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Numerical Analysis of the Bond Behaviour of High-Strength Concrete-Filled Steel Square Columns with Different Shear Connectors

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Abstract: This paper deals with a numerical method of analysis of the performance of shear connectors in transferring load in high-strength concrete-filled steel tube square sections. The novelty of the model is that it considers all the important parameters that affect performance at once: bond strength, the transfer rate of each connector, and the stress distribution and deformation of each element. Four specimens fabricated using different types of connectors were validated using ABAQUS version 2017 software. The deformation of the connectors, concrete damage, and the local instability of the steel tube were extensively investigated. The main parameters considered were the ultimate bond strength and load transfer ratio. The shear connector arrangement consisting of four specimens, namely C1 with 16 studs, a circular rib (C2), a circular rib with 8 studs (C3), and a circular rib with 8 vertical ribs (C4), had a significant influence on the key parameters. Connectors C2, C3, and C4 transferred more than 80% of the total load. The circular rib was more effective in transferring the load and limiting slip than the vertical rib and the studs. The circular rib (C2) transferred the load mainly through the four corners. The deterioration of the concrete and local instability of the steel tube had complex deformations which were influenced by the geometry of the inserted connectors.

Keywords: concrete-filled steel tube; connectors; bond strength; load transfer; circular ribs



Citation: Boudali Errebai, M.; Ihaddoudene, A.N.T.; Saidani, M. Numerical Analysis of the Bond Behaviour of High-Strength Concrete-Filled Steel Square Columns with Different Shear Connectors. *J. Compos. Sci.* **2024**, *8*, 443. <https://doi.org/10.3390/jcs8110443>

Academic Editors: Ying-Liang Chen and Pai-Haung Shih

Received: 7 September 2024

Revised: 22 October 2024

Accepted: 25 October 2024

Published: 30 October 2024



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1. Introduction

Concrete-filled steel tube (CFST) columns exhibit many advantages compared to their ordinary steel or reinforced concrete counterparts. The force can be transferred between the steel tube and the concrete core either by natural bond strength resulting from three mechanisms, namely chemical bonding, microlocking, and macrolocking [1], or by mechanical shear connectors. This interface is a key issue for understanding the behaviour of CFST columns [2–4].

1.1. Research Motivation

Concrete-filled steel tubes enjoy a very wide range of applications in civil engineering, such as tall buildings, bridges, retaining walls, and underground structures, to name a few. Therefore, understanding their behaviour will be key in adequately and efficiently designing them. The current research aims to contribute to developing this understanding to add to the existing body of knowledge by adopting an approach that will encompass all the key parameters into one model.

1.2. Bond Strength and Shear Stress Transfer

Several research studies were carried out consisting of experimental testing to investigate how shear stress is transferred and the parameters influencing the bond strength.

Tao et al. [5] conducted a series of push-out tests on circular and square concrete-filled steel tubes. The results indicated that the bond strength depended on parameters such as cross-section, steel tube, and concrete type and age. Starossek and Falah [1] concluded from their research that the load transferred increases with the increasing length of the specimens, and the bond strength results varied between 0.8 and 1.0 MPa. Qu et al. [6] found that bond strength is greatly influenced by the compressive strength of the concrete and the section size of the steel tube. Aly et al. [7] indicated that by using normal strength concrete, the bond strength is higher than that of high-strength concrete. Wang F. [8] found that bond strength decreases with the increasing age of concrete and increases with the surface roughness of the steel tube. Chen et al. [9] concluded that the diameter-to-thickness (D/t) ratio has an insignificant influence on the shear strength.

1.3. Mechanical Shear Connectors

The most effective method to increase bond strength is to use mechanical shear connectors [5,10,11]. Many studies have been carried out to investigate the bond strength of CFST columns with different types of connectors. Dong et al. [12] and Alatshan et al. [13] assumed that welding internal rings is the most effective method to enhance bond strength. De Nardin and El Debs [14] investigated a series of CFSTs with connectors and concluded that the angles and studs as mechanical shear connectors were very effective in increasing the bond strength and transferring higher load rates between the steel and the concrete. To improve load transfer, Starossek and Falah [1] recommended using four bolts on each side. Dai et al. [15] concluded by tests that bond strength increases significantly following the addition of rib stiffeners. Ghannam and Metwally [16] conducted a series of numerical simulations to study the influence of three types of connectors in circular concrete-filled double tube columns. They found that the use of R stiffeners (rectangular plates) or T stiffeners welded through the whole column length significantly increased the bond strength, whereas bar stiffeners did not have much effect. Additionally, they found that a decrease in slenderness and an increase in the number of connectors can lead to an increase in bond strength of more than 15%. Soltanalipour et al. [17] observed a 12.31-fold (81.23%) increase in bond strength by the creation of crown-shaped protrusions on the steel tube. Petrus et al. [18] presented a study on bond strength in CFSTs, introducing a tap stiffener on the vertical rib. An increase of 40% in bond strength for a tap stiffener spacing of 100 mm was observed. Hasan and Ekmekyapar [19] concluded that internal rings are more effective in increasing bond strength than shear studs. Qiao et al. [20] conducted a series of push-out tests on 14 CFSTs with steel plate connectors. The results indicated that the steel plate is very effective in transferring the load between the steel tube and the concrete core. Dong et al. [21] concluded that the use of studs and circular ribs led to an increase in bond strength.

1.4. Numerical Modelling

Several researchers [3,17,18,22–25] have numerically studied bond strength in composite structures with and without connectors to gain an understanding of the bond strength at the interaction interface and the performance of the connectors in increasing the load-bearing capacity and to study their contribution to load transfer. Ghannam and Metwally [16] carried out a numerical study on the behaviour of three types of stiffeners, welded to the interior interface of the tube using surface-to-surface interaction. Falah and Starossek [1] developed a numerical model to analyse the load transfer in concrete-filled steel tubes combined with shear stud connectors using a surface-based interaction with a nonlinear spring element in the tangential direction and a contact pressure-over-closure model in the normal direction. Dos Santos et al. [23] investigated the influence of the numerical parameters of damaged concrete using a numerical approach and concluded that significant concrete compressive damage was occurring in the region around the connectors.

Alemayehu et al. [24] observed that rib plate connectors with circular holes exhibited high bond strength.

1.5. Research Contribution

The main objective of this research is to provide a novel and rigorous numerical simulation procedure to analyse the performance of connectors in transferring the load between a concrete core and a steel tube for high-strength concrete-filled steel square columns by including all the key parameters of shear connectors, the transfer rate of each connector, the bond strength, and the stress distribution and deformation of each element.

