. The confinement effectiveness of the FRP-steel tube on the concrete core was stronger than those in the concrete specimens. For example, although the steel tube yielded in several specimens, their bearing capacity kept increasing (see RC-C-3-3). This implies that the CFRP-steel tube confined RC columns present better ductility and deformability compared to the confined plain concrete columns.
3.4 Stress-strain responses of confined concrete

Applying the stress analysis of steel tube, the axial load resisted by steel tube can be discussed. In addition, the main fibres of CFRP wrap are only oriented in the hoop direction, so that the stiffness of the CFRP wrap in the direction perpendicular to the hoop direction is very small and can be ignored. When the axial stiffness of CFRP wrap is ignored, the load supported by concrete core can be calculated as the total load of the specimens deducted the load resisted by steel tube. Assuming the compressive stress on the entire section of concrete core is uniformly distributed, the compressive load of confined concrete can be calculated by dividing the deducted load by its cross-sectional area. Moreover, for confined RC specimens, the axial bearing contribution of the longitudinal reinforcement
should be deducted from the load resisted by whole column. In summary, the axial stress of confined
concrete can be obtained by,

\[
\sigma_c = \begin{cases} 
\frac{N - \sigma_v A_s}{A_c} & \text{for confined plain concrete} \\
\frac{N - \sigma_v A_s - f_y A_d}{A_c} & \text{for confined reinforced concrete}
\end{cases}
\]

(1)

where \( \sigma_c \) is the axial stress of confined concrete; \( N \) is the axial load resisted by whole column; \( \sigma_v \) is
the axial stress of steel tube; \( f_y \) is the yield strength of longitudinal reinforcement in the columns; \( A_s \),
\( A_d \) and \( A_c \) are the cross-sectional areas of the steel tube, the longitudinal reinforcement and the
concrete core, respectively. Besides, the axial deformation of the confined concrete is believed to be
identical to the nominal axial strain of the specimens. Table.3 lists a summary on the calculated results
of the axial stress and measured strain of the concrete cores in the specimens, while Fig. 10 shows the
stress-strain curves of the confined concrete.

Results plotted in Fig. 10 demonstrate that the initial elastic moduli of the confined plain concrete
and RC are basically identical when compared within the same group. The first peak stress of the
CFRP-steel tube confined plain concrete specimens in Groups PC-A and PC-B (or Groups RC-A and
RC-B for confined RC specimens) were larger than those of the STCC specimens. The difference
among the CFRP-steel tube confined concrete or RC specimens was small, especially in Groups PC-C
and RC-C. This is explained by the fact that the \( B/t \) ratio of steel tube in Group A is large (\( B/t = 152 \))
indicating that the confining stress of the steel tubes was much smaller than others for it is prone to be
buckling failure. This also is the reason why the relatively weak confinement to suppress the
expansion deformation of the concrete cores in the specimens. When FRP wrap was used, the wrap
can not only restrain the lateral dilation of concrete core but also suppress the local buckling
deformation of steel tube, so that steel tube can continue to exert its confinement effect.

![Graphs showing stress-strain curves for different groups](image-url)
Fig. 10. Axial stress-strain curves of confined concrete.

Table 3. Summary of axial stress and axial strain of confined concrete.

