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PERFORMANCE EVALUATION OF BOND STRESS IN THIN-WALLED CONCRETE-FILLED STEEL TUBE COLUMNS WITH SHEAR CONNECTORS

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Abstract. *The performance of interfacial bond stress in thin-walled concrete-filled steel tubular columns (CFST) with headed shear connectors has been investigated. Square CFST columns were considered and were designed as both compact and slender cross-sectional columns. Investigations have been carried out with different interfacial conditions including with and without headed shear connectors. For the specimens with headed shear connectors, they were arranged in different layout of shear connectors having variations in connector horizontal and vertical spacing. Push-out tests were conducted, where the load was applied only to the concrete surface. It was observed that, compact composite cross-sections had higher bond stress as compared to slender cross-section columns. Use of shear connectors have led to significantly enhance the bond stress by 4 times for composite sections. For the slender sections, with minimum use of shear connectors having vertical spacing of width/2 can achieve a bond strength of 1.0 N/mm^2. The enhancement of bond stress is not always proportional to the reduced spacing of shear connectors, as it can lead to early local buckling of steel tube.*

1 INTRODUCTION

With rapid adoption of high-strength steel in construction industry it has also become an attractive option to use thin-walled steel components, provided they are designed to eliminate local buckling. Apart from the traditional benefits of concrete-filled steel tubular (CFST) column like resisting higher loads in post-yield capacity, better fire resistance, lesser vibration sensibility and higher ductility [1-4], the use of thin-walled tubes for CFST columns can also be beneficial for higher performance and optimised sections with reduced self-weight [5, 6]. In actual structures, the load is transferred from the beam connection to the CFST column by interfacial bond stress between the inner surface of the steel tube and the infill concrete core. Therefore, to enhance the load transfer and thereby to improve the performance of thin-walled tubes for CFST columns various stiffening methods were previously investigated. Performance for bond stress with use of internal steel rings were conducted by Tao et al. [7] involving various types of concrete, including recycled aggregate and expansive concrete. Qu and Liu [8] investigated bond between steel and self-compacting lower expansion concrete in composite column with different interfaces including few specimens with shear studs. Stiffeners like, tab stiffeners, horizontal angles, headed stud stiffeners have also been part of the investigation by various researchers including Petrus et al. and

Nardin et al. [9, 10]. To enhance the bond, specimens with single and double steel plate layer, steel bar, including crown-shaped protrusion on steel surface have also been investigated [11, 12]. But there are limited systematic investigation solely on influence of headed shear connectors in improving the interfacial bond stress, in particular for thin-walled CFST columns. As the composite behaviour in CFST columns is dependent on the interfacial bond stress between steel tube and concrete, and at the same time arresting local-buckling in thin-walled CFST tubes is important, the use of headed shear connectors can serve both the purposes and therefore necessitates this research.

In this study, an experimental investigation has been carried out to evaluate the interfacial bond stress between steel tube and concrete in CFST columns having shear connectors. A series of laboratory push-out tests have been conducted to determine the bond stress. The parameters studied include, composite cross-section type (compact and slender sections), interface without shear connectors, vertical spacing of shear connectors and horizontal spacing of the shear connectors. The findings of this work will be useful in not only enhancing the bond stress of the thin-walled CFST column which is crucial for load transfer, but also in utilising the steel tube strength to its potential by arresting its premature local buckling.

2 EXPERIMENTAL PROGRAMME

2.1 Specimen design and fabrication

A laboratory testing programme was conducted to determine the interfacial bond stress between steel tube and concrete core of the CFST columns. The steel tubes for the CFST columns were fabricated from mild steel plates of 4 mm thickness with nominal yield strength of 525 N/mm^2, which were first made in to angle- or L-sections by press-breaking, and later welding two angle sections to form the steel tube. Thus, the tube formed from the steel angle sections were welded in two diagonal corners. Before the two angle-sections are welded to form the hollow tube, the shear connectors were welded to both the faces of the angle-section (base material) by duty stud guns with use of ceramic ferrule. In this test series, two composite cross-sections were considered, compact and slender cross-sections, as defined by the AISC 360-22 [13] where the limiting width-to-thickness (*b/t*) ratios for box sections are expressed as in Table 1. Thus, the compact cross-section of 150×150×4, and slender cross-section of 300×300×4 were considered in the tests. The height (L) of the specimens were considered to be 3.5 times the width of the outer dimension of the cross-section as per the recommendations by Tao et al. and Ziemian [7, 14].

Table 1: Limiting width-to-thickness ratios for composite members in compression.