2. Materials and Methods

2.1. Numerical Models

A series of push-out tests for square specimens carried-out by Dong et al. [21] were chosen, including four configurations of shear connectors: C1 with 16 studs, a circular rib (C2), a circular rib with 8 studs (C3), and a circular rib with 8 vertical ribs (C4). These are the configurations that are the most used in construction practice. The steel size and length, yield strength (f_y), ultimate strength (f_u), and concrete strength are shown in Table 1 and Figure 1. In the figure, red colour represents the studs, the blue colour the steel tube, the orange colour the horizontal rib, the magenta colour the vertical rib, and the grey the concrete. The mechanical properties of the steel tube and connectors are given in Table 2.

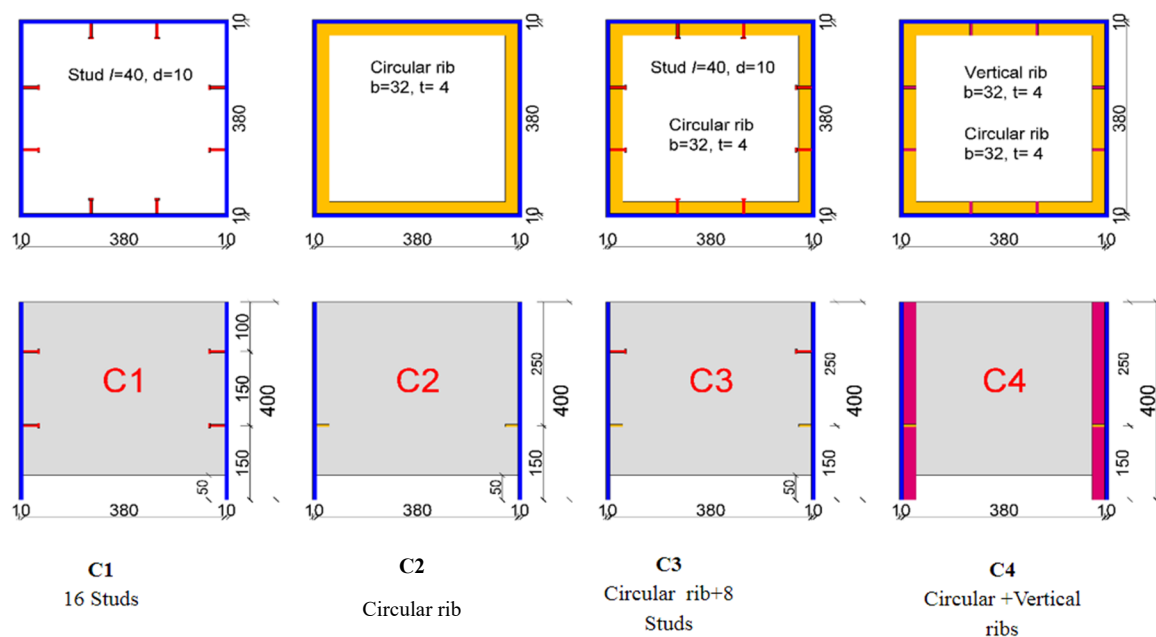


Figure 1. Properties of specimens (Dong et al. [21]).

Table 1. Properties of concrete-filled square steel (Dong et al. (2020) [21]).

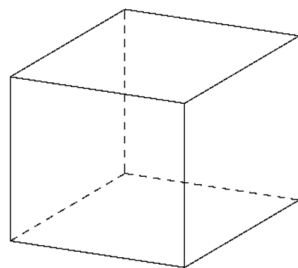
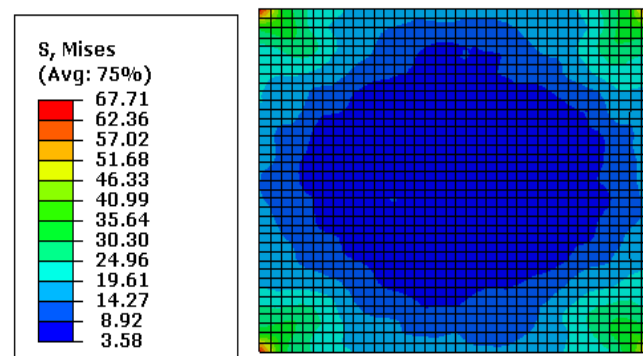
Specimen	Steel Thickness (mm)	Length (mm)	f_y (MPa)	f_c (MPa)	Shear Connector	P_{exp} (kN)	P_{num} (kN)	P_{exp}/P_{num}
C1	10	400	352.33	67.98	16 studs	1221.8	1164.8	1.04
C2					Circular rib	2607.4	2698.7	0.96
C3					Circular rib + 8 studs	3162	2987.45	1.05
C4					8 vertical + circular rib	3874.00	3686.79	1.05

Table 2. Properties of steel [21].

Elements	Steel Size (mm)	Yield Strength (MPa)	Peak Strength f_u (MPa)	Elastic Modulus E_s (GPa)
Steel tube	10	352.33	498.67	203.6
Circular rib	4	369	515.33	201.3
Vertical rib	4	369	515.33	201.3
Stud	10	398.68	569.54	201.3

2.2. Element Modelling

The use of numerical methods to study the bond strength in CFST specimens is a commonly adopted technique [26]. ABAQUS software [27] is often used for this purpose. The concrete core, steel tube, and connectors were simulated using the three-dimensional eight-node linear brick element C3D8R, with three degrees of freedom at each node as shown in Figure 2. The brick elements were found to be more effective at simulating the non-linear behaviour of the CFST in particular, as they can capture stress gradients, deformation effects, and contact interactions more accurately than shell elements [3,23,25,28–31]. The numerical simulation of the push-out test is based on the application of a load, in the form of a uniform displacement, at the top of the concrete core through a non-deformable rigid plate (R3D4), with a section slightly smaller than the concrete section to exert pressure on the upper surface of the concrete core only to push it through the steel tube. This pressure is greater in the areas near the four corners than in the central part, as shown in Figure 3. To allow the concrete core to move downwards, a space of 50 mm is designed at the bottom end between the outer stainless steel tube and the concrete core.

**Figure 2.** C3D8R linear brick.**Figure 3.** Specimen C2 pressure on the top surface of the concrete at post-peak load.

