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# Load transfer mechanism in concrete-filled steel tubular columns: developments, challenges and opportunities

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**Abstract:** This article provides an in-depth review on the understanding of load transfer mechanism in concrete-filled steel tubular (CFST) columns, with emphasis on tube-concrete interface behaviour, secondly, load transfer to encased concrete by bond strength, and lastly, load transfer by blind-bolt bearing. This article summarizes the diverse observations on obtaining a reliable bond strength between infill concrete and steel tube of CFSTs, as this is the key component that ensures the load transfer from the beam to the column. After assessing the bond strength, the identification of the zone for load introduction from fin-plate connection to CFST column is pertinent, as it is assumed to occur through bond strength via an introduction length, and studies on this aspect have been reviewed in detail. The review further extends to understanding of load transfer through bearing mechanism of blind-bolts, and provides an overview of the current developments in blind-bolted connections, which might overcome some of the challenges related to uncertainties in load transfer through bond strength. The article aims to highlight the authors' reflections on exploring and connecting the above-stated three sub-topics related to load transfer in CFST columns, discussing the challenges, and concludes by presenting the avenues for futue research.

**Keywords:** CFST, load introduction, bond strength, bearing, blind bolted connections, review

## 1. Introduction

The concrete-filled steel tubular (CFST) members have been widely adopted in the construction industry because of its performance like high strength, ductility, and its aesthetic appeal. The advantages of CFST includes its ability to carry high loads even in post-yielding stage, having better fire resistance, and high stability under various dynamic loads [1]. As a result, these composite members have been adopted even in countries that have active seismic zones like Australia, China, Japan and the United States. Apart from these advantages, the non-requirement of form works for its fabrication can significantly reduce the construction cost. In the literature, a significant amount of investigation on CFSTs can be found. Apart from investigations on its axial compressive performance, a wide range of experimental testing programmes for CFST columns have been conducted that studied its behaviour under cyclic or seismic loads, and eccentric tension [2-8]. Recently, in addition to the regular geometrical shapes like rectangular and circular CFSTs, tubes of polygonal geometries, like hexagonal and octagonal CFSTs have also been studied to a considerable extent [9-12]. Apart from these tests, experimental investigations on bond-strength and slip are also available in the literature [13, 14].

The behaviour of CFST member is usually dependent on the composite action of the surrounding steel tube and the infilled concrete core. To achieve this composite action, it is important that the load gets distributed in both the steel tube and concrete core, and this can be easily achieved during the direct axial load tests that are carried out on isolated CFST stub columns, and can be referred from existing literature [15-18]. As when the load is applied directly on the top of the CFST stub column, the force is applied simultaneously on the tube and concrete core, and as a result, composite action is achieved, as shown in Fig. 1. But, in a realistic condition the load gets introduced to the column via the beam connection, and a part of the load is introduced via the top. These beam-to-CFST column connections can be either welded fin-plate connections, or bolted end-plate connections. In welded connections, the steel I-beam is connected via a fin-plate, which is welded to the column steel tube only, and the transfer of the load from the steel tube to the concrete core of the CFST column is primarily dependent on the bond strength between the concrete and steel tube. It is therefore, pertinent to study the bond strength between the concrete surface and the steel tube of the CFST column.

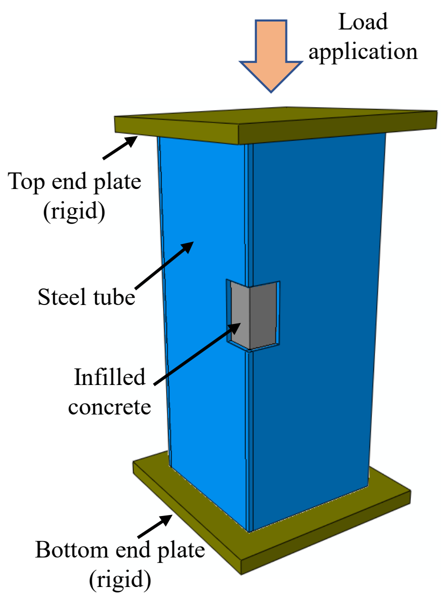
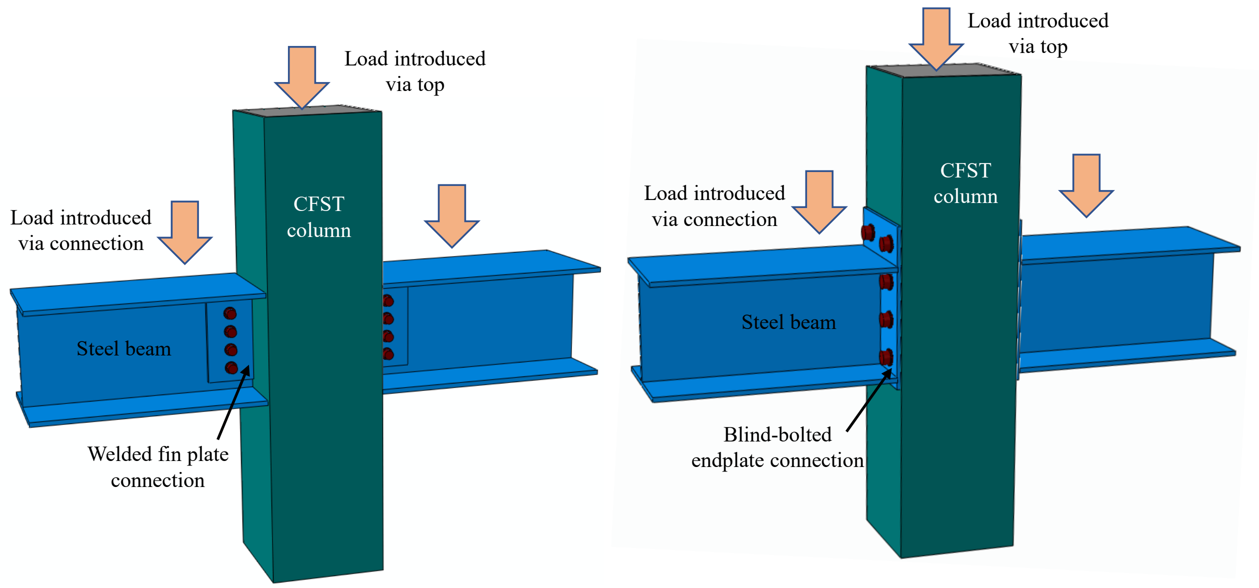



Fig.1: Axial loading of CFST stub columns.

Secondly, in a welded fin-plate connection between the steel beam and CFST column, the load is transferred through the bond strength within an "introduction length", based on which the failure load of the CFST column is determined. Therefore, in this paper detailed review on load transfer via bond strength mechanism in CFST column is presented. A typical beam-to-CFST column having welded fin-plate connection is presented in Fig. 2 (a). Lastly, in bolted beam-to-CFST column connections, blind-bolts are usually adopted as the fabrication is easy without accessing the inner face of the steel tube, and can be tightened with the required torque. Also, in blind-bolted connections, the bolts can fully penetrate the infill concrete and thus help in increasing the strength and stiffness of the connection [19, 20]. That is, the transfer of the load may not fully be dependent on bond strength between the steel tube and concrete surface, and a considerable amount of load might be transferred to the core by bolt shank bearing. Thus, in this article, a systematic review of the load transfer by bearing mechanism of blind-bolt in CFST column is conducted, and thereby highlights the scope of research for enhanced composite performance. The blind-bolted beam-to-CFST column connection with actual loading conditions is represented in Fig. 2 (b).




a) Welded connection: load application via top and connection

(b) Blind-bolted connection: load application via top and connection

Fig. 2: Welded fin-plate and bolted end-plate beam to CFST column connections.

The significance of this review lies in the fact that, though a number of excellent state-of-the-art articles on axial compressive behaviour of CFST columns can be found [6, 21-23], including review on stiffened CFST [24], fire and impact load performance [25, 26], but reviews on interfacial bond behaviour, load transfer through bond strength and blind-bolt bearing mechanism are rare. Few reviews on blind-bolted column connections are available in the literature [27, 28], but they are based on bare steel tubes, and the recent review by Cabrera et al. [29] is specifically based on anchored hollo-bolted connections. Due to diverse observations by researchers on interfacial bond behaviour, insignificant attention and scarce review on load transfer mechanism via bond strength and blind-bolt bearing necessitates the discussion and critical analysis of these areas. Therefore, this paper will attempt to focus on the aforementioned areas, and will drive to figure out the deficiencies in the existing work, and curve out the need for further research in the area.

## 2. Definitions and scope

The *composite action* in CFST columns refers to continuity of strain in both steel and concrete components when they are simultaneously loaded at the end of the column [30], as shown in Fig.1, and is usually conducted to determine the cross-section resistance. In a CFST column, the *bond strength* is the shear strength which is developed between the concrete and the steel section, which is free from oil, grease and loose rust [31] and is prominent when longitudinal shearing stress is very high. It is of prime importance, in particular, in regions of beam-to-CFST column welded connections, where the external loading is transferred to the column by bond strength between the inner surface of steel tube and infill concrete. To determine the bond strength, *push-out* tests are conducted, where the load is applied only to the concrete core of the CFST column at the top, and the *slip* is measured by a transducer at the bottom, as shown in Fig. 3. Now in CFST columns, the *load introduction length* is the region via which the shear loading or the reaction force from the beam connections is introduced into the concrete core of the column. In connections, the *welded fin plate connections* are those where the steel beam is connected to the CFST column via a fin plate, and the load is usually transferred to the concrete core by bond strength mechanism. Whereas in a *blind-bolted connection*, the steel beam is connected to an endplate, through which the blind-bolts penetrate the concrete core of the column, and thus the load is transferred to the core by bearing mechanism.

As previously discussed, though there are some existing reviews on CFST axial compressive behaviour, performance with internal or external stiffeners, and thus, are also out of the scope of this paper. The bond-strength performance with various stiffeners, bond strength after exposure to fire, and CFST columns with stainless steel tube are out of the scope of this review. However, findings on bond-strength behaviour of CFST columns using recycled aggregate, expansive concrete, high-strength concrete, and concrete with manufactured sand are included in this review as they are increasingly being adopted as infill materials in CFSTs. Secondly, the observations on load transfer in welded shear connections by bond strength are reviewed. Lastly, load transfer by bearing of blind-bolts in CFST column, types of blind-bolted connection assemblies, and the influence of important parameters and loading patterns are reflected upon. Since direct bearing of blind-bolts to CFST column is the focus at the later part of this review, therefore blind-bolted connections with channel sections, concrete-filled double steel tubular columns, columns with other internal arrangements, stainless steel CFST, etc. are out of the scope of this article. The authors have experience in working with blind-bolted CFST connections, whereas the second and third authors have extensive experience in studying load-transfer mechanisms in CFSTs, the behaviour of CFSTs under different loading conditions, welded connections in concrete-filled tubes, and thus have highlighted the reflections on the challenges and future needs in the field.