Description of element	Width-to-thickness ratio	Compact composite/ noncompact composite	Noncompact composite/ slender-element composite	Maximum permitted
Walls of rectangular HSS and box sections of uniform thickness	*b/t*	$2.26\sqrt{\frac{E}{f_{y,s}}}$	$3.00\sqrt{\frac{E}{f_{y,s}}}$	$5.00\sqrt{\frac{E}{f_{y,s}}}$

The details of the specimen dimensions and arrangement of shear connectors are presented in Table 2, where *B* and *D* are outer width and depth of the columns, *b* is the internal width, and *t* is the thickness of the steel tube. The shear connectors adopted in the study were of 10 mm shank diameter, and the overall length was 55 mm. The diameter and height of the shear connector head was 19 mm and 7 mm, respectively. As the study primarily focuses on the influence of shear connectors in interfacial bond stress of the tube wall and concrete, therefore the connectors were arranged in various configuration to develop the understanding of the bond behaviour. Specimens were prepared without and with shear connectors, where the specimen without shear connectors can be considered as control specimen. The connectors were arranged with varied horizontal (s_h) and vertical (s_v) distances along the width and height of the specimen. For the compact sections, only one column of shear connectors was adopted, that is, horizontal spacing (s_h) is 0, and the vertical spacing of B and $B/2$. For the slender sections, the shear connectors were placed in one column and two columns, thus horizontal spacings (s_h) considered were $B/2$ and $B/3$, and the vertical spacings considered were of $B/2$ and $B/4$.

Table 2: Details of specimens in the testing programme.

Specimen	$B×D×t$ (mm)	b/t	L (mm)	L_i (mm)	No. of column(s) of connectors	s_h (mm)	s_v (mm)
POT-B150-T4-C0-H0-V0	150×150×4	34.5	525	475	0	0	0
POT-B150-T4-C1-H0-V150	150×150×4	34.5	525	475	1	0	150
POT-B150-T4-C1-H0-V75	150×150×4	34.5	525	475	1	0	75
POT-B300-T4-C0-H0-V0	300×300×4	72	1050	1000	0	0	0
POT-B300-T4-C1-H0-V150	300×300×4	72	1050	1000	1	0	150
POT-B300-T4-C1-H0-V75	300×300×4	72	1050	1000	1	0	75
POT-B300-T4-C2-H100-V150	300×300×4	72	1050	1000	2	100	150
POT-B300-T4-C2-H100-V75	300×300×4	72	1050	1000	2	100	75

(a) Fabricated specimens (b) Concrete filling

Figure 1: (a) Steel tubes fabricated with welded shear connectors; (b) filling of concrete.

Commercial concrete with concrete grade of C40 was used to fill the hollow tubes, where proper compaction was carried out during the placing of fresh concrete. Concrete offset of 40 mm was provisioned at the base of the columns for the concrete core to travel under applied forces, and an offset of 10 mm was kept at the top to place the loading plate with ease. Thus, the interface length (L_i) between the concrete and the steel tube van be obtained after deducting the offsets from the total length (L). Figure 1 shows a section of steel tubes after fabrication with shear connectors, and placing of concrete in tubes. The specimens are named as following: POT refers to push-out test, B refers to specimen width, T represents tube thickness, C refers to number of columns of shear connectors in each face of tube, H refers to horizontal distance between connectors and V refers to the vertical distance between the connectors. For example, POT-B300-T4-C2-H100-V150 refers to push-out test of specimen width 300 mm, having tube thickness 4 mm, with two columns of shear connectors in each face of the tube, the horizontal spacing between connectors is 100 mm and the vertical spacing between the connectors is 150 mm.

2.2 Test setup

An experimental test setup was developed where a 1000 tons capacity servo-control loading system loading capacity was used at The Hong Kong Polytechnic University. The CFST specimens were mounted on a rigid steel platform, which is fixed with the laboratory strong floor. As the experimental programme investigates the interfacial bond stress between the steel tube inner surface and the infill concrete, therefore, the loading plate was placed only on the concrete surface at the top of the specimen. The loading plate was fabricated with slightly reduced dimension as compared to that of the infilled concrete width so as to avoid any possible overlapping of the loading plate and steel tube surface. Before the loading plate is placed on the specimens, the concrete surface was coated with gypsum and checked with spirit level to ensure flat surface and avoid any undulations, so that uniform loading is applied to the specimens. Figure 2 presents the experimental test set up.

2.3 Instrumentation

To measure the slip (displacement) of the concrete relative to the steel tube the linear variable differential transducers (LDVTs) were used. A total of 3 LVDTs were used, where two of them were placed at the left and right sides of the specimen, and the other was placed at the rear side of the specimen. The average of all the three LVDTs were taken as the slip/displacement measurement. To measure the strain developed in the specimens and monitor the change in bond stress along the length of the CFST column, several strain gauges were fixed on the steel tube surface. Out of four faces of the square CFST specimens, the strain gauges were fixed on two faces, in two alternate directions. The first strain gauge was placed at 75 mm distance from the column top surface, and the remaining strain gauges were placed at an interval of 100 mm along the tube length. Strain gauges were also placed at larger intervals along with width of the specimen to further observe the bond stress distribution along the width of the CFST column specimen. During the testing, a preload of 20 kN was applied to check the instrumentation. During the actual tests, loading was applied with a preload of 10 kN, and the loading rate of 0.3 mm/min was adopted for all the tested specimens. The loadings were terminated either after initiation of local steel tube buckling or at load drop of 40% of ultimate load, whichever was earlier.