<table>
<thead>
<tr>
<th>Groups</th>
<th>Specimens</th>
<th>$f_{c1}$ /MPa</th>
<th>$\varepsilon_{c1}$ /%</th>
<th>$f_{c2}$ /MPa</th>
<th>$\varepsilon_{c2}$ /%</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC-A</td>
<td>PC-A-1-0</td>
<td>58.84</td>
<td>0.207</td>
<td>59.32</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td>PC-A-1-2</td>
<td>67.50</td>
<td>0.389</td>
<td>66.30</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td>PC-A-1-3</td>
<td>68.11</td>
<td>0.428</td>
<td>63.30</td>
<td>1.51</td>
</tr>
<tr>
<td></td>
<td>PC-B-2-0</td>
<td>79.23</td>
<td>0.313</td>
<td>59.32</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td>PC-B-2-1</td>
<td>79.89</td>
<td>0.490</td>
<td>53.33</td>
<td>1.43</td>
</tr>
<tr>
<td></td>
<td>PC-B-2-2</td>
<td>80.90</td>
<td>0.498</td>
<td>72.79</td>
<td>1.62</td>
</tr>
<tr>
<td></td>
<td>PC-B-2-3</td>
<td>83.24</td>
<td>0.512</td>
<td>84.86</td>
<td>2.78</td>
</tr>
<tr>
<td></td>
<td>PC-C-3-0</td>
<td>82.14</td>
<td>0.418</td>
<td>63.30</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td>PC-C-3-1</td>
<td>83.86</td>
<td>0.378</td>
<td>65.67</td>
<td>1.82</td>
</tr>
<tr>
<td></td>
<td>PC-C-3-2</td>
<td>82.28</td>
<td>0.388</td>
<td>72.02</td>
<td>2.24</td>
</tr>
<tr>
<td></td>
<td>PC-C-3-3</td>
<td>81.71</td>
<td>0.402</td>
<td>78.80</td>
<td>2.12</td>
</tr>
<tr>
<td>PC-D</td>
<td>PC-D-2-2 (10)</td>
<td>75.03</td>
<td>0.425</td>
<td>63.30</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td>PC-D-2-2 (30)</td>
<td>85.94</td>
<td>0.692</td>
<td>83.24</td>
<td>1.63</td>
</tr>
<tr>
<td>RC-A</td>
<td>RC-A-1-0</td>
<td>63.95</td>
<td>0.274</td>
<td>59.32</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td>RC-A-1-2</td>
<td>64.87</td>
<td>0.300</td>
<td>50.86</td>
<td>1.86</td>
</tr>
<tr>
<td></td>
<td>RC-B-2-0</td>
<td>67.80</td>
<td>0.445</td>
<td>59.32</td>
<td>1.45</td>
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<tr>
<td></td>
<td>RC-B-2-2</td>
<td>73.24</td>
<td>0.526</td>
<td>76.28</td>
<td>2.72</td>
</tr>
<tr>
<td></td>
<td>RC-B-2-3</td>
<td>69.67</td>
<td>0.503</td>
<td>78.24</td>
<td>2.39</td>
</tr>
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<td>RC-C</td>
<td>RC-C-3-0</td>
<td>78.47</td>
<td>0.489</td>
<td>63.30</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td>RC-C-3-2</td>
<td>74.84</td>
<td>0.622</td>
<td>76.98</td>
<td>2.12</td>
</tr>
<tr>
<td></td>
<td>RC-C-3-3</td>
<td>76.98</td>
<td>0.662</td>
<td>84.02</td>
<td>2.31</td>
</tr>
</tbody>
</table>

Note: $f_{c1}$ and $\varepsilon_{c1}$ are the first peak stress and corresponding nominal axial strain of confined concrete; $f_{c2}$ and $\varepsilon_{c2}$ are the ultimate stress and corresponding nominal axial strain of confined concrete at the rupture of FRP wrap.

In the confined plain concrete and RC specimens, following the first peak axial stress, the effective confining stresses of the steel tube and FRP wrap in the square section are relatively small. Similar to previous research, the confinement is effective only in a limited confinement area in square concrete. It cannot prevent the expansion deformation of concrete in the non-effective confinement area. This was the reason why the stress-strain curves of the concrete exhibited different degrees of softening. The softening segment was smaller as the number of CFRP layers increased, and the stress-strain curves of confined concrete after this stage increased with varying degrees. This indicates that the
lateral expansion deformation of the concrete core increased and the confining stress of CFRP wrap increased, leading to an increase in confining stress to the concrete core. The axial stress of the confined concrete increased until the hoop rupture of CFRP wrap. The slope of the secondary ascending branch of the axial stress-strain curves increased with the number of layers of CFRP. Besides, the corner radius of the steel tube has a significant influence on the stress-strain curves of confined concrete, as shown in Fig. 10 (d). Results show that the strength and ductility of confined concrete corresponding to a steel tube with a corner radius of 30 mm is significantly better than that of the specimen with a corner radius of 10 mm.