ABAQUS software [27] allows the application of boundary conditions at a single point RP that connects all surface nodes to only one reference by the constraints option available. A controlled displacement mode is applied to the rigid plate at reference point RP1 at the top of the column; the boundary condition is applied to restrain the concrete core $\delta x = \delta y = 0$, allowing the displacement to take place in the z direction, and the steel tube is

restrained against the displacement $\delta x = \delta y = \delta z = 0$ at the bottom at RP2. The connectors are welded to the steel tube by the tie option. The mesh of 10 mm gives a good convergence, as shown in Figures 4 and 5.

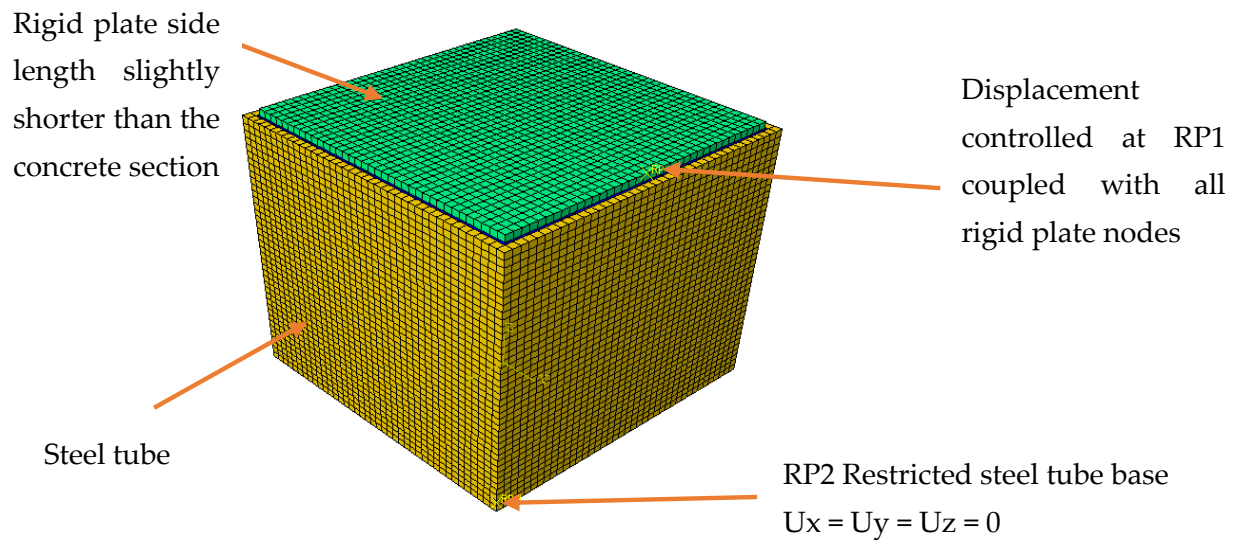


Figure 4. Finite element mesh and boundary condition of concrete-filled steel tube column.

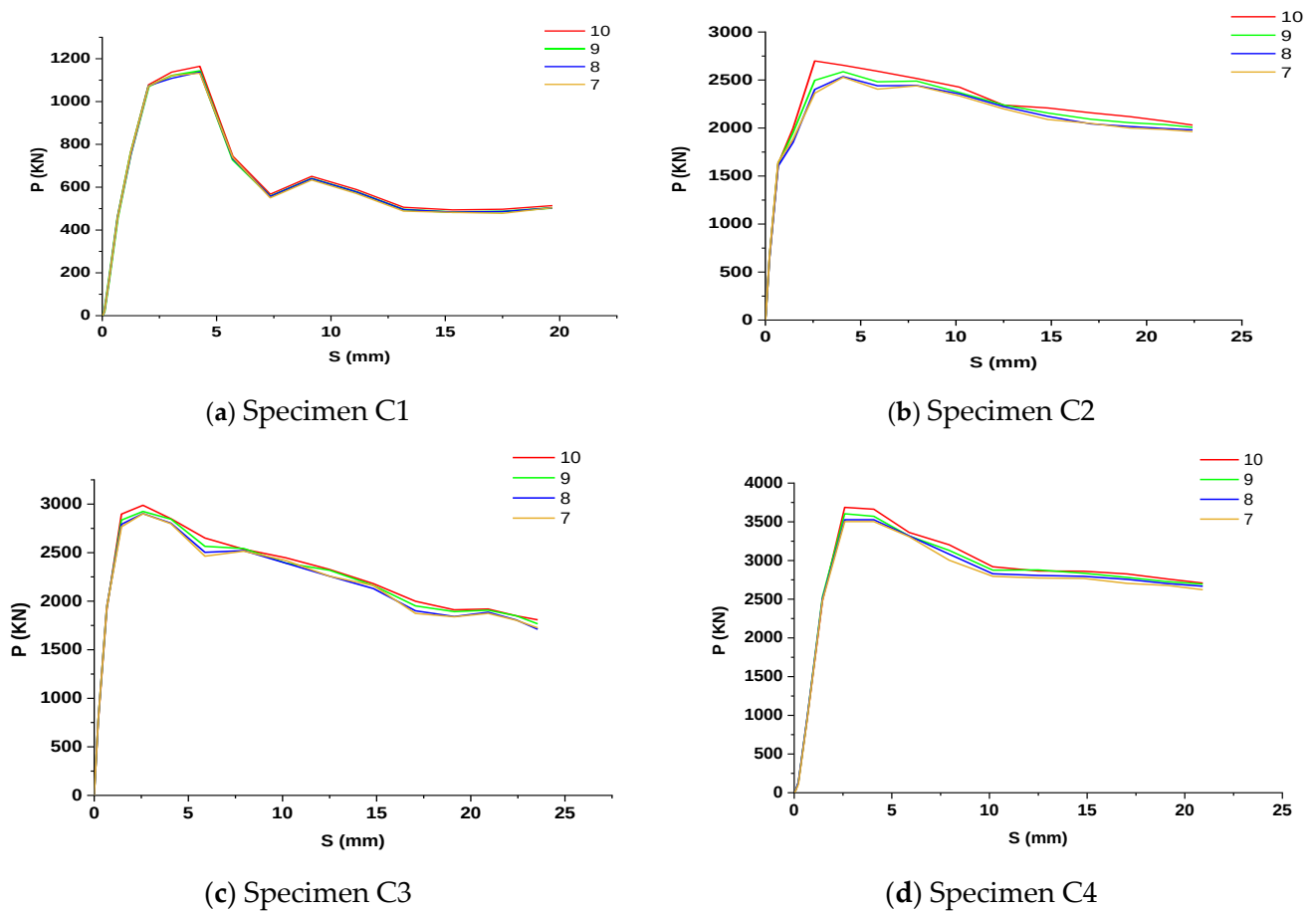


Figure 5. C1, C2, C3, and C4 load–slip behaviour comparison for different mesh sizes.