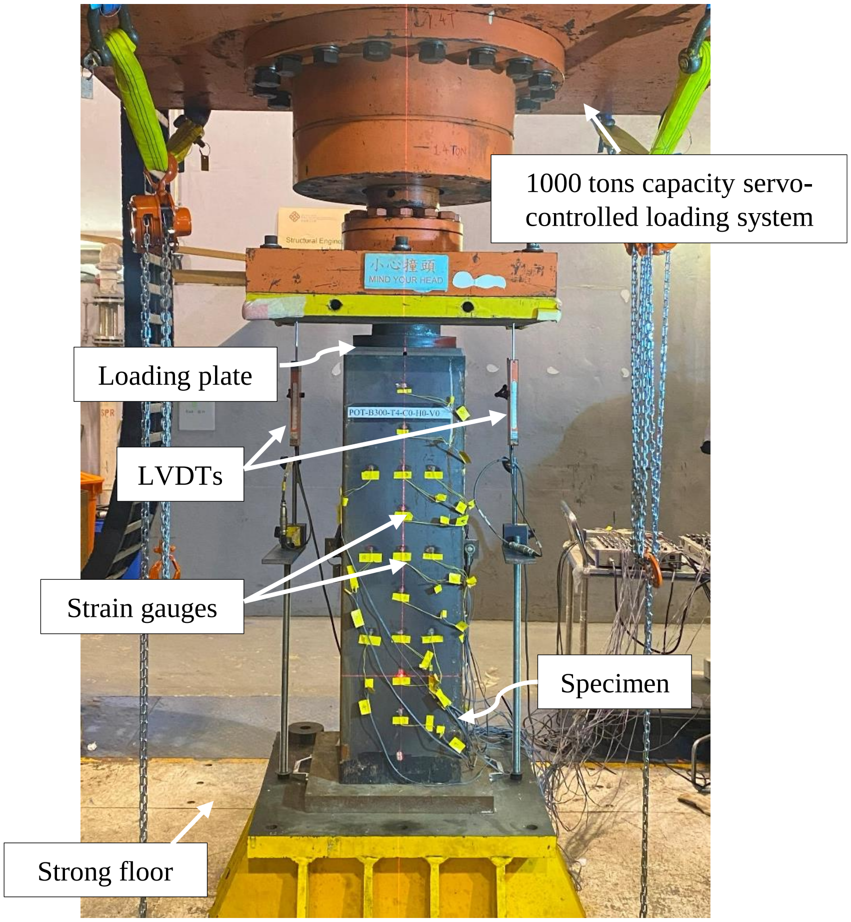


Figure 2: Experimental test setup.

2.4 Material tests

Material tests were conducted to determine the mechanical properties of steel tube and concrete. Steel coupons were extracted from the steel plates of the same batch as used for fabrication of the hollow steel tube specimens. The steel flat dog-boned shaped coupons were designed as per EN ISO 6892-1: 2019 [15] and tested using Instron 8803 servo-hydraulic testing system. For steel material tests, the average of three coupon tests were used to determine the yield strength ($f_{y,s}$) at 0.2% strain, the ultimate tensile strength ($f_{u,s}$) and the elastic modulus (E_s).

To obtain the mechanical properties of the concrete material, standard cylinders of size 100 × 200 mm were casted, and cured by wrapping the cylinders with cling film to simulate similar conditions as infilled concrete in CFST column specimens. Before the concrete cylinder compressive tests, two strain gauges of 120 mm gauge length were attached to the concrete cylinder surface, the data of which was used to determine the concrete elastic modulus. The three concrete cylinders were used to obtain the compressive strength and elastic modulus, and the average values of 44.5 N/mm^2 and 29,400 N/mm^2 was obtained, respectively. Similarly, concrete split tests were also considered to determine the concrete tensile strength, which was obtained as 3.25 N/mm^2. The mechanical properties of the shear connectors were referred from the mill certificates. The summary of the material properties of steel tube, and shear connectors are presented in Table 3.