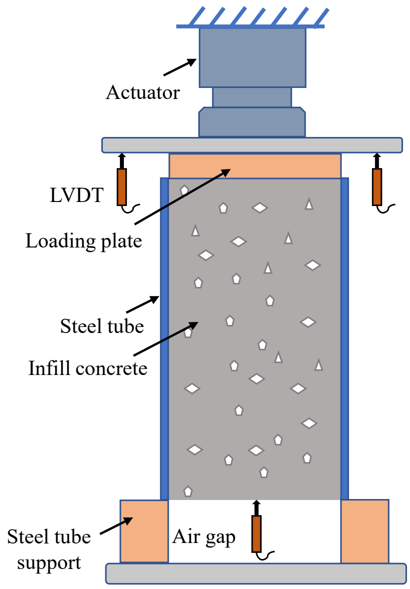



Fig. 3: Setup of push-out test for CFST column.

## 3. Bond-strength and slip in CFST columns

### *3.1. Discussions on research findings*

The contribution of bond strength and the slip resistance of concrete in steel tubes is considered the most vital for shear load transfer in CFST column. Other than the significance of cross-section type and the roughness of the tube, the steel tube internal interface needs to be unpainted and free from oil, grease and rust, and no other factors affecting bond strength is accounted for in EN 1994-1-1 (2004) [31]. The representative experimental photos during and after the test are presented in Fig. 4. The major factors influencing the bond strength are discussed in the following sections.


(a) Application of load (b) Concrete-steel interface (c) Slip

Fig. 4: Bond strength testing procedure, reproduced from [32].

### *3.1.1 Steel-concrete interface*

The research on bond strengths can be traced back to 1967 [33], which identifies three different characteristics of shear resistance between steel tube and concrete, namely chemical bond, fictional and mechanical resistance. A chemical bond refers to the adherence of the cement paste to the steel surface, and breaking such a bond requires excessive relative displacements. Secondly, the frictional resistance refers to the interface pressure between steel and concrete, and thirdly, the mechanical resistance is a physical interlocking of the concrete and steel tube, which may be attributed to the surface roughness. The investigation by Roeder et al. [34] identifies more explicitly three interface conditions between the tube and the concrete, which depend upon the radial enlargement of CFST ($\Delta_1$), radial reduction due to the shrinkage of concrete ($\Delta_2$) and amplitude of rugosity of the interior of the tube ($\Delta_3$). The possible interface conditions are:

$$\text{A: } \Delta_1 + \Delta_2 > 0 \tag{1}$$

$$\text{B: } \Delta_1 + \Delta_2 < -\Delta_3 \tag{2}$$

$$\text{C: } 0 \geq \Delta_1 + \Delta_2 \geq -\Delta_3 \tag{3}$$

The interface conditions of most CFST columns tend to be in state B or C, and state A cannot generally be achieved. During the investigation, there was 0 slip as the force application increases, but it drops after a point where the slip in concrete starts, and as a result, the load carrying capacity decreases. Refer to Fig. 5, the curve with solid square data points, where a clear separation occurs in the curve at the ultimate load capacity with an associated amount of slip and simultaneously resulting in decreasing resistance. At around 140 kN is the breakaway point of the initial contact between the tube and concrete and marks the beginning of sliding frictional resistance. This initial transition occurred at loads from 40% to 80% of the ultimate load which was achieved at slip displacements of around 0.25 mm. The slip displacements were between 20 and 200 times larger than the initial breakaway slip.

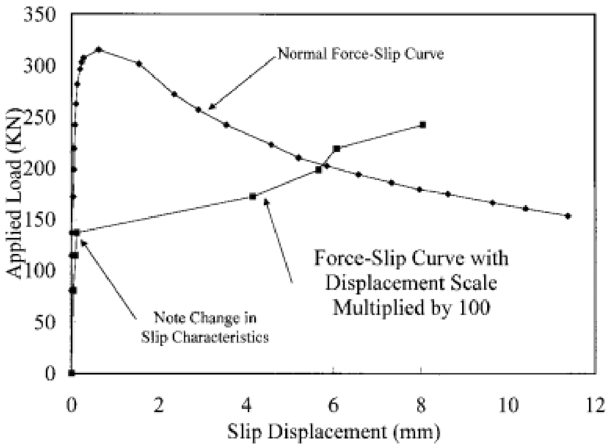



Fig. 5: Typical load-displacement (slip) curve [34]

The importance of imperfection and interlocking in the concrete and tube have also been acknowledged by Virdi et al. [35]. The roughness of the steel or the micro locking contributes to the ultimate bond strength and can be related to the initial stiff region of the load-deformation curve. The specific motion of the breaking of this bond was observed when the concrete interface attained a local strain of 0.0035 related to the compressive crushing of concrete. More works by Shakir-khalil [36] carried 40 push-out tests having variations in shapes and sizes and investigated the slip behaviour with and without studs within the tube, with and without oil in the steel-concrete interface. The experiment observed a possible chance for a reduction in average bond strength due to a larger cross-section, which results in shrinkage, as also stated by Wium [37]. The sensitivity of the interface was well gauged from comparative results where the bond strength was reduced to half in the case of an "oiled" steel-concrete interface as compared to the "dry" steel-concrete interface. The experimental program by Qu et al. [38] investigated a total of 18 columns having different interface conditions and observed the effect of the lubricated interface has resulted in slips at very low load levels, as noted by previous researchers. Qu et al. [39] conducted load-reversed push-out tests on six rectangular shaped CFST specimens to investigate the nature of the bond and the contribution of chemical adhesion, microlocking, and macroclocking. Macrolocking was observed to be the dominant mechanism, followed by microloacking and chemical adhesion. It is noteworthy that the conventional push-out test does not fully resemble the real scenario where shear loading comes from the beam connections in a joint region. A modified push-out test was carried out by Nardin et al. [40] involving a connection with studs and angles but did not present a comparison with normal push-out tests and made no commentary on bond strength increase, if any.

#### *3.1.2 Influence of concrete strength and age, section geometry, aggregate type and interface length*

The works of Wium et al. [37, 41] have identified the importance of concrete shrinkage due to ageing in the bond strength of CFST columns. Within the column, due to shrinkage in concrete, there is a reduction in transfer of forces from the tube to the concrete core. Supporting the findings of Virdi et al. [35], experiments by Shakir-Khalil [36] revealed that circular sections were much more effective in resisting push-out tests as compared to square and rectangular sections. The bond strength values of 2 types of rectangular hollow section (RHS) columns achieved in this experiment were 0.82 N/mm$^2$ and 0.44 N/mm$^2$, whereas the BS5400, Eurocode EN 1994-1-1 [31], Australian code AS5100.6-2004 [42], and North American code ANSI/AISC 360-16 [43] provide a value of 0.4 N/mm$^2$. The dimensions were as 120×80×5 mm and 150×150×5 mm for 0.82 N/mm$^2$ and 0.44 N/mm$^2$, respectively, and the reason was attributed to the section size, where the larger dimension column could have a higher possibility of shrinkage and thus a lesser bond strength.

Investigations by Roeder et al. [34] had also identified 104 circular and 49 rectangular CFST tests, which were carried out by previous researchers to investigate the effect of concrete strength and section geometry, and concluded that the average bond stress of rectangular tubes was 70% lesser than the average of circular tubes. Also, the concrete compressive strength had non-consistent effect on the bond strength capacity. The effect of diameter size of tubes along with diameter to thickness (*d/t*) ratio was also examined and concluded that with larger diameters and larger *d/t* ratios, there are more possibilities for shrinkage, and can lead to lesser bond capacities. Experiments by Roeder et al. and Tao et al. [34, 44] have observed that the bond strength decreases with an increase in slenderness for circular hollow sections. In experiments of Tao et al. and Song et al. [13, 45], the bond strength for circular columns was 0.60 N/mm$^2$ which was reduced to 0.04 N/mm$^2$ when measured after 1165 days, whereas, with the use of expansive concrete, the value achieved was 1.02 N/mm$^2$ and reduced to 0.76 N/mm$^2$ at 1168 days. But in the case of square columns, the effect of expansive concrete did not seem very considerable and suggested to use of expansive concrete for circular columns up to a diameter of 400mm. In the tests conducted by [46] with both normal and expansive concrete with square CFST specimens having a dimension of 160 mm observed, an increase in bond strength by at least 30% compared to specimens using normal concrete. This indicates that the size of the CFST tube can be crucial for bond strength, with smaller dimension tubes displaying the beneficial effect of expansive concrete. Investigation with lightweight concrete was conducted by Mouli et al. [47], and observed improved bond strength as compared to normal weight concrete. In the experiment by Virdi et al. [35], it was observed that the ultimate bond strength is not varied with the length of the concrete-steel interface, diameter and thickness of the steel tube, and concrete strength.

The importance of a reliable bond-slip constitutive relationship for CFSTs for developing reliable numerical models is very necessary, whereas the only way to attain this is by considering the average bond stress slip relationship as the basis of a constitutive relationship. But again, the distribution of bond stress is not uniform along the length of the column. In this regard, Yin et al. [48] have carried out an investigation to obtain an equation with the help of position function that can represent the average bond stress-free end slip with influences of column diameter and tensile strength of concrete that can truly represent the local bond stress-slip behaviour. As evidenced by Fig. 6, the bond stress-slip curve represents three stages, viz, the non-slip stage, ascending stage, and the residual stage. A theoretical model was proposed to define the stress-slip behaviour for simplification, as shown in Fig. 7.

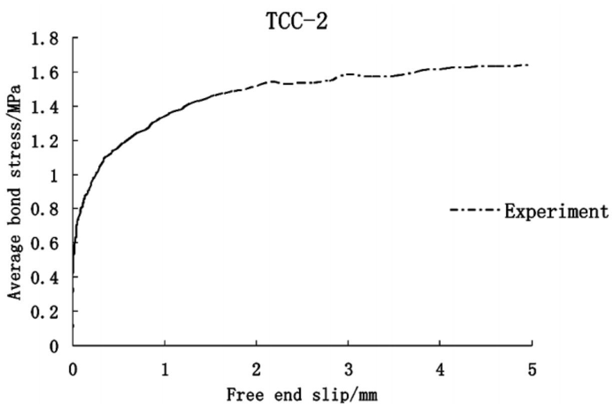



Fig. 6: Average bond stress-free end slip [48].

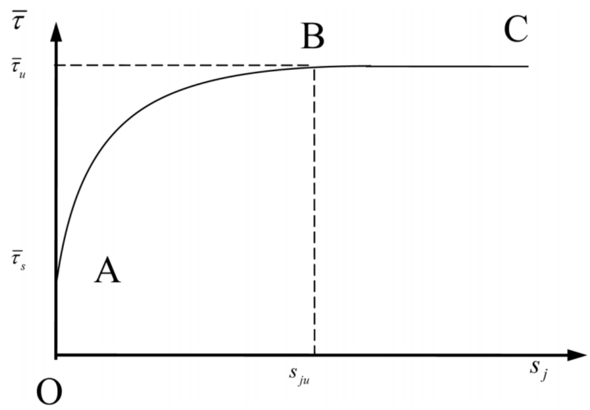



Fig. 7: Theoretical model for average bond stress and slip curve [48].