A mesh sensitivity study was undertaken for all four specimens, C1, C2, C3, and C4, for mesh sizes 7 mm, 8 mm, 9 mm, and 10 mm. Figure 5 shows the results of the sensitivity test, and Table 3 gives the peak load resistance for different mesh sizes. As can be seen from the table, the size of the mesh had little influence on the results of the peak load resistance, giving the following maximum difference between the different mesh sizes for each specimen: 3% for specimen C1, 6% for specimen C2, 3% for specimen C3, and 5% for specimen C4. The results clearly show that a mesh size of 10 mm gives good convergence for all four specimens.

Table 3. Ultimate load capacity for different mesh sizes for each of the specimens.

Mesh Size	Specimen	Peak Load Resistance (kN)
10 mm mesh	C1	1164.8
	C2	2698.7
	C3	2987.45
	C4	3686.79
9 mm mesh	C1	1137.32
	C2	2606.26
	C3	2924.13
	C4	3603.94
8 mm mesh	C1	1132.69
	C2	2560.68
	C3	2902.28
	C4	3526.94
7 mm mesh	C1	1132.30
	C2	2545.82
	C3	2901.72
	C4	3505.42

Different coefficient values of friction have been proposed between 0.25 and 0.6 [16,22–24,28,30–33]. In Figure 6, the load–slip behaviour of specimen C2 is plotted for three coefficients of friction, 0.25, 0.3, and 0.35, for comparison. The coefficient value of 0.3 shows a good convergence compared to the experimental results [28].

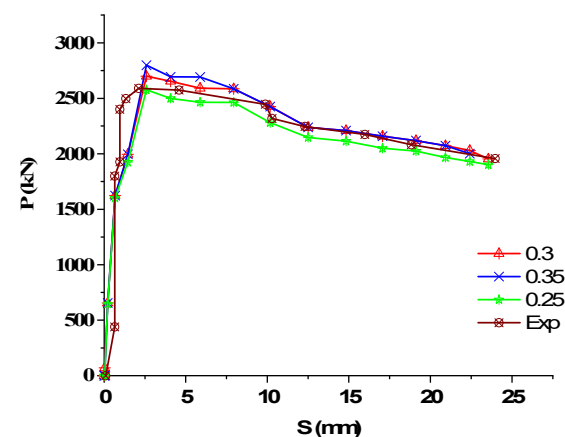


Figure 6. C2 load–slip behaviour comparison for different coefficients of friction.

The interaction between the concrete core and the steel tube was defined based on the general contact in ABAQUS. Two types of contacts were used: (a) hard contact with the sliding option in the normal direction and (b) penalty contact in the tangential direction. This option allows for defining contact between some or all regions of a model with a single interaction and permits separation and slip between the concrete and the steel tube [27].

2.3. Concrete Confined Modelling

The concrete damaged plasticity model was used to simulate concrete behaviour under compressive and tensile stresses. The model is a three-dimensional continuum plasticity-based damage model [34]. It assumes that the main two failure mechanisms are tensile cracking and compressive crushing of the concrete material [27].

Several analytical models describing the behaviour of concrete damage plasticity have been proposed. Our experimental tests were carried out with square sections for high-strength concrete as we chose the confined concrete modelling procedure proposed by Thai et al. [32], which accounts for the confinement effect of concrete infill, residual stresses, and initial local imperfections for confined concrete.

Figure 7 shows the confined and unconfined concrete, where f_c (MPa) and ϵ_c are the compressive strength and strain of the unconfined concrete, f_0 (MPa) and ϵ_0 are the peak stress and the corresponding strain of the confined concrete, and f_{re} (MPa) is the residual stress after the peak [32].

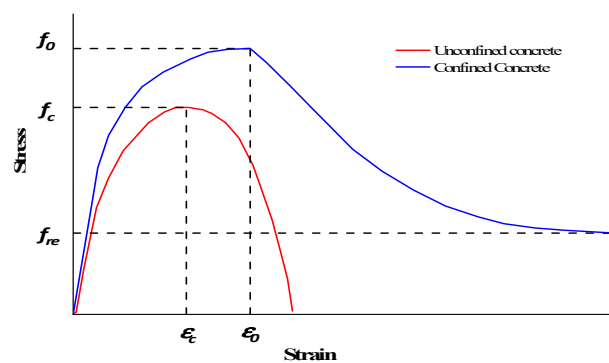


Figure 7. Stress–strain for confined and unconfined concrete [32].

The confining pressure on the concrete was estimated using an empirical equation. The residual stress of the confined concrete f_{re} was calculated as $0.1 f_c$ [33]. Equation (1) represents the strain at the peak stress of the unconfined concrete ϵ_c , proposed by Tasdemir et al. [35].

The peak stress f_0 and corresponding strain ϵ_0 were calculated from Equations (2) and (3), respectively, proposed by Xiao et al. [22].

$$\epsilon_c = (-0.067f_c^2 + 29.9f_c + 1053) \times 10^{-6} \quad (1)$$

$$\frac{f_0}{f_c} = 1 + 3.24 \left(\frac{f_r}{f_c} \right)^{0.8} \quad (2)$$

$$\frac{\epsilon_0}{\epsilon_c} = 1 + 17.4 \left(\frac{f_r}{f_c} \right)^{1.06} \quad (3)$$

The formula for the confining pressure on the concrete f_r was proposed by Thai et al. [32], as shown in Equation (4):

$$f_r = \frac{(-42428 + 236f_y)e^{-0.04(B/t)}}{7773 + f_c^{1.6}} \quad (4)$$

where B and t (mm) are, respectively, the width and thickness of the square section.