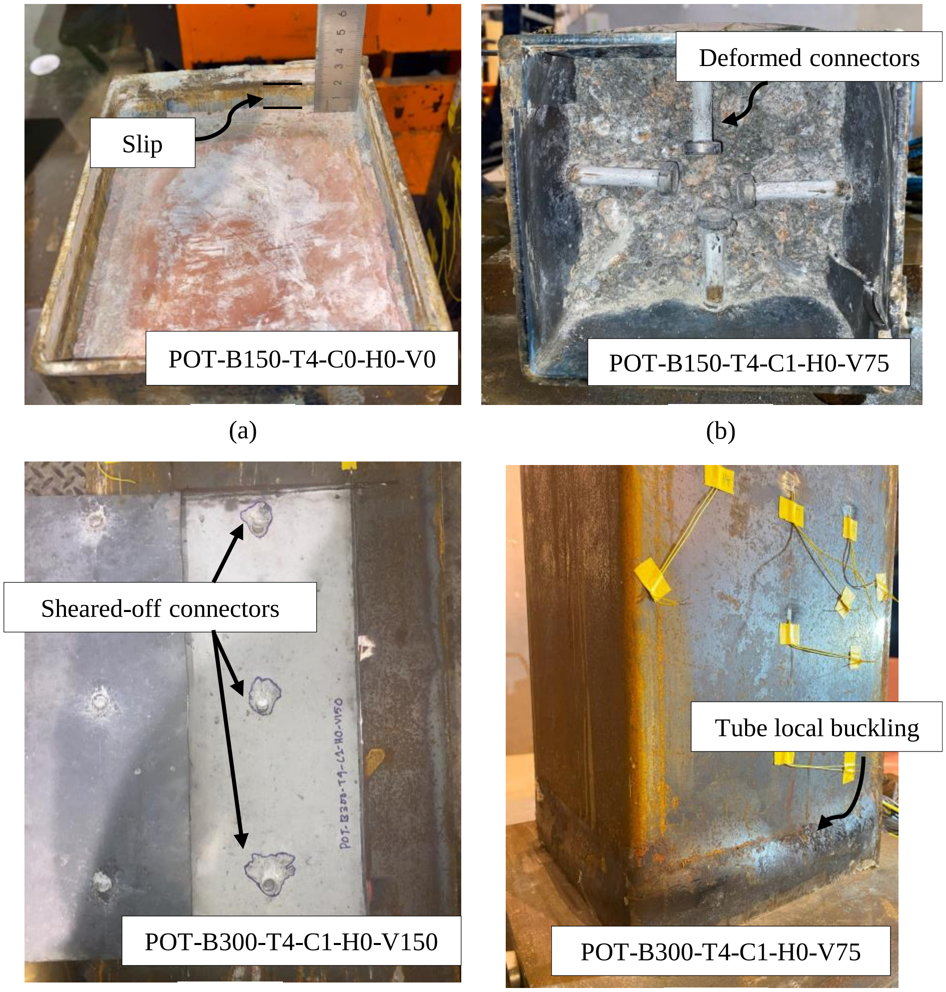
Table 3: Material properties of steel tube, shear connector and infill concrete.

Component	$f_{y,s}$ (N/mm^2)	$f_{u,s}$ (N/mm^2)	E_s (N/mm^2)
Steel tube	525	675	201,200
Shear connector (as per mill certificate)	385	495	200,000

3 TEST RESULTS

3.1 Failure modes

In the testing programme the parameters studied included the compact and slender composite cross-section, steel-concrete interface without shear connectors, interface with shear connectors with varied horizontal and vertical spacings. Based on these studied parameters, four prominent failure modes were observed and reported here. The failure modes were governed by the interface type and also the type of cross-section. For the specimens, both compact and slender cross-sections



(a) (b)

(c) (d)

Figure 3: Failure modes of the tested specimens, (a) concrete slip-out, (b) deformed connectors, (c) sheared-off connectors, (d) local buckling of tube wall.