The non-slip stage (OA) represents no slippage, and nonlinear behaviour is identified (AB). In the residual stage (BC), there is a sharp increase in slip without any load increment; thus, the residual push-out load depends on the friction, and the mechanical interlock is already broken. An average bond stress-slip curve has been suggested by Yin et al. [48] based on this constitutive model. But again, as the average bond stress-slip curve did not represent the local bond stress-slip, the research group studied several bond stress distribution patterns and bond slip distribution patterns (refer to Fig. 8) and used position functions to attain another bond stress-slip relationship that can represent the local bond stress-slip relationship. As mentioned by the research group [38] that also investigated the influence of concrete strength, cross-sectional dimensions, and interface lengths, observed that the bond stress distribution is not uniform throughout the length of the column. But the bond stress distribution along the specimen length is not identical as observed by [48], as there were numerous fluctuations in the stress distribution at the middle length of the specimen, as can be seen in Fig. 9.

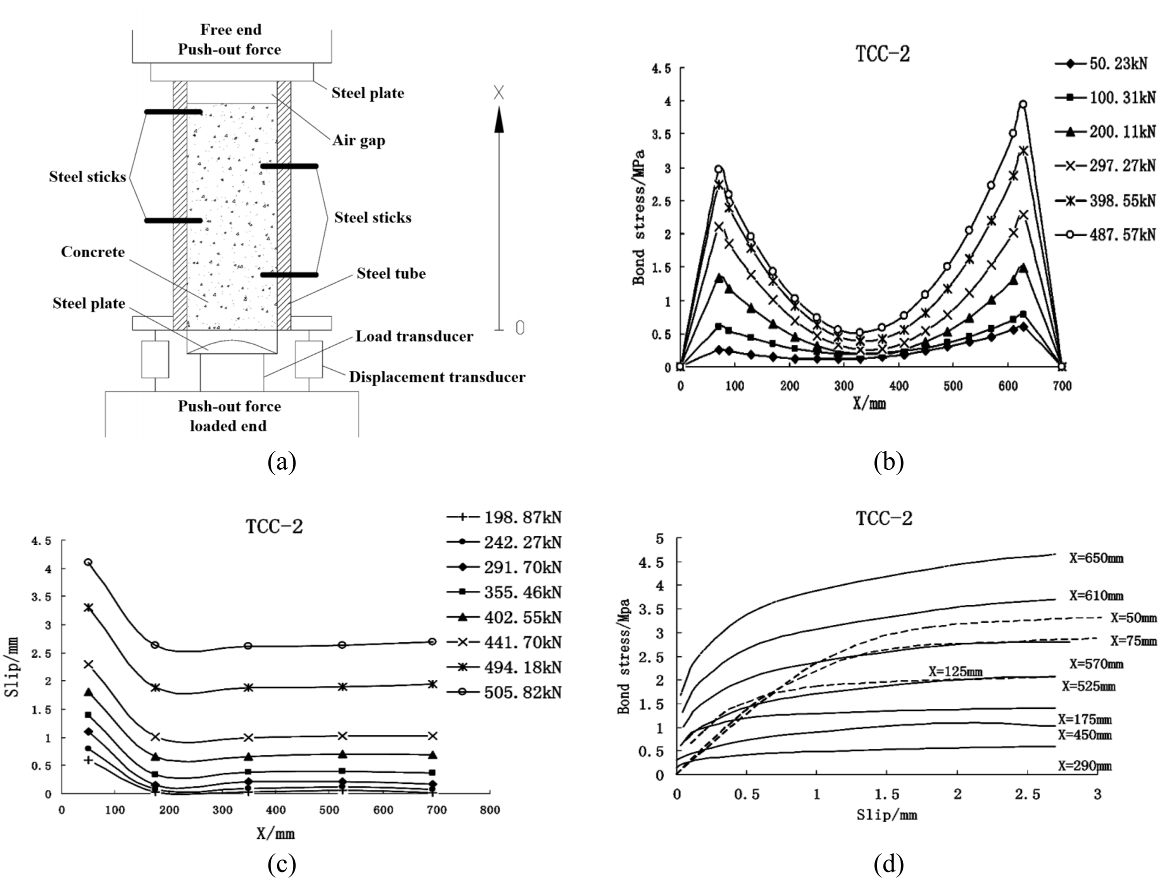



**Fig. 8.** (a) Test setup; (b) bond stress distribution under different loading; (c) relative slip curves under different loading levels; and (d) bond stress-slip relationships of different positions. Reproduced from [48].

The findings of the experiment also include that there is no effect of length of the interface on the bond strength, and an increasing of concrete grade corresponds to increased bond strength. The test by [38] also observed that the bond strength is insensitive to the tube cross-section slenderness for rectangular hollow sections, which is contrary to the findings of [34, 44] for circular CFST columns. Other researchers also observed a negligible effect of tube interface length on bond strength in rectangular CFST specimens [39] and circular CFST specimens [35], thus confirming that interface length is not a dominant factor for bond strength. Extensive tests have also been conducted by Lyu et al. [49] to determine the bond strength using recycled aggregate concrete, and observed cross-section type and cross-section dimensions as the influential primary factors, and similar bond strength was achieved for both normal and recycled aggregate concrete. Study by Yang et al. [50] on recycled aggregate observed that shrinkage and creep strains were 6% to 23% higher than normal aggregate CFSTs. Tests with manufactured sand from limestone and pebbles were also conducted by Guan et al. [51] to determine its influence in bond behaviour, and observed higher bond strength than conventional CFST columns, decrease in *d/t* resulted in higher bond strength, but the concrete compressive strength did not have a consistent influence on the CFST bond strength.

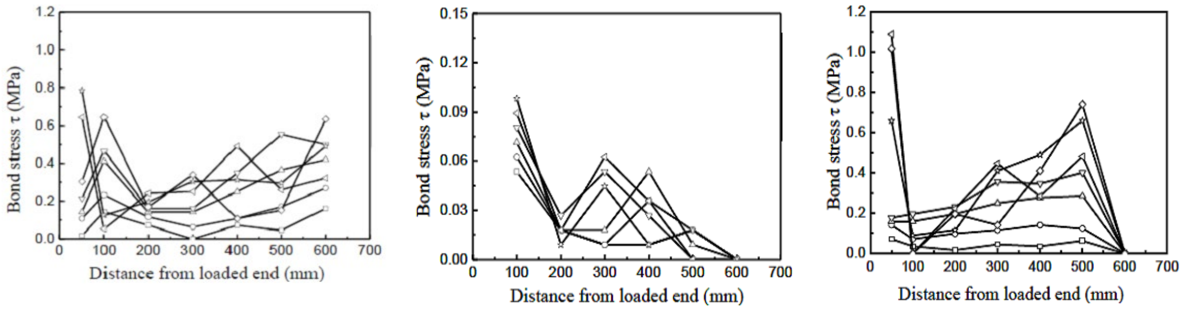



Fig. 9: Bond stress distribution along specimen length [38].

The works of Qu et al. [38] considered a total of 28 data sets comprising the experimental data from other researchers [36, 44, 52] to determine an empirical equation for bond strength for RHS columns. The earlier researchers, including Xu et al. and Cai [53, 54], have also proposed empirical equations based on concrete compressive strength, where bond strength increases with increasing concrete strength, but did not consider the effect of column section sizes. On the other hand, Roeder et al. [34] proposed an equation considering only column cross-section, as the concrete strength does not have a pronounced effect, as discussed earlier. But as both concrete strength and column dimension is expected to have certain effects on the bond strength of the CFST, the empirical formula by Qu et al. [38] tries to address both dimensions. Following a regression analysis, Qu et al. [38] proposed an equation but is valid only for normal concrete strengths up to 55MPa. Empirical equations proposed by researchers are presented in Table 1 for ready reference.

Finally, the authors in this article have compiled a total of 279 test results [13, 32, 34-36, 38, 39, 46, 47, 49, 51, 53], of which 203 test data are for circular CFST, and 76 test data are for square/rectangular CFST columns. The test results have been compiled based on a geometric cross-section as presented in Fig. 10 (a) and (b).

Table 1: Empirical equations to determine bond strength proposed by researchers.

| Sl. no. | Author | CFST section | Proposed empirical equation | Remarks |
|---|---|---|---|---|
| 1 | Roeder et al. [34] | Circular | $\tau_u = 2.109 - 0.026\left(\frac{d}{t}\right)$ N/mm$^2$<br>$d$: column diameter; $t$: tube thickness | $d/t$ no greater than 80 |
| 2 | Qu et al. [38] | Rectangular | $\tau_u = 0.082\sqrt{f_{cu}} - 0.00105D$ N/mm$^2$<br>$D$: column depth; $f_{cu}$: concrete cube strength | Valid up to concrete strength of 55MPa |
| 3 | Lyu et al. [49] | Circular | $\tau_u = 0.24 + 177\left(\frac{1}{D}\right)$ N/mm$^2$<br>$\tau_u = 0.071 + 4900\left(\frac{t}{D^2}\right)$ N/mm$^2$ | Tests on recycled and normal aggregate concrete. |
| | | Square | $\tau_u = -0.44 + 58\left(\frac{1}{B}\right)$ N/mm$^2$<br>$\tau_u = 0.043 + 1100\left(\frac{t}{B^2}\right)$ N/mm$^2$<br>$d$: diameter; $t$: tube thickness; $B$: width | |
| 4 | Xu et al. [53] | Circular | $\tau_u = 0.1 \times f_c^{0.44} \times \left(1 + 9.2 \times \left({}^{q}/{}_{f_c}\right)^{0.62}\right)$ N/mm$^2$<br>$q$: radial prestress; $f_c$: compressive strength of concrete | |
| | | | $\tau_u = 0.1 \times f_c^{0.44}$ N/mm$^2$ | Without radial pre-stress |
| 5 | Cai et al. [54] | Circular | $\tau_u = 0.1 \times f_c^{0.4}$ N/mm$^2$<br>$f_c$: compressive strength of concrete | |
| 6 | Parsley et al. [55] | Square | $\tau_u = 1.9 + 10{,}000\left(\frac{t}{B^2}\right)$ (psi)<br>$t$: tube thickness; $B$: width | |
| 7 | Qu et al. [32] | Square | $\tau_u = \left(0.71 - 0.01 \times \frac{D}{t} + 0.0002 \times \sqrt{f_{cu}}\right) \times \left(\frac{2.781}{1+e^{-0.333\times(K-1.75)}}\right)$ N/mm$^2$ | Based on self-compacting concrete, cold-form steel tube |
| | | Rectangle | $\tau_u = \left(-0.216 - 0.011 \times \frac{D}{t} + 0.17\sqrt{f_{cu}}\right) \times \left(\frac{2.716}{1+e^{-0.14\times(K-3.83)}}\right)$ N/mm$^2$<br>$K$: the dosage of the concrete expansive agent; $D$: column depth; $t$: tube thickness; $f_{cu}$: Concrete cube strength | |

As seen for both circular and square/rectangular CFSTs, there is significant variation in bond strength even with similar parameters, but it can be stated that circular CFSTs have higher bond strength compared to the square/rectangular counterparts. In Fig. 10 (a), the trend is very clear for circular CFSTs, indicating the significant decrease in bond strength with increasing steel tube diameter irrespective of the type of concrete aggregate, including recycled aggregate and concrete made from manufactured sand. As can be referred from Fig. 10 (b), showing the influence of column width of rectangular CFSTs, a gradual steady slope of decrease in bond strength with increasing width is obtained, but the drop is not as sharp as in circular CFSTs. When considering the slenderness ratio (*d/t*) of circular CFSTs, the bond strength decreases with increase in slenderness, indicating the direct influence of tube thickness as presented in Fig. 10 (c).