Equation (5) represents the axial stress–strain relation of the confined concrete in the pre-peak stress–strain phase, as proposed by Popovics [36] and later modified by Mander et al. [37].

$$\frac{f}{f_0} = \frac{(\epsilon/\epsilon_0)^r}{r - 1 + (\epsilon/\epsilon_0)^r} \quad (5)$$

$$r = \frac{E_C}{E_C - (f_0/\varepsilon_0)}$$

where the elastic modulus of the concrete [38] is as given below:

$$E_c = 4700 \sqrt{f_c} \quad (\text{MPa})$$

The stress–strain curve of the confined concrete post-peak proposed by Binici [39] is given by Equation (6) below:

$$f = f_{re} + (f_c - f_{re}) \exp \left[- \left(\frac{\varepsilon - \varepsilon_0}{\alpha} \right)^\beta \right] \quad (6)$$

where the factors α and β are given by Tao et al. [33]:

$$\alpha = 0.005 + 0.0075 \cdot \xi_c$$

$$\beta = 0.92$$

where ξ_c is the confinement factor:

$$\xi_c = \frac{A_s \cdot f_y}{A_c \cdot f_c}$$

The flow potential eccentricity and viscosity parameter are 0.1 and 0, respectively [29]. Equations (7)–(9) give the dilation angle Ψ , the ratio of biaxial to uniaxial compressive strength f_{b0}/f_c , and the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian K_c , respectively, proposed by Tao et al. [33].

$$\frac{f_{b0}}{f_c} = \frac{1.5}{f_c^{0.075}} \quad (7)$$

$$\Psi = 40^\circ \quad (8)$$

$$K_c = \frac{5.5}{5 + 2f_c^{0.075}} \quad (9)$$

2.4. Steel Tube Modelling

The plastic option in ABAQUS was used to model the steel material by defining its elastic and plastic mechanical properties. The stress–strain curve of the steel tube was assumed to be elastic–perfectly plastic. The elastic modulus and Poisson's ratio were, respectively, 200 GPa and 0.3 [29]. This value of elastic modulus was chosen as it is the closest whole number value to the values of the elastic modulus of the steel (203.6 GPa) and that of the connectors (201.3 GPa).

3. Results and Discussion

3.1. Verification of Finite Element Models

The numerical results obtained were compared to the experimental results presented by [21], consisting of two main comparisons of peak load strength and axial load capacity vs. slip curves. Based on Figure 8, the numerical results were found to be in good agreement with the experimental test results for the peak–load, with a small difference of 5%. For each specimen, the trend of the two curves is similar. For all the specimens, the bond strength reached the highest value with small slip. Specimen C4 presented the highest peak load of 3.68 MN, followed by specimen C3 which recorded a value of 2.98 MN for a slip value of 2.58 mm. For specimen C2, a higher peak load value of 2.69 MN was observed for a slip value of 2.58 mm, and the load then started to gradually decrease with increasing slip.

For specimens C1 and C3, which contained shear stud connectors, the load decreased after the peak load to residual values of 0.5 MN and 2.4 MN, respectively, with the rapid increase in the slip [21].

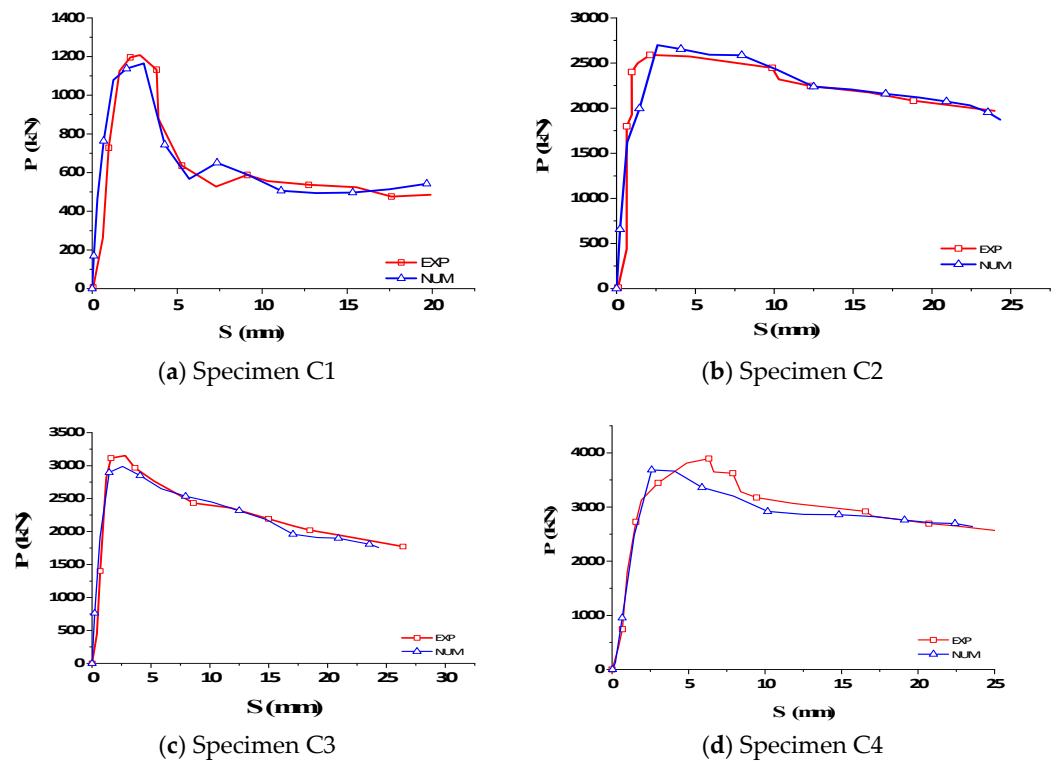


Figure 8. Numerical and experimental load-slip behaviour comparison.

3.2. Modes of Failure

The deformation of the shear connectors depended on their geometry and mechanical characteristics. The deformation of the steel and concrete was largely influenced by the geometry of the inserted connector. In the following sections, the failure of each member is discussed.

3.2.1. Stud Connectors (C1)

Figure 9 shows the stresses in the two rows of studs (upper and lower) for several peak load levels (from 61% to 100% P_u). At 0.61 P_u , the welded sections were subjected to very high stresses of 412.4 MPa for a slip value of 0.67 mm, and the studs started to twist. At the peak load P_u for a slip of value 3.03 mm, all the studs reached their ultimate strength of 569.54 MPa, indicating failure of connection at the welded sections.