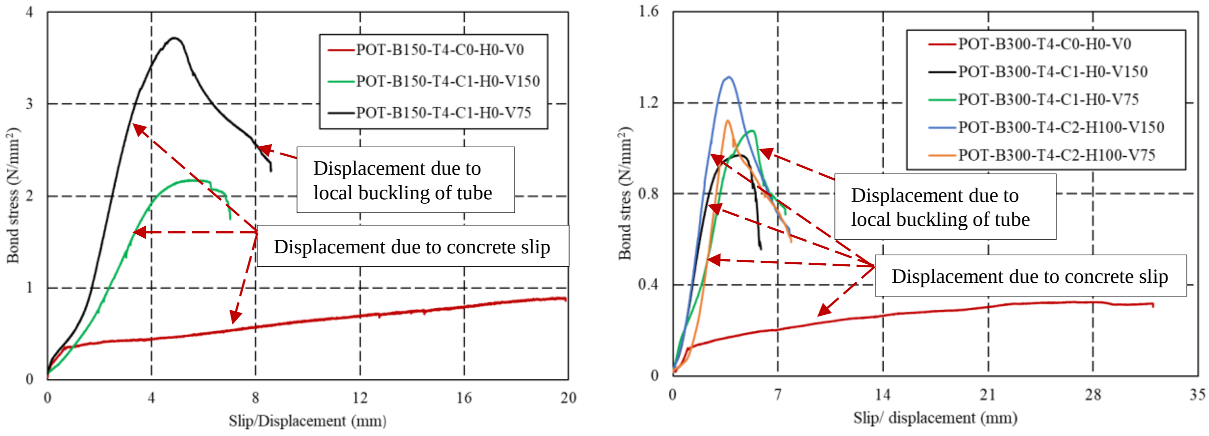
without any shear connector, the failure mode was by significant amount of slip of the concrete core, and no visible deformation of the steel tube. Whereas, for specimen with shear single column (referred as C1 in specimen nomenclature) shear connector with vertical spacing of *B* for compact specimen POT-B150-T4-C1-H0-V150, and *B*/2 for specimen POT-B300-T4-C1-H0-V150 had failed by deformation and shearing-off of the connectors, respectively. On the other hand, the specimens having closely spaced connectors of *B*/2 and *B*/4 for compact and slender cross-section specimens failed by local buckling of the steel tube at the base of the specimens. Similarly, specimens having 2 columns of shear connectors (referred as C2 in specimen nomenclature) irrespective of the vertical spacing distance, have also failed by local buckling of the steel tube at the base. The failure modes observed in the tests are shown in Figure 3.

3.2 Bond stress behaviour

The LVDTs were used to measure the slip of the concrete core relative to the steel tube of the CFST column specimens. The slip versus bond stress behaviour for all the tested specimens are provided in Figure 4. The interfacial bond stress was obtained from the test results by dividing the peak force by the steel-concrete interface area, and can be calculated as follows:

$$\boldsymbol{\tau_u = \frac{N_u}{4bl_i}} \tag{1}$$

where, τ_u is the bond stress at the ultimate load, N_u is the ultimate load, b is the internal width of the square steel tubes, and l_i is the interface length between concrete and tube wall surface. As can be seen from Figure 4 (a) for compact composite cross-section column specimens, the specimen POT-B150-T4-C0-H0-V0 without any shear connector had an average bond stress value of 0.89 N/mm^2, whereas for the specimen POT-B150-T4-C1-H0-V150 displayed a significant increase of bond stress of 2.17 N/mm^2 due to the presence of shear connectors. To investigate further the influence of shear connectors in enhancing the bond stress, more closely spaced connectors in specimen POT-B150-T4-C1-H0-V75 was used, which provided a bond strength of 3.72 N/mm^2, that is approximately 4 times the bond strength as compared to POT-B150-T4-C0-H0-V0. It is important to note that, for specimens with shear connectors, the slip of the concrete has significantly reduced, and the slip at the ultimate bond stress was attained at approximately less than 5 mm.



(a) Bond stress – slip curves for specimen with dimension 150×150×4

(b) Bond stress – slip curves for specimen with dimension 300×300×4

Figure 4: Bond stress versus slip curves for testes specimens.

As can be referred from Figure 4 (b) for the slender composite cross-section CFST column specimens, the specimen with no shear connectors POT-B300-T4-C0-H0-V0 achieved a bond stress of 0.32 N/mm^2 accompanied by a large slip of beyond 25 mm. And for the specimens with shear connectors had a significant improvement of bond stress values from 0.97 N/mm^2 to 1.31 N/mm^2, which is approximately 3 to 4 folds increase as compared to specimen without any shear connectors. In case of specimens with shear connectors, the slip at the ultimate bond stress was approximately at 3 mm. It is worth mentioning that, for specimens with closely spaced shear connectors, the load dropped beyond the ultimate load due to the local buckling of steel tube at the base of the specimens, and not due to concrete slip. The summary of ultimate force applied, slip, achieved bond stress at the peak load and the prominent failure modes are presented in Table 4. The yield load values for both the compact and slender steel cross-section ($N_{y,steel}$) are also included in Table 4 for comparison of steel capacity with respect to the ultimate load values.

Table 4: Summary of bond stress test results.