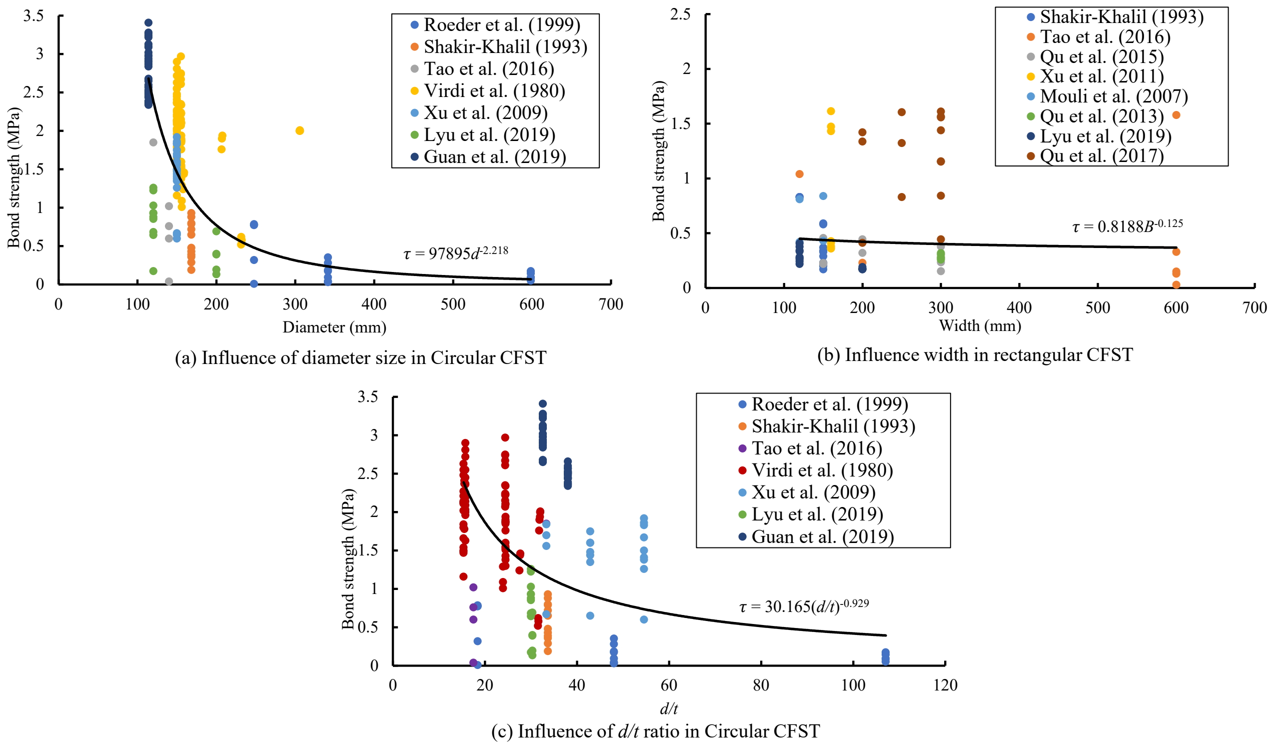
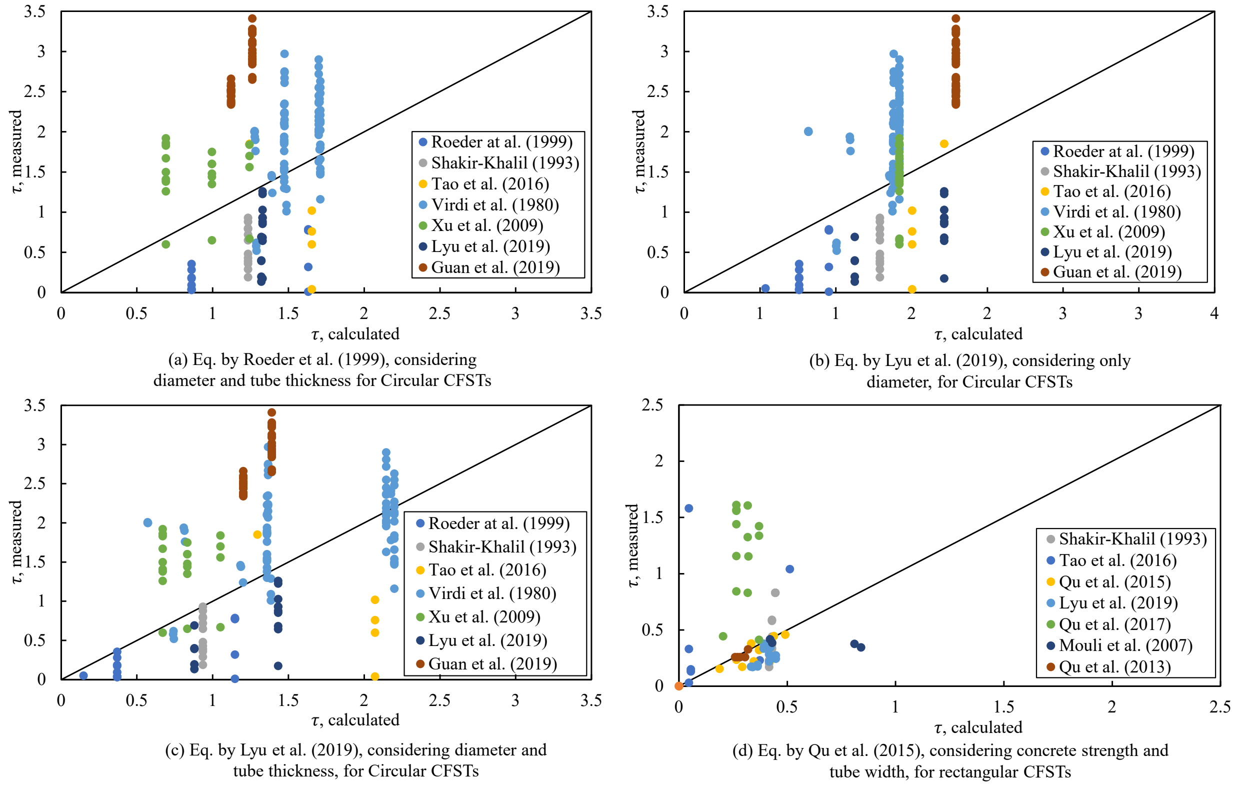



Fig. 10: Influence of cross-section dimension on bond strength.

Further, to examine the applicability of the proposed empirical equations by various researchers, as presented in Table 1, a representative analysis is presented in Fig. 11, where bond strength is calculated ($\tau_{calculated}$) by the proposed equations, and is compared with the bond strength obtained from the test results ($\tau_{measured}$). Fig. 11 (a) presents the comparison with the equation for prediction of bond strength as proposed by Roeder at al. [34], and Figs. 11(b-c) compares with the equation proposed by Lyu et al. [49] for the circular CFSTs. Whereas, Fig.11(d) compares the measured bond strength values with the predicted bond strength values according to the equation proposed by Qu et al. [38] for rectangular CFSTs. As can be seen from Figs. 11 (a-d), with all the existing bond strength prediction equations for both circular and rectangular CFSTs, there exists large scatter in the bond strength values, and thereby indicating that the existing empirical equations might not be able to accurately predict the CFST bond strength.



(a) Eq. by Roeder et al. (1999), considering diameter and tube thickness for Circular CFSTs

(b) Eq. by Lyu et al. (2019), considering only diameter, for Circular CFSTs

(c) Eq. by Lyu et al. (2019), considering diameter and tube thickness, for Circular CFSTs

(d) Eq. by Qu et al. (2015), considering concrete strength and tube width, for rectangular CFSTs

Fig. 11: Evaluation of bond strength prediction equations proposed by various researchers.

Influence of other factors such as combination of sustained loading and corrosion can also lead to much earlier and severe local buckling in CFST columns as studied by Hua *et al.* [56, 57], and thereby can also lead to reduction in bond strength. The influence of long-term sustained loading and corrosion can significantly reduce the ultimate strength and ductility index, including loss of steel wall thickness as was observed by Han *et al.* [58], which can directly influence the bond strength of the CFST. Moreover, due to coupled long-term loading and chloride corrosion, the decrease in wall-thickness of the CFST outer tube can lead to load transfer to concrete core, and shrinkage and creep of concrete could lead to load transfer from concrete to outer tube [59]. Thus, such loading conditions can be detrimental to confinement effect of the CFST columns and the tube-wall to concrete interaction will be affected, which will influence the effective load transfer to the concrete core.

### *3.2. Commentary on findings and potential research gaps*

According to the findings by various research groups and individuals, it may be stated that the bond strength plays an important role in load transfer in CFST columns, but the effectiveness of this strength depends on various parameters. The takeaways from the above-discussed contents can be summarised below:

a. The EC4-1-1 [31] suggests a bond strength of 0.55 $N/mm^2$ and 0.44 $N/mm^2$ for circular and square CFST members, respectively. The Australian standard AS5100.6-2004 [42] does not distinguish between the CFST column cross-sections and suggests a value of 0.4 $N/mm^2$ as the bond strength. Whereas American standard ANSI/AISC 360-16 [43] provides an upper bound value of 1.4 $N/mm^2$ for circular CFSTs and 0.7 $N/mm^2$ for square CFSTs. From the authors' analysis of the compiled 279 CFST test data, it is found that the mean bond strength for circular CFST is 1.35 $N/mm^2$, and for square CFST the mean strength is 0.42 $N/mm^2$. Thus, for circular CFSTs, the American code provides a similar value, whereas for square CFSTs, the European and Australian code provide a closer value.

b. Concrete shrinkage has been given due importance by the researchers as it reduces the bond strength within the tube. The factor of shrinkage is related to the age of concrete, and thus, the average bond strength achieved by the tests at 28 days deteriorates rapidly, which again questions the validity of the use of the obtained bond strength in structural design. A relation of bond strength with concrete age needs to be established to have an optimal design process.
c. Experiments on the effects of concrete strength have been found to have contrasting remarks. Works by research groups like [35] and [34] have found no effect of bond strength due to increasing concrete strength, whereas [38, 53, 54] have found links between increasing bond strength with increasing concrete strength. But, [38] has stressed both strength and cross-section as both factors play a role in the bond strength. For obtaining a general formulation on bond strength based on concrete strength and cross-section, more extensive research needs to be conducted.
d. The better performance of circular columns over rectangular columns is pronounced in the studies. The larger diameter of the column and larger *d/t* ratios can also be detrimental to bond strength. There is also no consensus on the influence of column interface length on the bond strength value.

e. The use of expansive concrete can be highly beneficial in increasing the bond strength. Thus, more experiments involving both effects of column cross-section and expansive concrete can be taken up.
f. The existing empirical equations proposed by various researchers may not accurately predict the bond strength, and a large discreteness is observed when compared with the predicted and experimental values. Therefore, an empirical equation that incorporates more parameters might be suitable to closely predict the bond strength.