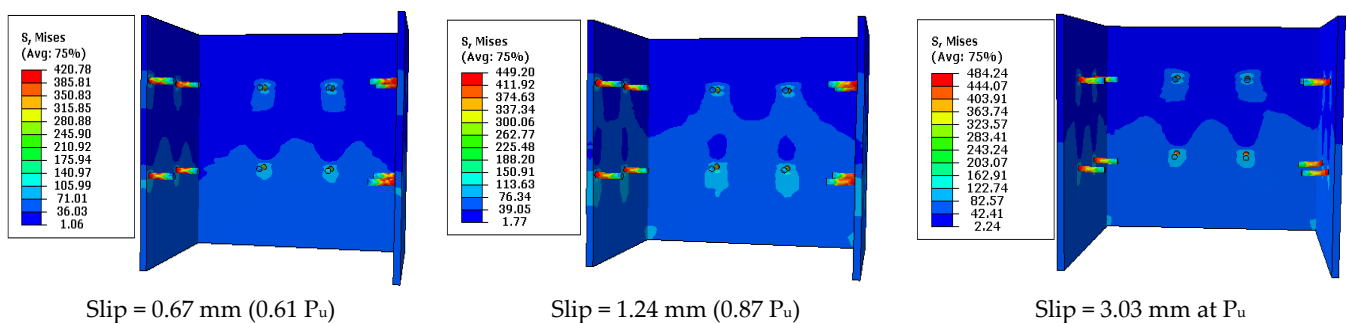


Figure 9. Stresses of studs at different peak load levels for specimen C1.

3.2.2. Circular Rib (C2)

Figure 10 shows the variation in the stress vs. length of the circular rib for one side at several load levels (from 2.6% to 100% P_u). At the start of the test, the circular rib stress started to increase for the first slip in the corners compared to the central part. At P_u , the circular rib reached its stress yielding limit of 369 MPa and the peak stress of 515.3 MPa at post-peak bond strength from the four inside corners (Figure 11).

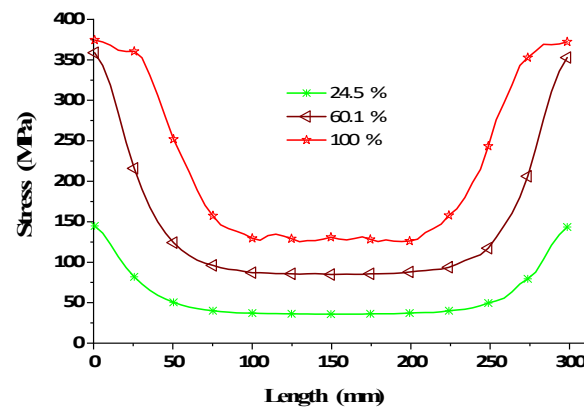
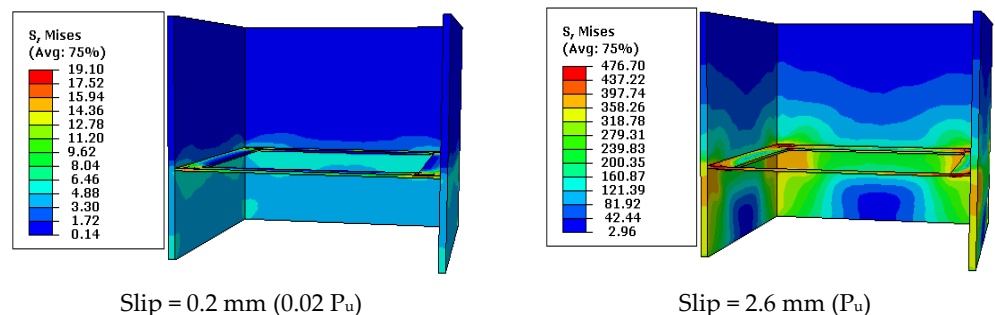


Figure 10. Stresses in the circular rib at different load levels for specimen C2.

(a) Stress pre-peak



(b) Stress post-peak

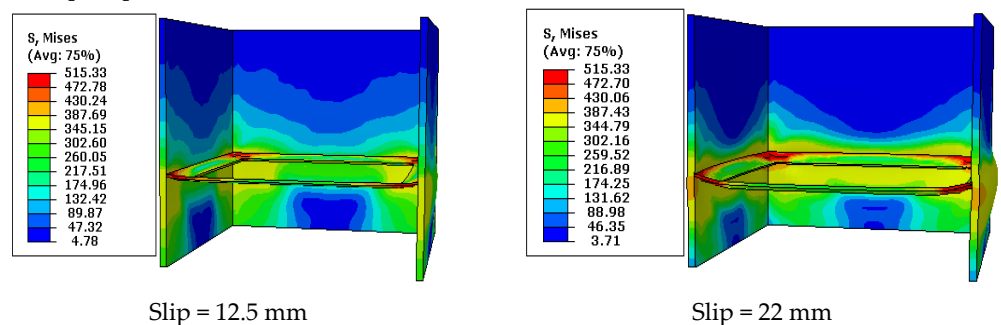


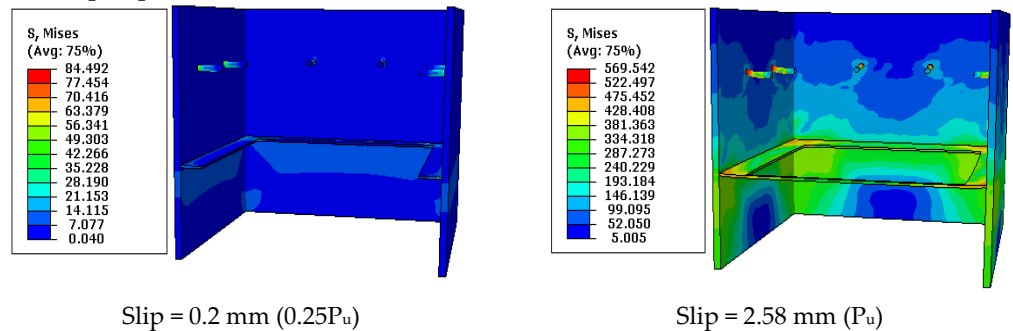
Figure 11. Specimen C2 stresses in the circular rib at different peak load levels and slip values.

3.2.3. Circular Rib with Studs (C3)

This model was composed of eight studs combined with a circular rib. This combination is important to improve the peak bond strength. The studs reached their peak limit strength of 569.5 MPa at P_u (2.58 mm) before failure (see Figure 12). While the bearing capacity of the column gradually decreased after the peak bond strength, the circular rib continued to transfer the loads principally from the four corners, which reached their strength limit post-peak. The central part underwent buckling in the elastic phase. In Figure 13, the curves of specimens C2 and C3 are identical, except at the peak bond

strength. The studs in specimen C3 increased the peak bond strength limit to 5.62 MPa before reaching failure.