In addition, it is worth mentioning that the size effect also is an important affecting factor of the composite confined columns especially for square columns. The hoop strain of CFRP wrap is non-uniformly distributed along the circumferential direction. The hoop strain of CFRP wrap at the corners varies with the sectional size of square columns, leading to a considerable influence on the compressive behaviour of confined concrete. To the best of the authors’ knowledge, the size effect in square FRP-steel tube confined plain concrete or RC columns has not been understood well. However, the study conducted by Wang et al. [37] on square FRP-confined RC columns can provide a significant reference to this issue. The experimental results [37] revealed that the compressive strength of square FRP-confined concrete decreased with cross-section size, while ultimate axial strain was influenced little by section size. Therefore, the size effect also may have an important impact on the axial compressive behaviour of square FRP-STCC elements, which deserves further concerns in the future.

3.5. Effects of test parameters

(1) Effect of the number of CFRP layers

Fig. 11 depicts the effect of the number of CFRP layers on the axial load-strain behaviour of steel tube confined concrete specimens and CFRP-steel tube confined concrete specimens, where the lateral strain is the measured strain at the corners of the specimens. Results show that the number of CFRP layers affects the first peak loads and corresponding axial strain. When the number of CFRP layers increased, the degree of post-peak softening of the specimens decreased significantly. After the first peak load, the curves of the CFRP-steel tube confined concrete specimens were much smoother than those of the steel tube confined concrete specimens. The more CFRP layers were used, the more gradual the curves exhibited and the higher the ultimate axial deformation of the specimens was. A significant increase was confirmed in the axial load-strain responses of the specimens with 3-ply FRP wrap after their softening stage, which is demonstrated by the fact that the bearing capacities of the
specimens even exceeded their first peak loads in some cases. This indicates that the CFRP wrap can work with steel tube together to provide an effective confinement to concrete core, where the steel tube can effectively prevent the local and sharp damage of FRP wrap while the FRP can confine the steel tube at large hoop deformations.

For the CFRP-steel tube confined RC specimens, the elastic behaviour and first peak load of the specimens are not significantly affected by the number of CFRP layers. The first peak loads were slightly larger than those of steel tube confined specimens. After first peak load, the axial load-strain curves of the CFRP-steel tube confined RC specimens continued to rise until the rupture of CFRP wrap. The ultimate bearing capacities of the CFRP-steel tube confined RC specimens with 3-ply FRP wrap corresponding to the rupture of FRP wrap were larger than their first peak loads. This means that with the increase of the number of CFRP layers, the co-confinement effectiveness of CFRP-steel tube to the square concrete core is significantly enhanced.

(2) Effect of the width-to-thickness ($B/t$) ratio of steel tubes

As shown in Fig. 12, the specimens with higher $B/t$ ratio present smaller bearing capacities. Compared to the load capacity of the specimens using a $B/t$ ratio of 152.0, the first peak loads of both the specimens with $B/t$ ratios of 52.0 and 77.0 were higher. This means that the $B/t$ ratio of the steel tube has a significant influence on the bearing capacity of the CFRP-steel tube confined concrete specimens. This is similar to the cases of the steel tube confined concrete elements. Besides, the
smaller the $B/t$ ratio was, the higher the load carrying capacity and ductility of the stub columns were. A similar result was found in the CFRP-steel tube confined RC specimens, but it seems that the $B/t$ ratio has a slightly stronger influence on the first peak loads and on the ductility of the specimens.

Fig. 12. Effect of width-to-thickness on axial load-strain curves at different FRP layers.