## 4. Load transfer through bond strength

### *4.1 Discussions on research findings*

In a CFST column, it is expected that both the concrete and the steel tube work in unison to transfer the shear load and thus, a full composite action is achieved. The international standards like the Eurocode 4, AISC 360-16, and CIDECT [31, 43, 60] were developed by considering strain compatibility in both steel and concrete in the CFST columns and confirming full composite action. But as most of the tests on CFST involve axial loading from the top of the column on both steel and concrete, the composite behaviour was easily obtained, but, in a realistic situation, a part of the load is applied from the top, and the other part is applied via the connecting beams using shear connections (as shown in Fig. 2). In such a situation, where loads are communicated via two ways, the beam reaction force might not be able to be fully transferred to the infilled concrete core, and hence the desired composite action may not be achieved [61]. Once the beam shear connection is fabricated and a load path is determined for the introduction of the external forces to the column, the steel-concrete interface should be designed to transmit the longitudinal shear and obtain force equilibrium within the column composite section [43]. To transfer this longitudinal shear force into the CFST column, a load introduction region is specified whose interfacial bond strength is utilised for the load transfer.

#### *4.1.1 Early findings and codal provisions*

The initial works on load introduction can be traced back to 1987 by Dunberry et al. [62], where a total of four test series have been carried out with square concrete-filled columns of varying sections and height and loaded from both tops of the column and via the beam connections. It was observed that the ultimate load of the columns loaded via shear connectors is less than the ultimate load of the stub column with the same sectional properties, with the greatest decrease being 8%, which indicated an inability of the connection to transfer the entire load into the core.

When a load is applied from both top and via the connections, a zone of strain incompatibility is observed within the connection length, which is said to indicate relative slip between the steel and concrete and extends up to a length of 3*b*-3.5*b* below and 1*b*-2*b* above the connection (*b*, being the width of square column). The test observed that having enough length above the connection (approximately 2*b*) or capping the top of the column can result in a load transfer mechanism. The CIDECT guideline [60] on load introduction length is based on the observations of Dunberry et al. [62] and also recommends a reduction in concrete strength for obtaining the composite column cross-section capacity for all types of simple shear connections if full load introduction is not achieved and no shear connector is used.

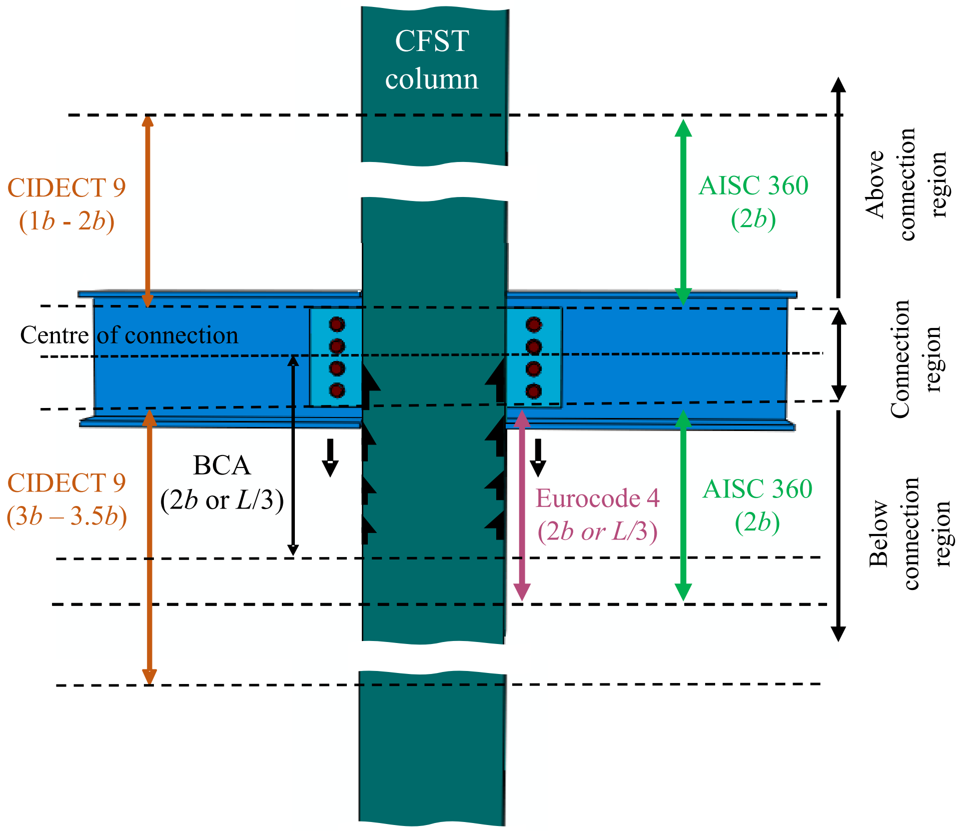



Fig. 12: Load introduction regions as per different international standards.

The current code-based provisions in Eurocode 4 [31], AISC specification ANSI-AISC 360-16 [43], and Buildings and Construction Authority guide BC4:2021 [63] of Singapore, the load transfer is assumed to occur through an *introduction length*. Eurocode 4 states the length to be a minimum of 2*b* or *L*/3, where *b* is the lesser transverse dimension of the column and *L* is the length of column, though it is not explicitly mentioned if the transfer length above or below the connection. Whereas, the BC4:2021 indicates that the load transfer length ranges from centre of the connection region towards the lower direction of the column. The AISC specification states the load introduction length for shear connections as no more than twice the minimum transverse dimension of a rectangular CFST column or twice the diameter of a circular CFST column, both above and below the load transfer region. The regions of load introduction as stated by various international codes are represented as in Fig. 12. Also, as per AISC, the nominal strength in the force transfer mechanism by shear connection is dependent on the load introduction length. Also, for the transfer mechanism by direct bond, interaction is dependent on both load introduction length and nominal bond stress. The Eurocode 4, AISC code and BC4 also suggest using shear connectors or steel anchors within the introduction length if the design shear strength within the length is exceeded, so that load can be easily transferred to the infilled concrete core. Also, the slip between the steel tube and concrete surface may not be large enough to enable the shear connectors to develop their own design strength, as typical shear studs require about 2.5 mm slip before their strength is fully mobilized, whereas the actual slip is generally less than 0.6 mm and thus the effects of studs may be ignored [64].

#### *4.1.2 Recent observations*

The most recent works conducted by Mollazadeh et al. [61, 65, 66] on the load transfer mechanism in CFST columns have attempted to redefine the load introduction length and proposed a new column cross-section resistance formula based on the findings. The test program comprised of five load introduction tests with a varying length above the connection, with and without capping and with and without shear studs below the connections (Fig. 13 and Fig. 14). But unlike Dunberry et al. [62], the loading was through shear connectors only using fin plates, where the connectors were welded to the column steel, as shown in Fig. 14. The study observed that the load was introduced from the beam to the infilled concrete core via the column length above and within the connection and via the cap, provided a cap is welded at the top of the CFST column. The length below the connection has been found to have no effect on load introduction, which is contrary to the recommendations of EC4, ANSI-AISC 360-16, CIDECT and BC4. The results state that specimens with a long length above the shear connection or with a capped column, both concrete and steel, reached their respective peak stress and yield strain just below the connection, indicating complete load introduction (Fig. 15).

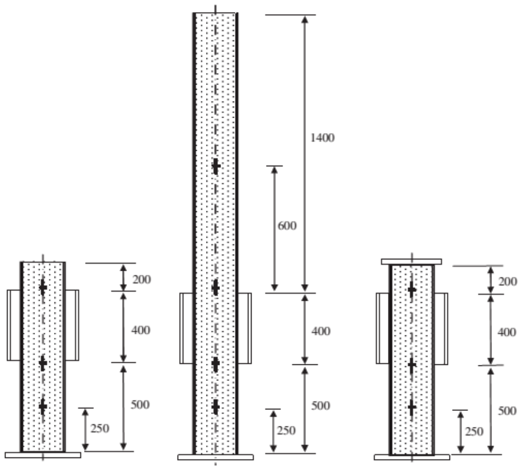



Fig. 13: Specimens with a varying length above the connection region [66].

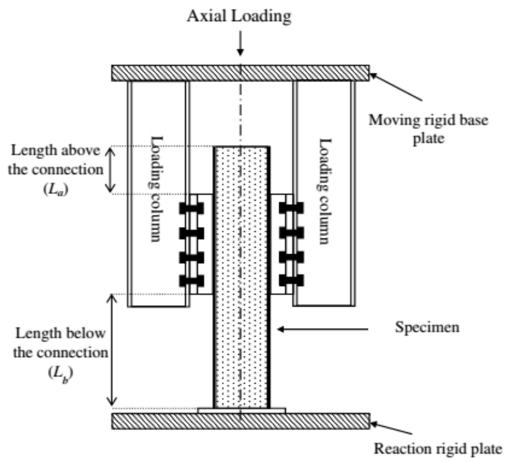



Fig. 14: CFST column test setup for investigating load transfer mechanism [66].

It was observed that, increasing the column length above the connection region increases the force in the concrete core, and can attain full plastic resistance below the connection, but samples without enough length above the connection could not achieve the full strength and load is merely transferred from steel to concrete core (Fig. 16). Based on the experimental investigations, the research group proposed a new column cross-section resistance equation:

$$P_{cal} = P_s + P'_c \tag{5}$$

$$P_s = A_s f_y \tag{6}$$

$$P'_c = Min\{PE \times (L_a + L_c) \times \tau, A_c f''_c \tag{7}$$

where $P_{cal}$ is the maximum load of the CFST column; $P_s$ and $P'_c$ is the load in tube and concrete core, respectively, at the bottom of connection; $A_s$ is the cross-sectional area of tube section; $f_y$ is the yield strength of the tube; $PE$ is the perimeter of the steel-concrete interface; $L_a$ is the length of column above connection; $L_c$ is length of connection; τ is tube-concrete interface bond stress; $f''_c$ is the uniaxial compressive cylinder strength of the concrete.

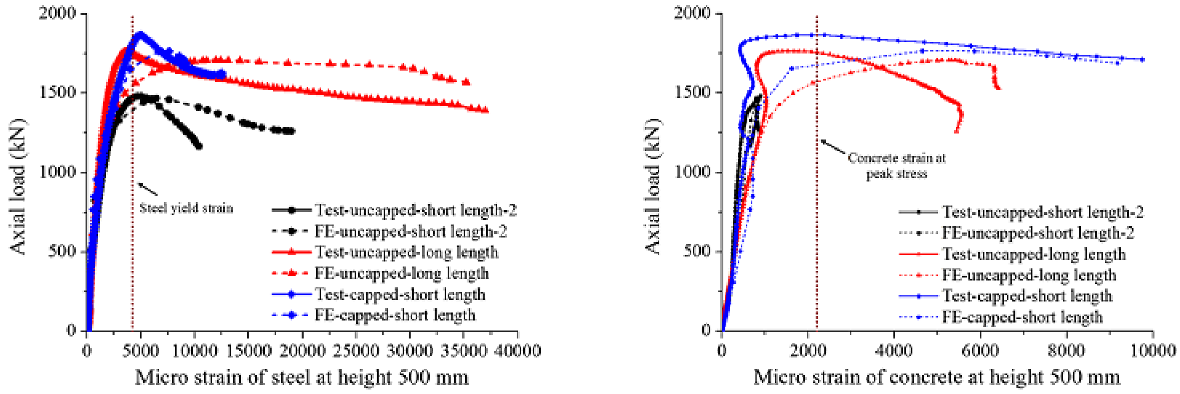



Fig. 15: Micro strain of steel and concrete just below the connection [66].