(a) Stress pre-peak



(b) Stress post-peak

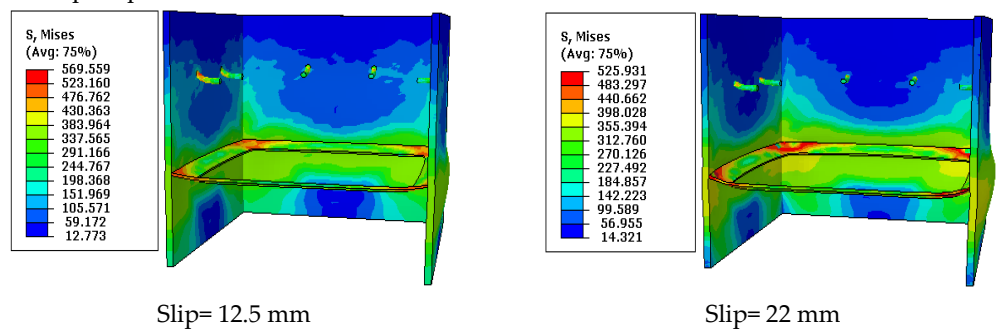


Figure 12. Specimen C3 stresses in the circular rib and the studs at different peak load levels and for different slip values.

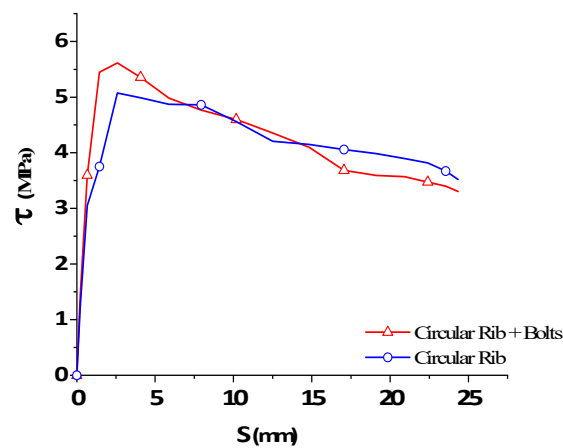


Figure 13. Bond strength–slip behaviour in specimens C2 and C3.

3.2.4. Circular and Vertical Ribs (C4)

The combination of circular ribs with vertical ribs was the most optimal combination to improve bond strength, reaching a value of 6.93 MPa. From Figure 14, it is observed that at $0.7P_u$, the lower vertical rib reached its elastic limit of 369 MPa. At peak load P_u , both parts of the vertical rib close to the circular rib reached their yielding stresses. With increasing slip, the circular rib and the lower vertical rib reached their peak strength limit of 515.33 MPa, mainly in the four corners.

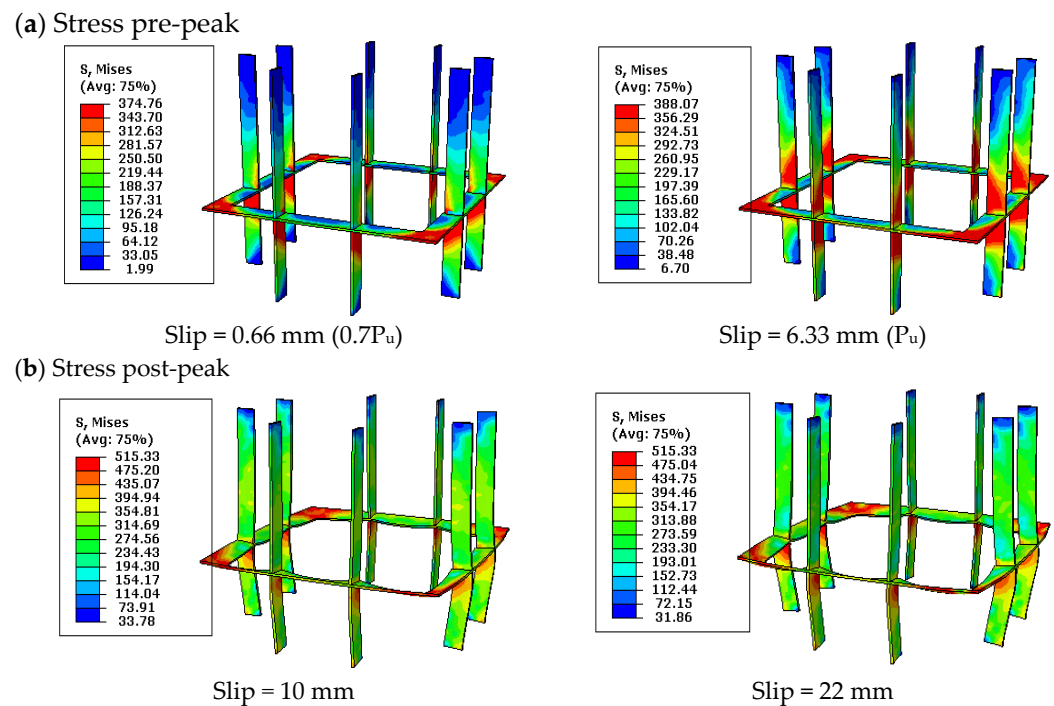


Figure 14. Specimen C4 stresses in the circular and vertical ribs at several peak load levels and slip values.

3.2.5. Concrete

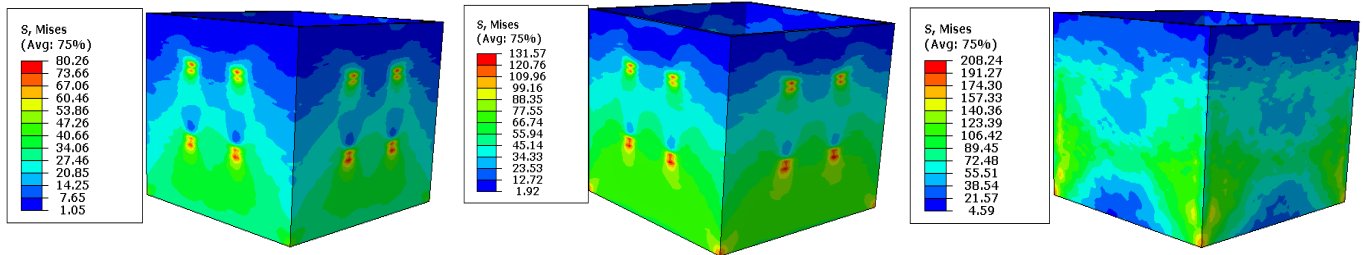
For all models, the concrete faced compression damage, especially in the upper part around the connectors, by varying forms. A value of zero “0” means no damage and a value of “1” indicates crushing of the concrete [29]. The concrete degradation for specimen C1 with studs is illustrated in Figure 15. At the start of the test, local crushing appeared at the upper part of the studs’ positions. At P_u , the degradation increased in circular form due to the rotation of studs. After the peak load, the degradation of the concrete stopped because the studs lost connection with the steel tube. For specimen C2 with a circular rib, at the start of the test, most of the concrete damage was observed in the middle of the upper position of the circular rib and continued to propagate with increasing slip. For specimen C4, greater degradation was observed at the position of the two vertical ribs.