(3) Effect of corner radius at sectional corners

The effects of three levels of the corner radius of steel tube were experimentally study, i.e., 10 mm, 20 mm and 30 mm, respectively, as shown in Fig. 11 (c). The results show that the ultimate load of the specimens increases significantly with the increase of the corner radius. The softening behaviour of the curves after the first peak load was significantly reduced and slowed down as the radius increases. This presents the potential to improve the mechanical properties of square sectional confined plain concrete or RC columns by properly increasing the corner radius of column section. This is explained by the fact that more concrete core can be effectively confined in the columns, which is illustrated later in the study.

4. Discussion on confinement mechanism

4.1 Effective confinement of steel tube and FRP in confined square section

With reference to the cases in traditional square stirrup confined concrete, the effective
The confinement mechanism of either steel tube confined concrete or FRP-steel tube confined concrete is presented in Fig. 13. In these sections, only the concrete in the area enclosed by four parabola lines with initial tangent lines 45° from the corresponding sides of the section (see Fig. 13 (a)) can be effectively confined. This is a significant difference compared to the cases in circular confined-plain concrete or RC. Pham and Hadi [38] proposed a confinement mechanism of the concrete in confined square columns, which is shown in Figs. 13 (b) and (c). The confining stress at the corners is much larger than that at the four sides since the curvature radius of sectional sides is much greater than that of the corners. The confining stress $f_r$ at the corners is given as

$$f_r = \frac{\sigma_{h,j}}{R}$$  \hspace{1cm} (2)

where $\sigma_{h,j}$ is the hoop stress of a confining jacket at the corners; $R$ is the corner radius.

According to Section 3.3, the confining stress provided by the steel tube $f_{r,s}$ is expressed as

$$f_{r,s} = \frac{\sigma_h}{R}$$  \hspace{1cm} (3)

where $\sigma_h$ is the hoop stress of steel tube at the corners.

Therefore, according to Fig. 13 (c), the confining stress of FRP wrap $f_{r,frp}$ is given as

$$f_{r,frp} = \frac{\sigma_{h,frp}}{R + t} = \frac{E_{frp} \varepsilon_{f,c} t_{frp}}{R + t}$$  \hspace{1cm} (4)

where $\sigma_{h,frp}$ and $\varepsilon_{f,c}$ are the hoop stress and hoop strain of the FRP wrap at corners, respectively; $E_{frp}$ and $t_{frp}$ are the Young’s modulus and thickness of FRP wrap, respectively.

**Fig. 13.** The confinement of square confined concretes: (a) effective confining area of confined concrete; (b) stress distribution; and (c) confinement mechanism of FRP confined concrete [38].
Fig. 14 shows the evolution of the confining pressure of the steel tube and the CFRP wrap in the specimens, as well as the total confining pressure with the increasing nominal axial strain of the stub columns. Results show that the confining pressure of the steel tube increases rapidly at the initial stage of loading, and then increases slowly or almost remains constant during the later period. This indicates that the confining pressure of steel tube to the concrete core is limited after the yielding of the steel tube. On the other hand, the confining pressure provided by CFRP wrap was not high at the initial loading. Due to the increase of the lateral deformation of the steel tube, the FRP wrap started to provide a higher confining stress, for example, from an axial strain of 0.004 to 0.006. After that, the confining pressure of the CFRP wrap increased until the rupture of the FRP wrap. No obvious difference was found between the CFRP-steel tube confined plain concrete and RC specimens.

4.2 Confinement mechanism of square FRP-steel tube confined concrete/reinforced concrete

Based on the above analysis, Fig. 15 shows an ideal evolution of various confining pressures in FRP-steel tube confined plain concrete and RC columns, which explains the confinement mechanism of the composite tube to concrete core. The evolution of the confining pressure provided by steel tube and FRP wrap in the composite columns is similar to that observed in FRP-confined CFT specimens reported by Hu et al. [1]. However, the confinement mechanism of the specimens still is different from that in FRP-confined CFT specimens for the steel tube does not directly carry the axial load.
According to Fig. 15, the confinement actions in FRP-steel tube confined plain concrete and RC columns can be divided into three stages as follows,