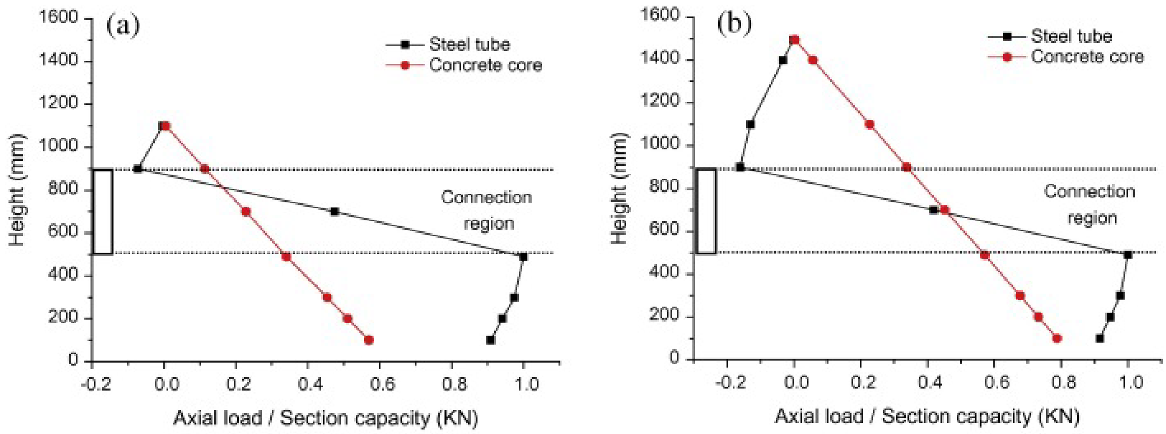



Fig. 16: Distribution of forces in tube and concrete having column length (a) 200 mm and (b) 1400 mm above connection [61].

The EC4, AISC 360 and BC4 recommend providing shear studs (or steel anchors, as per AISC) in the load introduction length if the design shear strength exceeds the interface between the steel and concrete in the CFST column. But in the testing programme of Mollazadeh et al.[66] it was observed that providing welded shear connectors like studs below the connection region merely transmits the load from the tube to the concrete core without any enhancement of the total laod in the column. Another recent study on the load transfer mechanism was conducted by Xu et al. [67], where investigation was conducted with steel-encased CFST column having shear connections. It is significant to note that the load transfer in each component, steel tube, concrete, and encased angle initiated from the region of the column above the connection, as presented in Fig. 17. The findings include that the load introduction length of 2*d* above the column connection region was conservative for compact sections but overestimated for non-compact and slender sections. And below the connection region, the load introduction region was close to the connection, possibly due to geometric discontinuity and pinching effect, and thus the load introduction immediately below the connection region can be neglected.

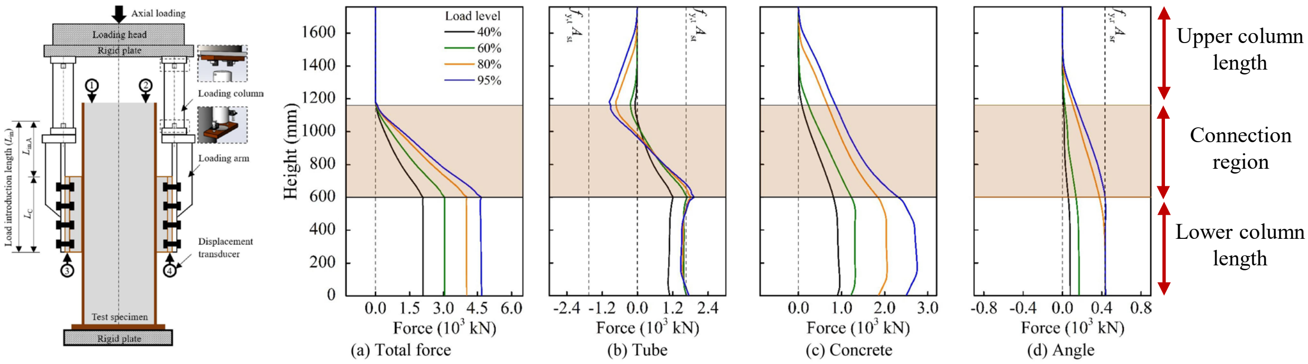



Fig. 17: Test setup and force distribution in each component of the CFST column [67].

### 4.2 Commentary on findings and potential research gaps

As observed, there are significant differences in early observation, codal provisions, and recent findings on load transfer mechanism by bond resistance for CFST columns having shear connection. Further remarks on the existing findings and potential research gaps are presented as follows:

a. In the experimental program of Dunberry et al. [62], most of the specimens had steel capping at the top of the column, but this configuration would only be considered in top-floor columns; secondly, many specimens had an experimental failure. Further investigation with part loading from column top and part loading via shear connection without column capping needs to be conducted to understand the load introduction mechanism with varied column slenderness, connection type (extended end-plate, short end-plate), and varied length above connection region.

b. The research investigations of Mollazadeh et al. and Xu et al. [66, 67] confirms that the load introduction occurs from above and within the column connection. More investigation is required to understand how much the compressive strains generated in the concrete core above the connection are balanced by the tensile strain generated in the steel tube above the connection, and thus also to assess the degree of designing the column above the connection for the additional load transferred from the connection below.

c. As seen in Eq. (7), the load transfer is dependent on the bond strength of steel tube and concrete interface. Therefore, calculating or predicting an unrealistic bond strength based on the empirical equations (as seen in Fig. 11) could possibly influence the estimation of the cross-sectional resistance of the CFST. Therefore, the calculation of a more accurate bond strength is necessary, as discussed in Section 3.2.

d. Though the shear studs merely transfer the load from the tube to the concrete core, the effectiveness of shear studs in reducing the tube stress and local buckling along the load introduction length can be investigated. Tests with shear studs in the connection region, column region above the connection region need to be conducted to check the effectiveness of shear studs in load transfer to the concrete core.
e. All the existing research on load introduction length is based on shear connections of steel beams welded with the column tube, and any observations based on the bolted connection are not found. Investigation with anchored blind-bolted connections could possibly redefine the hypothesis of load introduction length in CFST columns.
f. It also remains to be investigated about the load introduction length for compact, non-compact and slender members with the different internal construction of CFST like, with internal diaphragm plates, external diaphragm plates, beam connections with steel corbel, etc. configurations.

# 5. Load transfer by bearing of blind-bolt

## *5.1 Discussions on research findings*

This section presents the review on load transfer by bearing of blind-bolts in CFST column. An exhaustive compilation of available blind-bolted CFST column connections in the literature is not intended. Rather the authors' perspective on how load transfer and composite behaviour of the CFST column can be enhanced by adopting blind-bolted connection, factors the influence its behaviour, and future directions for overcoming issues of load transfer through bond strength (as discussed in section 3 and 4) are reflected here. Generally, composite construction has three varied connection types, viz: welded connection, bolted connection, and a combination of welded and bolted connection in a single joint. Apart from these, steel beams passing across the CFST composite column, transferring load by full bearing on the concrete core is also an effective load transfer mechanism [68]. But in such cases, though better composite action may be achieved, but steel beams passing through the columns in interior joints and for smaller diameter columns may be cumbersome. Amidst various connection types, direct blind-bolted connections have been explored to a lesser extent, and currently no guidelines are available for such bolted connections [69] and thus, the benefits and functioning of blind-bolted connections from previous works are featured here.

### *5.1.1 Blind-bolts for enhanced composite action*

Though welded connections in steel-beam and CFST columns are more widely used [70], the use of bolted connections has been limited due to the excessive slippage of bolts, column tube wall deformation, and low moment-resisting capacity [71]. Moreover, welded connections have to deal with heat-affected zones and fabrication can be expensive owing to requirement of skilled workers. Whereas, these issues can be overcome by adopting blind-bolted connections that offers faster fabrication, stability and less cost [71]. The Lindapter hollo-bolt, Ajax blind-bolt, Flowdrill bolt, and Molabolt are the most used bolt for connection fabrication, and they have been modified in several arrangements to enhance their performance for moment-resisting frames.

Shakir-Khalil [72] conducted tests with the use of fin-plates welded with the CFST columns, where the steel beam was assembled with the welded fin plate by using bolts. In this testing program, shear connectors were adopted to investigate the effective transfer of beam loads to the infilled concrete of the CFST, and reported failure due to yielding of shear plates and bearing failure around the bolt holes. An analysis of six distinct connection types with circular CFST columns was conducted by Alostaz et al. [73], which involved investigation of simple connections, through steel bars, through plate connection, and headed bolts embedded in the concrete core. The findings revealed that the elastic stiffness and inelastic performance of the connection have significantly increased with the presence of headed stud connection, as shown in Fig. 18. Interestingly, it was also noted that, the studs located away from the beam web did not contribute much for the flange tensile capacity.

Investigation on blind-bolts by Goldsworthy et al. [74] with various forms of welded extensions were conducted, where the blind-bolts were welded with straight bars and cogged bars, that were embedded in the concrete core. The effectiveness of the bolts with cogged bars in improving the strength and stiffness parameters of the T-stub connection in tension was significant, and for the bolts with straight bars, there has been a rise in the secant stiffness.

Yao *et al.* [75] developed a moment-resisting connection for circular CFST columns by modifying the standard Ajax blind bolts (Fig. 19 (a)). The bolts had a cogged extension, as shown in Fig. 19(b), that enables it to enhance the pull-out capacity. The test exhibited improved strength and stiffness that can help transferring moments from the beam to the CFST column. The cogged bar extension enhances the behaviour in tension due to anchorage in the concrete core. The research group [76] carried out the test on connections to rectangular CFST columns using the same type of blind-bolts, and the connection was classified as rigid connection for braced frames and stiff semi-rigid for unbraced frames. But fabrication of such cogged connections can not only be difficult, but also might have brittle fracture of the extended cogged part which is welded, and therefore, the headed anchored blind-bolt was proposed (Fig. 19(c)) by Agheshlui et al. [77]. The experimental program involved direct pull-out test of the blind-bolted connections, where the specimens having anchored blind-bolts failure commenced with concrete cone formation, followed by tube wall yielding or necking of the bolt shank. Also, it was observed that blind-bolts closer to the corners of the column faces contributed for higher strength and stiffness relative to blind-bolts in the middle of column face due to the transfer of load to the tube side face. Moreover, the bolts in the middle face of the column section were not effective in group behaviour, for both strength and stiffness. A further modification of the headed Ajax blind-bolt by Oktavianus et al. [78] called the Double Headed Anchored Blind Bolt (DHABB) (Fig. 19(d)) is also found to have a higher secant stiffness under cyclic loading.