Specimen	$N_{y,steel}$ (kN)	N_u (kN)	τ_u (N/mm^2)	S_u (mm)	Governing failure mode
POT-B150-T4-C0-H0-V0	1226	241.3	0.89	19.65	Concrete slip-out
POT-B150-T4-C1-H0-V150	1226	586.6	2.17	5.40	Deformation of connectors
POT-B150-T4-C1-H0-V75	1226	1004.6	3.72	4.85	Local buckling of tube at base
POT-B300-T4-C0-H0-V0	2486	380	0.32	26.34	Concrete slip-out
POT-B300-T4-C1-H0-V150	2486	1133.1	0.97	4.36	Shearing-off of connectors
POT-B300-T4-C1-H0-V75	2486	1258.7	1.07	5.27	Local buckling of tube at base
POT-B300-T4-C2-H100-V150	2486	1533.8	1.31	3.72	Local buckling of tube at base
POT-B300-T4-C2-H100-V75	2486	1310	1.12	3.67	Local buckling of tube at base

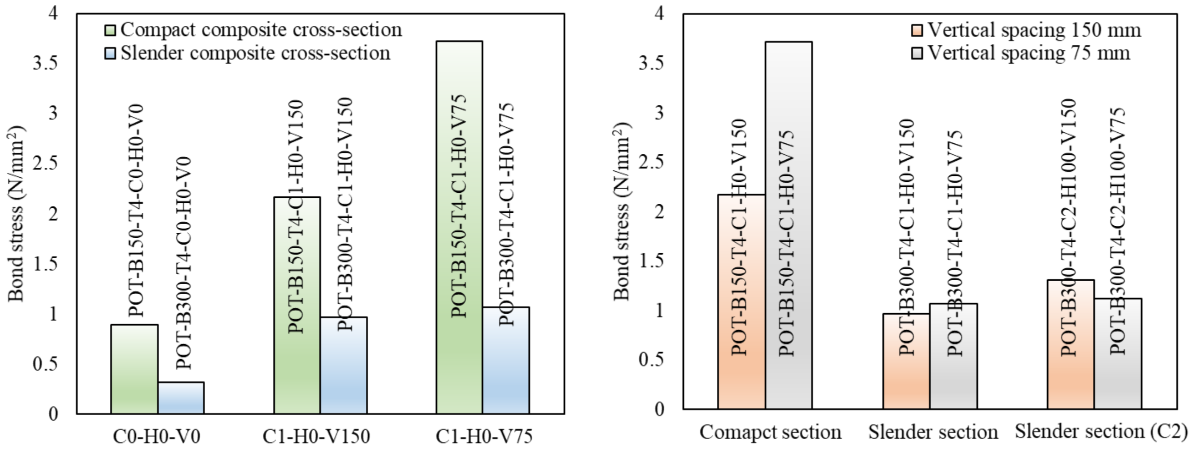
4 ANALYSIS AND DISCUSSION

4.1 Effect of cross-section

The effect of composite cross-section influencing the interfacial bond stress is presented in Figure 5(a). The compact section specimens exhibited higher bond stress irrespective of presence of shear connectors and the internal arrangement of the connectors. The compact specimen POT-B150-T4-C0-H0-V0 showed approximately 2.8 times higher bond stress as compared to the slender section POT-B300-T4-C0-H0-V0, which is possibly due to the higher steel confinement of 1.35 in the compact section, as compared to 0.65 in the slender section. When compared with presence of shear connectors, with similar vertical spacings, the compact sections had 2.2 times to 3.4 times higher bond stress as compared to the slender counterparts. This indicates that the design bond stress of 0.7 N/mm^2 and 0.4 N/mm^2 as suggested in the American [13] and European code [16] code may only be applicable for compact sections, whereas for slender sections a further conservative value may be adopted. On the other hand, with the use of shear connectors, the bond stress values for slender sections can be improved to approximately 1.0 N/mm^2 and beyond, as was observed from specimen POT-B300-T4-C1-H0-V150.

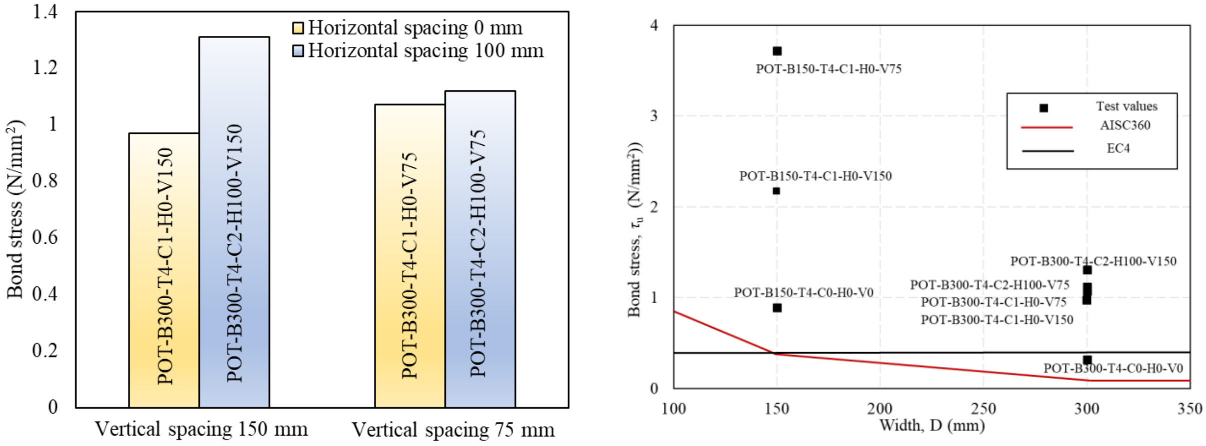
4.2 Effect of connector vertical spacing