1. **1st stage – steel tube confinement stage**
   In this stage, the confining pressure of the concrete core comes mostly from the confinement of steel tube, while the confinement from FRP wrap can be nearly neglected. This is because the test specimens are only subjected to a small axial compression load, resulting in a very small lateral expansion in the concrete core at this stage. There are few obvious differences between the confined plain concrete and the confined RC columns as the stirrups were limited and only to erect the longitudinal reinforcements in the study. Therefore, it is believed that the stirrups only provide a quite small confinement to the concrete core. The small lateral deformation induced by a small axial strain in the concrete core does not need the confinement action of FRP wrap. Therefore, if the potential deformation of the confined plain concrete or RC columns remains at this level, the additional FRP confinement is not necessary from the point of view of the mechanical performance of the elements.

![Fig. 15. Ideal confinement in FRP-steel tube confined concrete columns.](image)

2. **2nd stage – FRP-steel tube co-confinement stage**
   The second stage can be considered as a co-confinement stage consisting of both the confining pressures from steel tube and FRP wrap. However, as shown in Fig. 15, the two types of confining pressures increase at different rates depending primarily on their hoop stiffness. This stage is similar to the case in FRP-confined CFT columns [1]. The total confining pressure increases rapidly in this stage, as the lateral deformation of concrete core starts to rapidly increase. Based on the experimental investigation in the present study, the second stage can be delimited to a nominal axial strain of around...
The FRP and steel tube work together in this stage and delay their respective fracture or local buckling due to the contribution of each partner.

(3) 3rd stage – FRP-dominated confinement increasing stage

The third stage of the confinement of FRP-steel tube confined concrete is dominated by FRP confinement. In this stage, the increasing total confining pressure to inner concrete comes mainly from the increasing confinement of FRP wrap, as the confinement of the steel tube keep almost a constant level after its yielding. The high strength feature of FRP materials becomes apparent at this stage. At the same time, the behaviour of the FRP material itself still is highly elastic, and the confining pressure of the FRP wrap can keep a similar increasing rate to that of the second stage. Therefore, at this stage, the increasing rate of the total confining pressure of confined concrete or RC columns at this stage becomes smaller than that of the second stage, which is similar to the previous research results of FRP-confined CFT columns [1].

5. Proposal for predicking axial bearing capacity of composite square stub columns

Referring to previous research [39, 40], the superposition principle was used to predict the axial bearing capacity of CFRP-steel tube confined plain concrete or RC stub columns \( N_u \), which is given as

\[
N_u = f_{CFS} A_c + f_a A_a
\]

where \( A_c \) and \( A_a \) are the cross-sectional areas of concrete core and longitudinal reinforcement, respectively; \( f_a \) is the yield strength of longitudinal reinforcement; and \( f_{CFS} \) is the compressive strength of CFRP-steel tube confined concrete.

Based on the test results reported in this paper, a superposition calculation method is applied to predict the axial bearing capacity of CFRP-steel tube confined plain concrete or RC stub columns, consisting of the contribution of steel tube and FRP wrap. The discussion on the steel tube, FRP and FRP-steel tube confined concrete is presented in the following sections.

(1) For steel tube confined concrete

According to the literature, the calculation model for steel stirrup-confined concrete strength \( f_{cc} \) proposed by Mander et al. [41] is given as

\[
f_{cc} = f_{co} \left( 1 + 2.254 \sqrt{1 + \frac{7.94 f_r}{f_{co} - 2.254}} - 2 \frac{f_r}{f_{co} - 2.254} \right)
\]

where \( f_{co} \) is the compressive strength of unconfined concrete, and \( f_r \) is the confining pressure provided by steel stirrups.
Referring to this model, the ultimate compressive strength of steel tube confined concrete \( f_{CS} \) is given as
\[
f_{CS} = f_{co} \left( 1 + 2.254 \left( 1 + \frac{7.94 f_{r,s}}{f_{co}} - 2 \frac{f_{r,s}}{f_{co}} \right) \right)
\]  
(7)