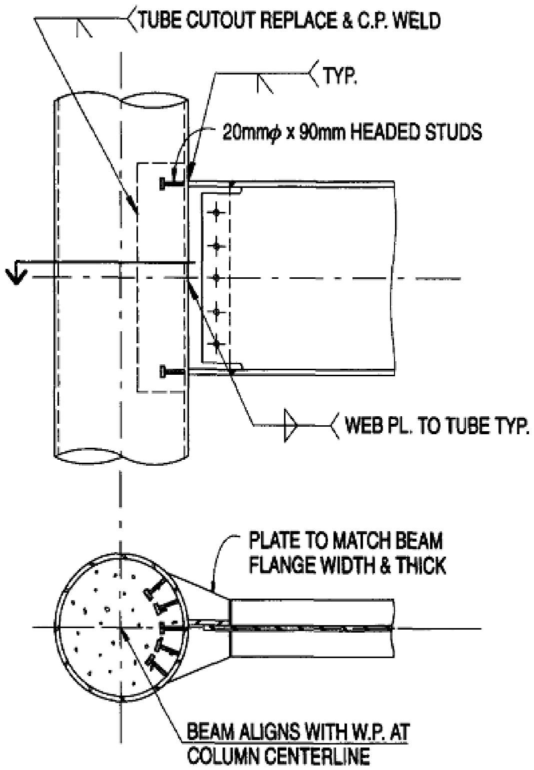



Fig. 18: Connection with interior-headed studs [73].

The Linadpter hollo-bolt (Fig. 19(e)) has also been modified to enhance the global connection performance. Proposed and investigation by Pitrakkos et al. [79] on the modified hollo-bolt, called the Extended Hollo-bolt (EHB) (Fig. 19(f)) that comprises the expandable sleeve, shank, nut, and washer, and includes a pre-load which is applied at the tightening stage, has been explored to evaluate its structural performance. The EHB is modified by extending the existing shank length and providing a nut, and the primary purpose is to have an anchorage into the concrete core of the CFST column, as presented in Fig. 20. Due to the mechanical anchorage provided by the headed nut of the blind-bolt, the connection displayed significant enhancement in strength and stiffness, and thereby considerably reducing the deformation and relative slip. Other performance parameters like rotation capacity, ductility and cyclic energy dissipation were also assessed for the extended hollo-bolted CFST connections [80]. The failure modes of the CFST column connection with EHB under tensile loading have been extensively studied by Tizani et al. [80-82], and prominent failure modes include bolt shank fracture and column tube wall deformation. Experimental, numerical, and analytical investigation of EHB connections with CFST column having combined failure mode of concrete crushing and tube wall bending was studied by Debnath et al. and Cabrera et al. [83-86], and observed effectiveness of EHB in enhancing the concrete contribution in resisting the applied forces. The major failure patterns identified were, a weak blind bolt – strong CFST, having bolt shank necking and limited tube wall yielding; and strong blind bolt – weak CFST, having concrete cone failure, column surface deformation and failure of bolt expandable sleeve. As observed by Oktavianus et al. [69], use of higher bolt diameter does not have a significant increase in stiffness if not accompanied by an increase in embedment depth or reduction in column diameter to tube thickness ratio.

A further modified version of the hollo-bolt was proposed by Jeddi et al. [71] called the Tube Bolt (Fig. 19(g)), that have two expendables sleeves with an headed anchor nut. The first sleeve can clamp to the column-face, and the second sleeve and the end anchor member can prevent the bolt slippage in the concrete core. As a result, load bearing capacity of the tube bolt was enhanced by 2.25 times as compared to the EHB as proposed in [79]. Investigation was also conducted by Tao et al. [3] by adopting standard hollo-bolts, but also had reinforcing bars at the connection region, and observed an increase in initial stiffness by 62% and reduction in separation between column tube wall and concrete. The T-head square-neck one-side bolt (Fig. 19 (h)) proposed by [87] though tested with hollow steel column (without infill concrete), provides another alternative to fabricating steel beams to closed-section columns like CFST.

(a) Standard blind bolt (Ajax Fasteners)

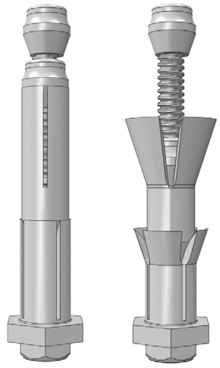
(b) Cogged bolt [75]

(c) Headed anchored blind bolt

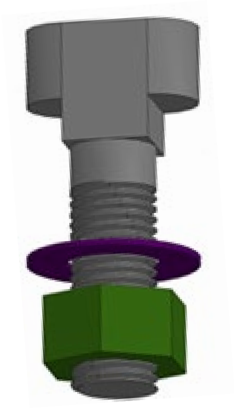
(d) Double-headed anchored blind bolt [78]

(e) Hollo-bolt (Lindapter)

(f) Extended hollo-bolt



(g) Tube bolt [71]



(h) T-head square-neck one-side bolt [87]

(i) Slip-critical blind bolt

(j) Hi-shear bolt (Ajax Fasteners)

(k) Toggle type blind bolt (Midfix)

Fig. 19: Blind-bolts and their various modified forms.

Another recent novel bolt proposed by Wang et al. [88, 89] that introduces a slip-critical blind bolt (Fig. 19(i)) was investigated and performed well in displaying strength, stiffness, and ductility under cyclic loading conditions. Thus, the blind bolts that have been modified either by a cogged bar or extended shank or (and) an additional expandable sleeve with a headed nut for anchorage into the concrete core of the CFST column have considerably improved the concrete contribution in resisting the applied external load, thereby enhancing the composite behaviour of the column. In addition to these modified blind-bolts, the conventional bolts that are used and currently available in the market and mostly manufactured in Australia and UK are shown in Figs. 19 (j)-(k). Further studies on numerical modelling and performance on blind-bolted CFST connections under direct tensile loading can be sourced from the works of Debnath et al. [90].

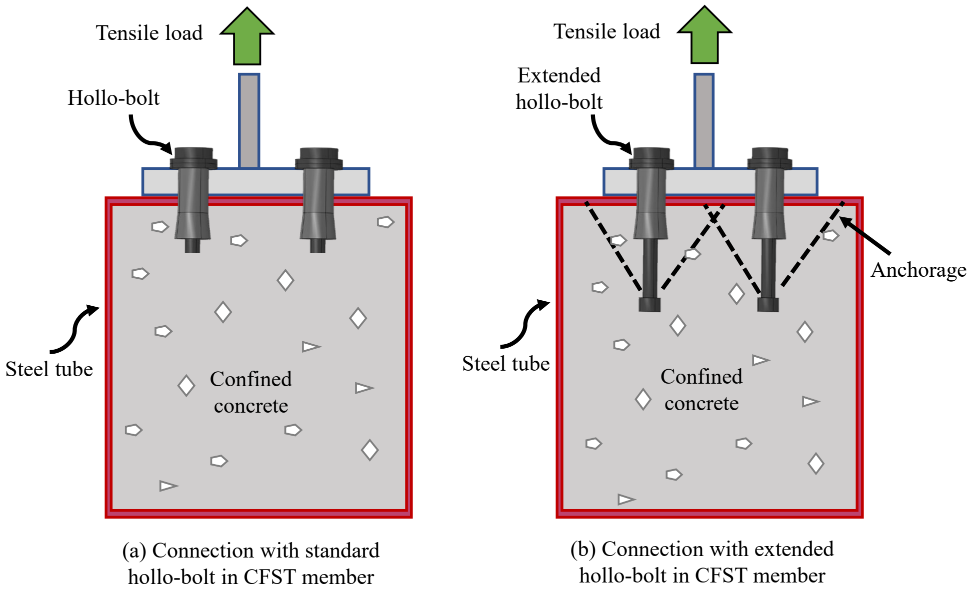
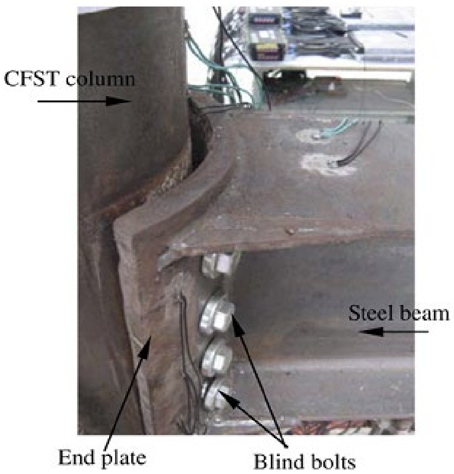



Fig. 20: Connection with standard and modified hollo-bolt showing anchorage/bearing in concrete.

### *5.1.2 End-plates in connection performance*

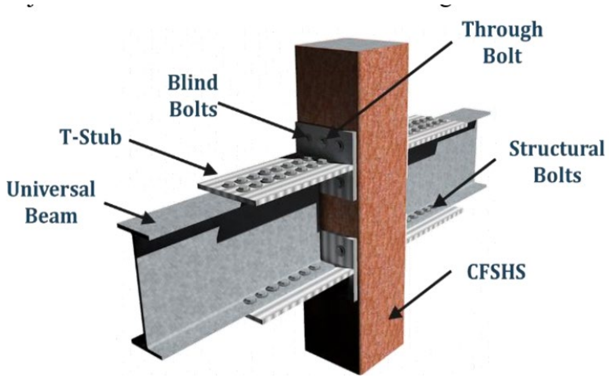
The type of endplate has always been an influencing factor in determining the performances of the open steel beam to CFST column blind-bolted connections. In the literature search for steel beam blind-bolted connection to CFST column, primarily four types of end-plates are adopted. They are, flush end-plate (Fig. 21 (a)), extended end-plate (Fig. 21 (b)), T-stub end-plates (Fig. 21 (c)) and L-stub end-plate (Fig. 21 (d)) connections. The test by [91] on the behaviour of flush end-plate showed that the performance of connection is affected by the thickness of the end-plate, and column cross-section. Another investigation of the structural performance of blind bolted endplate composite connections to CFST columns by Thai et al. [92] observed the effects of column shapes and endplate types in composite joints under static loads. The findings include that, the circular CFST column shaving extended end-plates can effectively reduce the transverse deformation as compared to the square CFST columns having flush end-plates. The beneficial effect of extended end-plate can also be gauged from the fact that it exhibited higher moment resistance and initial stiffness as compared to the connections having flush end-plate. Though not many works on a thorough-bolt connection are found, an investigation by Tao et al. [93] on cyclic behaviour of composite joints using through-bolt connection with flat and curved extended end-plates having 10 mm thickness was adopted for joint fabrication for square and circular CFST columns, respectively, and found to have high energy absorption capacity. The thickness of the endplate also enhanced the rotation stiffness and moment capacity of a connection and thus supports the claim by Wang et al. [91]. Though another parametric study carried out by Wang et al. [94] observed that the moment carrying capacity and initial connection stiffness increases with an increase in thickness of the flush endplate till the thickness is 16 mm, but it seems to have no effect beyond this thickness. The investigation also reports the influence of steel tube strength, diameter of the blind-bolt, applied bolt pretension, and beam-column yield strength ratio on ultimate moment-resisting capacity of the blind-bolted end-plate CFST column connection. The effect of higher diameter blind-bolt is also reflected in the investigation of Jeddi et al. [71].