The specimens with shear connectors having vertical spacing of 150 mm and 75 mm were compared to analyse its influence on the bond stress behaviour, and the comparison is presented in Figure 5 (b). For compact section, when the shear connectors are spaced at an interval of *B*/2 (75 mm) for specimen POT-B150-T4-C1-H0-V75 there is a significant increase in bond strength by almost 70% as compared to specimen POT-B150-T4-C1-H0-V150, where the connectors are spaced at an interval of *B*. On the other hand, for the slender sections, the enhancement in bond stress due to connector vertical spacing of *B*/4 (75 mm) is only by 10% as compared to specimen having connector spacing of *B*/2 (150 mm). This is due to significant amount of load transfer from the concrete to the steel tube via the densely placed headed anchored shear connectors, and as a result the tube failed by local buckling at the base. Interestingly, for the slender cross-section specimens having two columns of shear connectors, that is, having shear connectors with constant horizontal spacing of *B*/3 (100 mm) but vertical spacing of *B*/2 and *B*/4 for specimen POT-B300-T4-C2-H100-V150 and POT-B300-T4-C2-H100-V75, respectively, there is a drop-in bond stress for POT-B300-T4-C2-H100-V75 inspite of having the highest number of shear connectors, and had failed by local buckling of steel tube at the base. This indicates that the enhancement of bond stress is not always proportional to the reduced spacing of shear connectors, and therefore an optimum spacing of shear connectors should be adopted to achieve the best performance.



(a) Influence of composite cross-section

(b) Influence of connector vertical spacing



(c) Influence of connector horizontal spacing

(d) Comparison with international standards

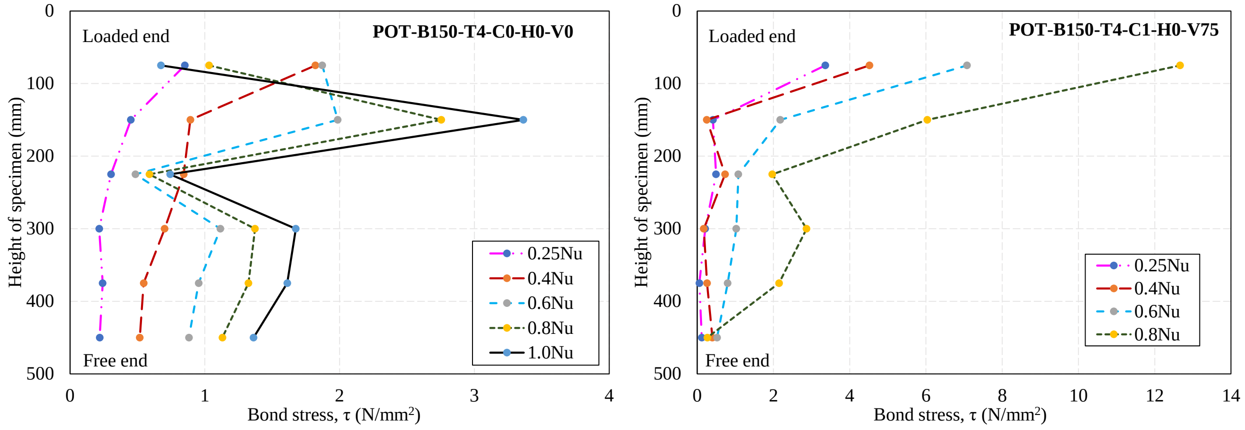
Figure 5: Influence of various parameters on bond stress of CFST columns.