where \( f_{r,s} \) is the confining pressure provided by steel tube calculated based on a static equilibrium, which is given as
\[
f_{r,s} = \frac{2 \sigma_h t}{B - 2t}
\]  
(8)
\[
\sigma_h = \beta f_y
\]  
(9)

where \( \sigma_h \) is the hoop stress of the steel tube corresponding to the peak load of confined concrete columns; \( B \) and \( t \) are the width and thickness of square steel tube, respectively; \( \beta \) is a reduction factor related to the yielding strength of steel \( f_y \). Previous studies [39, 40] proposed a similar prediction model and suggested the factor \( \beta \), which is influenced by the width-thickness ratio of steel tube ranging from 50 to 100. However, based on the test results in this study, an average value of 0.62 was taken for the simplification of the calculations.

(2) For FRP-confined concrete

Based on the model proposed by Lam and Teng [42], the ultimate strength of square FRP-confined concrete \( f_{CF} \) is suggested as
\[
f_{CF} = f_{co} \left[ 1 + k_1 k_{s1} \left( \frac{f_{r,FRP}}{f_{co}} \right) \right]
\]  
(10)

In this equation, \( f_{r,FRP} \) is the confining pressure provided by FRP wrap to an equivalent circular column [42], and the confinement effectiveness coefficient \( k_1 = 3.3 \), same as defined in Lam and Teng model [43] for uniformly confined concrete. Referring to Ref. [42], \( k_{s1} \) is defined as a shape factor calculated as
\[
k_{s1} = 1 - \frac{2 (B_0 - 2R)^2}{3B_0^2 - (4 - \pi)R^2}
\]  
(11)

where \( R \) is the corner radius of inner concrete. Referring to the literature [38, 44], the confinement effectiveness is reduced at the corner of concrete [45]. Therefore, the confining pressure of FRP to concrete \( f_{r,FRP} \) is expressed as
where \( n \) is the number of layers of FRP wrap; \( D \) is an equivalent diameter which is taken as \( \sqrt{2}B_0 \) in this paper; \( t_{frp} \) is the thickness of FRP wrap; \( E_{frp} \) and \( \varepsilon_{rup} \) are the elastic modulus and the hoop rupture strain of FRP wrap. Referring to the method introduced by Hadi et al. [44], a corner-effect coefficient \( k_c \) was introduced to reduce the stronger confining stress at the corner. The factor was defined as the ratio of the sum of the corner length to the sectional perimeter and given as

\[
k_c = \frac{\pi R}{2B_0 - (4 - \pi)R}
\]

Besides, to consider the effect of the large curvature of the corners on FRP wrap leading to a stress concentration of the FRP wrap, the reduction factor \( k_r \) is introduced. Based on the literature [45], the factor is taken as

\[
k_r = \left(1 - 0.2121 \times \frac{\sqrt{2}R}{2B_0} + 0.2121 \times \frac{\sqrt{2}}{2}\right)
\]

The FRP efficiency factor \( (k_e) \) is defined as the ratio of recorded hoop rupture strain of FRP \( (\varepsilon_{rup}) \) to the ultimate tensile strain of FRP obtained from flat coupon tests \( (\varepsilon_{frp}) \), which is shown in Eq. (15) and taken as 0.33 based on the test results of the study.

\[
k_e = \frac{\varepsilon_{rup}}{\varepsilon_{frp}}
\]

(3) For FRP-steel tube confined concrete

The steel tube confinement is generally regarded as an active confinement because the confining pressure provided by steel tube almost remains constant after the yielding of steel tube. On contrast, the FRP confinement is generally considered as a passive confinement because the confining pressure provided by FRP wrap increases continuously with the lateral dilation of concrete. Therefore, the FRP-steel composite confinement might be a confinement type between active confinement and passive confinement. Theoretically, the steel tube-FRP composite confinement in the study can be regarded as one integral confinement since the two confining materials are well bonded based on the tests in the study. However, up to now the theoretical model of FRP-steel composite confined concrete is not researched well. In the present study, a simplified superposition calculation method was used based on the understanding of steel-confined concrete and FRP-confined concrete. As a start, the simplified method is relatively rough but easier to be understood by structural engineers.