Contrary to the observations made by [94], the investigation by [95, 96] demonstrated that using moderately thick end-plates can lead to enhanced strength both for extended and flush type end-plate connections. Apart from exhibiting high strength and stiffness, the blind-bolted extended end-plate connection also displayed excellent rotation capacity thereby meeting the requirements for higher seismic performance. Also, in the study by [97], comparing the flush and extended end-plate, the extended end-plate displayed higher bearing capacity and initial stiffness of the frame. Apart from flat T-stubs as used by Pokharel et al. [98], curved T-stubs have also been adopted for CFST blind-bolted connection assemblies as shown by Oktavianus et al. [78]. From the experimental investigation involving curved T-stub connections, the flange thickness of the stub has been regarded as the most influential parameter as it can enhance the secant stiffness of the connection, and thereby, recommended a minimum value of flange thickness as 1.25 times the blind-bolt diameter, as suggested in the studies of Oktavianus et al. [78]. It is also reported that the use of blind-bolts with stub plates decreases the effect of increasing thickness of the T-stub flange, and thus investigation for defining the higher limit of flange thickness is required. For the investigation with anchored blind bolts using L-stub plates by Agheshlui et al. [77], failure modes of concrete cone failure, L-stub yielding, and bolt failure were observed, but any parametric study on L-stub thickness and its other configuration was not included in the investigation.

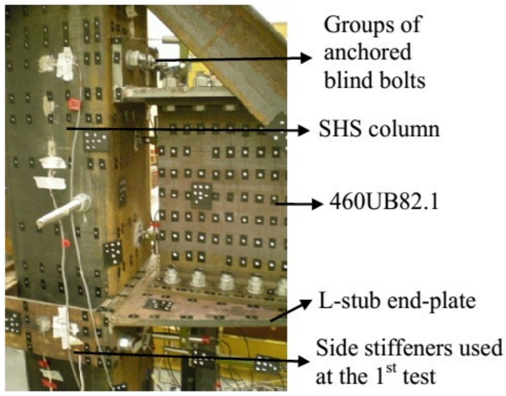



(a) Flush end-plate [96]


(b) Extended end-plate[95]




(c) T-stub end-plate connection [99]




(d) L-stub end-plate connection [77]

Fig. 21: Forms of different end-plates, having blind-bolted beam-to-CFST column connection.

### *5.1.3 Blind-bolted connection assemblies*

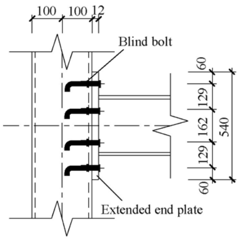
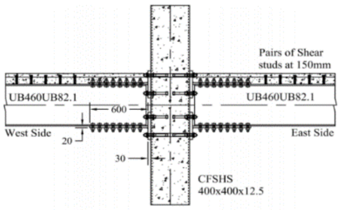
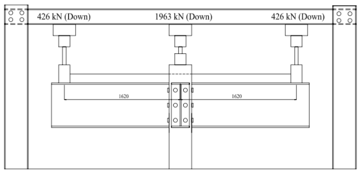
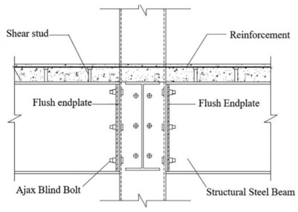
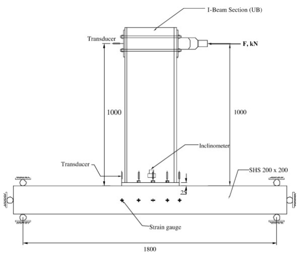
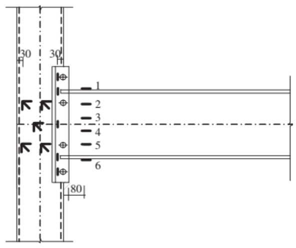
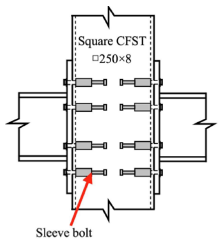
In the literature, several studies on directly blind-bolted CFST column connections are available, where the primary focus was on developing semi-rigid or rigid connections as it is generally assumed that the bolted connections though having sufficient tying and shearing resistances but are considered pinned connections. In this sub-section, the different types of blind-bolted CFST column connection assemblies that displayed rigid or semi-rigid behaviour are reviewed. The experimental investigation with cogged blind bolts having CFST column connection by Wang et al. [97] under cyclic loading exhibited semi-rigid behaviour having large hysteretic loops, high ductility, and energy dissipation capacity. The large-scale testing by Agheshlui et al. [100], which involved the use of Ajax anchored blind-bolt and through bolts to fabricate the CFST column connection designed along with composite slab, investigated the joint under gravity and cyclic loadings. The investigation showed that the proposed connection has high stiffness and capacity, and it can be categorized as a semi-rigid connection as per Eurocode 3. Composite beam to CFST column flush end-plate connections using Ajax blind-bolts were tested by Mirza et al. [101] that compares connection setup under static and quasi-cyclic loading. It was observed that composite beams with semi-rigid connections performed well under severe earthquakes.

The standard Ajax blind-bolts were also used by Waqas et al. [102] to test full-scale composite beam-to-CFST column connection under static and cyclic loading. The authors observed that the semi-rigid composite connection displayed satisfactory yielding, strength, and ductility. The rotational stiffness of the steel beam-to-CFST column connections using the extended hollo-bolt was investigated by Tizani et al. [103], where the connection was assessed for its moment performance.

It was observed that all the connections failed by bolt fracture in tension and thus achieved full capacity of the connectors, and though the tube thickness and concrete infill influenced the behaviour but did not dominate. The connections mostly exhibited semi-rigid behaviour, and none of the specimens behaved as pinned joints. The hysteretic performance of steel beam-to-CFST column using extended hollo-bolt was also evaluated by the same research group [80] and observed no damage in the extended part of the blind-bolt, signifying its performance for improvement of strength and stiffness of the connection. The failure mode of strong bolt – weak CFST column demonstrated higher energy dissipation and ductility ratio and, therefore, can be adapted for moment-resisting frames.

Apart from the above tests, circular and square CFST column connections with blind-bolt having welded rebar extension were also explored by Wang et al. [104] subject to cyclic loading and assessed the connection performance, including the anchorage action of the modified bolt. All the joints displayed semi-rigid behaviour, with most of them having full strength as per the classification of Eurocode 3. A similar type of anchored blind bolt called the threaded sleeve bolt was used by Liu et al. [105] to fabricate steel beam to CFST column connection and subjected to cyclic loading. The investigation observed that the proposed connection behaved as a rigid and full-strength non-sway frame. A layout of different blind-bolted CFST column connection assemblies that behave as semi-rigid or rigid moment connections is presented in Table 2.

Table 2: Layout of different connection assemblies of blind-bolted CFST moment connections.

| Sl. No. | Author | CFST section | Type of blind bolt used | Applied loading | Connection configuration |
|---|---|---|---|---|---|
| 1 | Wang et al. [97] | Square | Cogged or hooked-type blind-bolt | Cyclic |  |
| 2 | Agheshlui et al. [100] | Square | Ajax anchored blind-bolt and through bolt | Monotonic and cyclic |  |
| 3 | Mirza et al. [101] | Square | Ajax blind-bolts | Monotonic and cyclic |  |
| 4 | Waqas et al. [102] | Square | Ajax blind-bolts | Static and cyclic |  |
| 5 | Tizani et al. [103] | Square | Lindapter anchored hollo-bolt | Monotonic |  |
| 6 | Wang et al. [104] [95] | Circular and square | Rebar welded to blind-bolt | Monotonic and cyclic |  |
| 7 | Liu et al. [105] | Square | Sleeve bolt having anchorage | Cyclic |  |

### *5.2 Commentary on findings and potential research gaps*

The purpose of reviewing standard CFST blind-bolted connection is to provide with the understanding of effective load transfer to encased concrete by bearing, factors influencing connection behaviour and highlighting its moment-resisting capacity. Secondly, the review presents a direction towards enhanced composite action of CFST. The prominent findings and potential research gaps are stated as follows:

a. In group performance of bolts, the positioning of blind-bolts is important, as gauged from literature. Research investigation on anchored blind-bolt positions in group performance is required.

b. The use of cogged bars and cogged extensions by welding in blind bolts have found to achieve higher stability, but using them may not be feasible. Therefore, the headed anchored ajax blind-bolts or hollo-bolts may be used preferred as there is no welding of the extended shank with the original shank length.

c. Double-headed anchored blind-bolt and tube bolt have been observed as the most effective blind-bolt based on tensile pull-out tests. To further evaluate their effectiveness, the group performance with different gauge and pitch distances of these blind bolts remains to be investigated as there might be overlapping concrete cone stresses.

d. Experimental investigation on steel beam to CFST column connections with a group of anchored hollo-bolts are found to behave as semi-rigid moment-resisting frames, but it was observed that most of the testing programmes had rigid end-plate, thus ignoring its influence. To evaluate the connection performance from a practical perspective, investigating of steel beam-to-CFST column connections having thin end-plates needs to be conducted which will consider the prying action.

e. As most of the test results on beam-to-CFST column connections with anchored blind-bolts exhibited excellent performance when subjected to both monotonic and cyclic loading, this could possibly be due to effective load transfer to the concrete core with the help of anchored blind-bolt shank. This indicates that the blind-bolt protrusion to the concrete core can also overcome the issues of bond strength.

f. Investigation into the load introduction mechanism of the beam-to-CFST connections using anchored blind-bolts needs to be carried out as the region of load introduction length for such connections might be significantly distinctive from the existing knowledge, which is based on welded shear connections.

g. The design guidance as provided in Eurocode 3 Part 1-8 [106] for steel joints is limited to structural joints between open and built-up welded profiles, and no component-based design guidelines are available, and thus more investigation to fully exploit the individual elements that contribute to the overall deformability of the connection is needed.

## 6. Conclusions

The article presents an up-to-date review and analysis of the existing research and findings on understanding load transfer in CFST columns, which includes interfacial bond behaviour, transfer of load though bond strength and blind-bolted bearing mechanism. The idea of presenting the above-mentioned specific areas are not only because there are rare state-of-the-art reviews on these topics, but also to connect them to each other that can effectively address the issues that are inter-related. For the bond strength in CFST columns, the previous research on factors influencing bond stress, empirical equations for prediction of bond strength, and data from a wide range of sources were collected and analysed. For the load transfer though bond strength mechanism, the existing and current knowledge along with the code provisions have been reviewed, and the scope for further investigation is laid out. For understanding load transfer though bearing mechanism of blind-bolts in CFST columns, the development of various types of high-strength blind-bolts, factors influencing the composite action of CFST, and the performance of these connections as semi-rigid or rigid connections have been discussed elaborately. Research in the discussed sectors with innovative use of expansive concrete, high-strength steel, high strength concrete can also open new trends and supplement the existing works. It is expected that the article will provide encapsulation for the three broad topics on CFST, and for in-depth details of the investigations the reader may easily access the references listed at the end of this article.

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