4.3 Effect of connector horizontal spacing

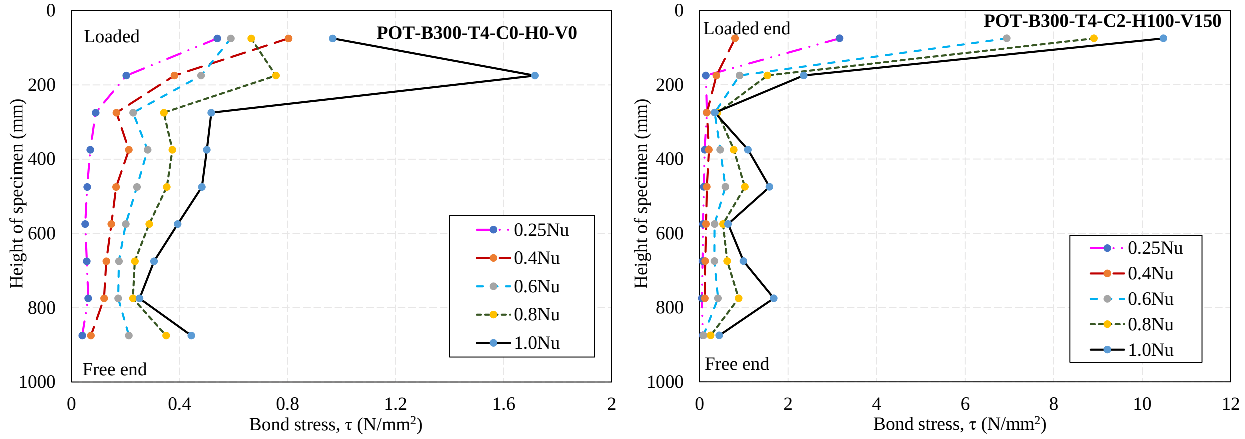
The comparison for the influence of horizontal spacing between the shear connectors is shown in Figure 5 (c). Only the slender composite CFST columns were considered for this comparison, where shear connectors placed in one column and two columns (with spacing of B/3) are presented. As can be referred, for the specimens with constant vertical spacing of *B*/2 (150 mm), there is about 35% increase in bond stress in specimen with horizontal spacing of B/3, that is, POT-B300-T4-C2-H100-V150 as compared to POT-B300-T4-C1-H0-V150. Whereas, for specimens with constant vertical spacing of *B*/4 (75 mm), the increase in bond stress was only about 4.5% for the specimen POT-B300-T4-C2-H100-V75 as compared to POT-B300-T4-C1-H0-V75.

4.4 Bond stress distribution

Strain gauges attached to the outer surface of the steel tube were used to investigate the stress distribution developed under the applied push-out forces, along the length and width of the CFST column specimens. A representative plot of the bond stress calculated from the strain data, assuming steel tube as elastic, is presented in Figure 6 for both compact and slender specimen. As can be seen from the Figure 6 (a), the compact specimen without shear connectors had mostly non-uniform bond stress beyond 40% of peak load, whereas the specimen with shear studs in Figure 6 (b) shows an improved distribution of bond stress along the length of the column, with an average of approximately 2 N/mm^2 at the mid region of the column.



(a) POT-B150-T4-C0-H0-V0 (b) POT-B150-T4-C1-H0-V75



(c) POT-B300-T4-C0-H0-V0 (d) POT-B300-T4-C2-H100-V150

Figure 6: Representative bond stress distribution for specimens without and with shear connectors.

The bond stress distribution for slender CFST specimen without and with shear connector can be seen from Figure 6 (c) and 6 (d). For the specimen without shear connectors there is an increase in bond stress with increase in applied load, but the mid-region of the column could not attain a bond stress of even 0.5 N/mm^2 at any load level. On the other hand, the specimen with headed shear connectors could achieve an average bond stress of 1.0 N/mm^2 along the mid-height of the column, signifying the effectiveness of the shear connectors.

4.5 Comparison with international standards

Further, the comparison of the obtained bond stress values without and with headed shear connectors is made with the design bond stress values in the European [16] and American [13] code, as presented in Figure 5(d). The Euro code (EC4-2004) suggests a design bond stress of 0.4 N/mm^2 for square CFST columns. Whereas, the American code (AISC360-22) suggests the minimum of 2100 t/D^2 and 0.7 N/mm^2 for square composite members. As can be referred from Figure 5(d), for tested specimens without shear studs, both EC4-2004 and AISC360-22 design bond stress value can be adopted safely for compact sections, but EC4-2004 overestimates the bond stress for slender section. For the specimens with shear studs, all of them had bond stress higher than the design values in the international codes, suggesting use of connectors for enhancing the bond stress. More data is required for developing prediction equation for bond strength in CFST columns with shear connectors.

5 CONCLUSIONS

A laboratory testing programme was carried out to investigate the interfacial bond stress between steel tube and concrete core for CFST columns. The primary aim was to study the influence of the shear connectors in enhancing the bond stress performance for CFST columns fabricated with thin tubes. In this study, both compact and slender composite cross-sections were considered, including the type of interface and arrangement of shear connectors. Push-out tests were conducted to determine the bond stress of the specimens. It was observed that, compact composite cross-sections had higher bond stress as compared to slender composite cross-section columns. Use of shear connectors have led to significantly enhance the bond stress by 4 times for composite sections with reduction in slip. For the slender sections, the minimum use of shear connectors with vertical spacing of $B/2$ can lead to approximately 1.0 N/mm^2, which is higher than the current design bond stress values in the international standards. The enhancement of bond stress is not always proportional to the reduced spacing of shear connectors, as it can lead to early local buckling of steel tube. To achieve the optimum performance, a proper combination of horizontal and vertical spacing of the shear connectors is required, and further studies for its design is currently under progress.

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