Based on the superposition principle, the ultimate strength of square FRP-steel tube confined
Concretes can be calculated as a total strength consisting of the contribution components of FRP wrap and steel tube, which is given as

\[ f_{\text{CFS}} = f_{\text{co}} \left[ 1 + \left( 2.254 \sqrt{1 + \frac{7.94f_{rs}}{f_{\text{co}}} - 2f_{rs} - 2.254} \right) + k_1k_{s1} \left( \frac{f_{r,\text{FRP}}}{f_{\text{co}}} \right) \right] \]  

(16)

Taking Eqs. (7) and (16) into Eq. (5), the axial bearing capacities of steel tube confined concrete stub columns and FRP-steel tube confined concrete stub columns are expressed as

\[ N_u = \begin{cases} f_{\text{co}} \left[ 1 + \left( 2.254 \sqrt{1 + \frac{7.94f_{rs}}{f_{\text{co}}} - 2f_{rs} - 2.254} \right) A_c + f_aA_d \right] \\
\left( f_{\text{co}} \left[ 1 + \left( 2.254 \sqrt{1 + \frac{7.94f_{rs}}{f_{\text{co}}} - 2f_{rs} - 2.254} \right) + k_1k_{s1} \left( \frac{f_{r,\text{FRP}}}{f_{\text{co}}} \right) \right] A_c + f_aA_d \right) 
\end{cases} \]  

(17)

Fig. 16 compares the prediction results of proposed model with the experimental results in this study. Regardless of the confinement types, the proposed model evaluates the ultimate bearing capacities of these confined plain concrete and RC columns with a great agreement.

In addition to the axial bearing capacity, ultimate axial strain of composite stub columns is a very important parameter. For square STCC specimens, as shown in Table 3, the strain capacity increases with the thickness of steel tube because a thicker steel tube usually can provide a larger confinement to concrete core. Moreover, the installation of longitudinal reinforcements also can improve strain capacity. For square FRP-STCC specimens, the strain capacity generally increases with the thickness of steel tube, the number of layers of FRP wrap and the installation of longitudinal reinforcements. Therefore, the confinements from steel tube and FRP wrap as well as the advantageous effects of longitudinal reinforcement should be considered when predicting the strain capacities of square STCC stub columns and square FRP-STCC columns, which is expected to be studied in the future.
6. Concluding remarks

This paper presented an experimental study to understand the monotonic axial compressive behaviour and confinement mechanism of square CFRP-steel tube confined concretes. The confinement from steel tube and CFRP wrap enhances the ultimate strength and ductility of core concrete. CFRP wrapping effectively constrains the deformation of steel tube, which delays its outward local buckling and constrains the continuous dilation of core concrete at the stage of large deformation. Based on this study, the following conclusions can be drawn:

1. The CFRP-steel tube confinement is highly effective in improving the bearing capacity and ductility of concrete columns, especially for plain concrete. The number of layers of CFRP wrap has a significant effect on the failure of the confined reinforced concrete columns. The width-to-thickness ratio of the steel tube is also a key factor affecting the axial bearing capacity of confined concrete columns.

2. The post-peak softening phenomenon of square confined concretes was observed in the specimens. However, the softening degree of the columns was improved by using a thicker CFRP wrap. The effect of the CFRP wrap is more pronounced for the CFRP-steel tube confined concrete columns with a larger width-to-thickness ratio of steel tube.

3. Through a detailed stress analysis, the stress-strain curves of the concrete core confined by composite action of steel tube and CFRP wrap were provided. The mechanical properties of the concrete core was greatly improved by the composite confinement. The study explained the confinement mechanism of the steel tube and the FRP wrap in confined plain or reinforced concrete columns, and the role of steel tube and CFRP wrap in each load stage, which provides a basis for the establishment of a calculation model of the bearing capacity for the columns. The three stages of the confinement mechanism include a steel tube confinement stage which is similar to steel tube confined concrete, and a CFRP-steel tube co-confinement stage in which the total confinement pressure increases rapidly due to the effective co-confinement from steel tube and CFRP wrap, and a FRP-dominated confinement increasing stage when FRP wrap keeps an effective confinement to steel tube and concrete core to resist axial compressive load.

4. Based on previous studies and discussion on the strength models for confined concrete, through a superposition principle considering the confinement of steel tube and CFRP wrap, this paper proposed a simplified calculation model to predict the axial bearing capacity of CFRP-steel tube confined plain concrete and reinforced concrete stub columns. Comparing with test results, the accuracy and
reliability of proposed model was confirmed.

Compared with CFRP, GFRP wrap may be more suitable to work together with the steel tube than CFRP in FRP-STCC elements, because of GFRP materials’ low cost, greater fracture strain. The potential galvanic corrosion issues also will be eliminated. In the future, the axial compressive behaviour of GFRP-STCC elements will be investigated.

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Notation

- $A_a$: cross-sectional area of longitudinal reinforcement
- $A_c$: cross-sectional area of concrete core
- $A_s$: cross-sectional areas of steel tube
- $B$: width of steel tube
- $B_0$: width of concrete core
- $D$: equivalent diameter
- $E_{frp}$: elastic modulus of FRP
- $E_s$: elastic modulus of steel
- $E_s^t$: tangent modulus steel in the elastoplastic stage
- $H$: height of the specimen
- $f_a$: yield strength of longitudinal reinforcement
- $f_y$: yield strength of steel tube
- $f_{yr}$: proportional limit of steel tube
- $f_{co}$: compressive strength of unconfined concrete
- $f_r$: confining pressure
- $f_{rs}$: confining pressure provided by steel tube
- $f_{r,FRP}$: confining pressure provided by FRP wrap
- $f_{CF}$: compressive strength of FRP-confined concrete
- $f_{CS}$: compressive strength of steel tube confined concrete
- $f_{CFS}$: compressive strength of FRP-steel tube confined concrete
- $f_{cc1}$: first peak stress of confined concrete
- $f_{cc2}$: ultimate stress of confined concrete corresponding to the rupture of FRP wrap
- $G$: shear modulus of the steel
- $k_1$: confinement effectiveness coefficient
- $k_{s1}$: shape factor
- $k_c$: corner-effect coefficient
669  \( k_r \): reduction factor considering stress concentration at corner
670  \( k_e \): FRP efficiency factor
671  \( n \): the number of FRP layer
672  \( N \): axial load resisted by the composite column
673  \( N_{ux} \): axial bearing capacity of the composite column
674  \( R \): corner radius
675  \( t \): thickness of steel tube
676  \( t_{frp} \): thickness of FRP wrap
677  \( \beta \): reduction factor
678  \( \mu_s \): Poisson’s ratio of steel in the elastic stage
679  \( \mu_{sp} \): Poisson’s ratio of steel in the elastoplastic stage
680  \( \sigma_h \): hoop stress of steel tube
681  \( \sigma_v \): axial stress of steel tube
682  \( \sigma_c \): axial stress of confined concrete
683  \( \sigma_{h,j} \): hoop stress of a confining jacket
684  \( \sigma_z \): equivalent stress of steel tube
685  \( \varepsilon_p \): equivalent strain of steel tube corresponding to \( f_p \)
686  \( \varepsilon_y \): equivalent strain of steel tube corresponding to \( f_y \)
687  \( \varepsilon_h \): hoop strain of steel tube
688  \( \varepsilon_v \): axial strain of steel tube
689  \( \varepsilon_{frp} \): ultimate tensile strain of FRP coupon
690  \( \varepsilon_{h,rup} \): hoop rupture strain of FRP wrap
691  \( \varepsilon_{cc1} \): nominal axial strain of confined concrete corresponding to \( f_{cc1} \)
692  \( \varepsilon_{cc2} \): nominal axial strain of confined concrete corresponding to \( f_{cc2} \)

694  References