3.2.6. Steel Tube

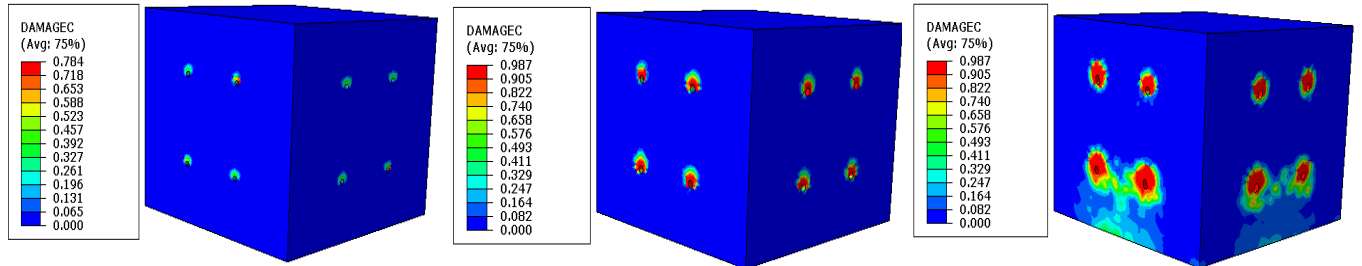
The surface of the steel tube nearer to the connector position exhibited local buckling, with different shapes depending on the geometry of the connectors. For the models with a circular rib, at P_u , the outward local buckling was very significant in the four corners of the steel and continued to propagate towards the central part with increasing slip. Meanwhile, the region close to the studs underwent very slight deformation.

Figure 15 shows the concrete and steel damage for all specimens, the values of the slip of each connector corresponding to different levels of ultimate load P_u , as well as the post-peak state.

Specimen C1: Steel



Specimen C1: Concrete

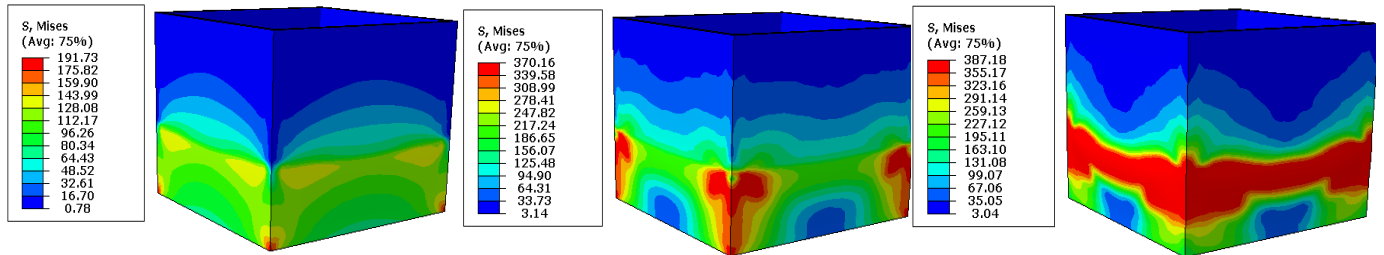


Slip = 0.3 mm (0.4 Pu)

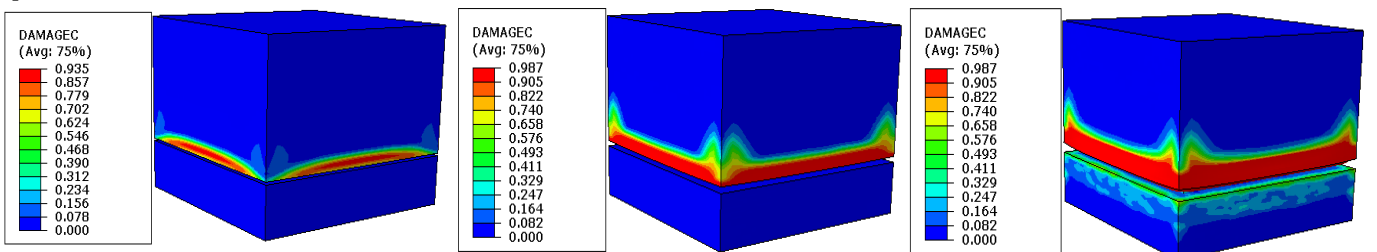
Slip = 3.03 mm (Pu)

Slip = 11.11 mm (post-peak)

Specimen C2: Steel



Specimen C2: Concrete

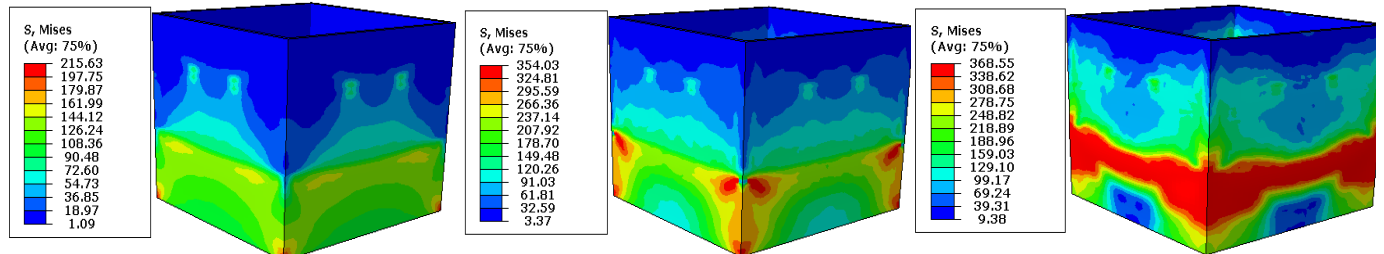


Slip = 0.66 mm (0.6 Pu)

Slip = 2.58 mm (Pu)

Slip = 18.83 mm (post-peak)

Specimen C3: Steel



Specimen C3: Concrete

Figure 15. Cont.

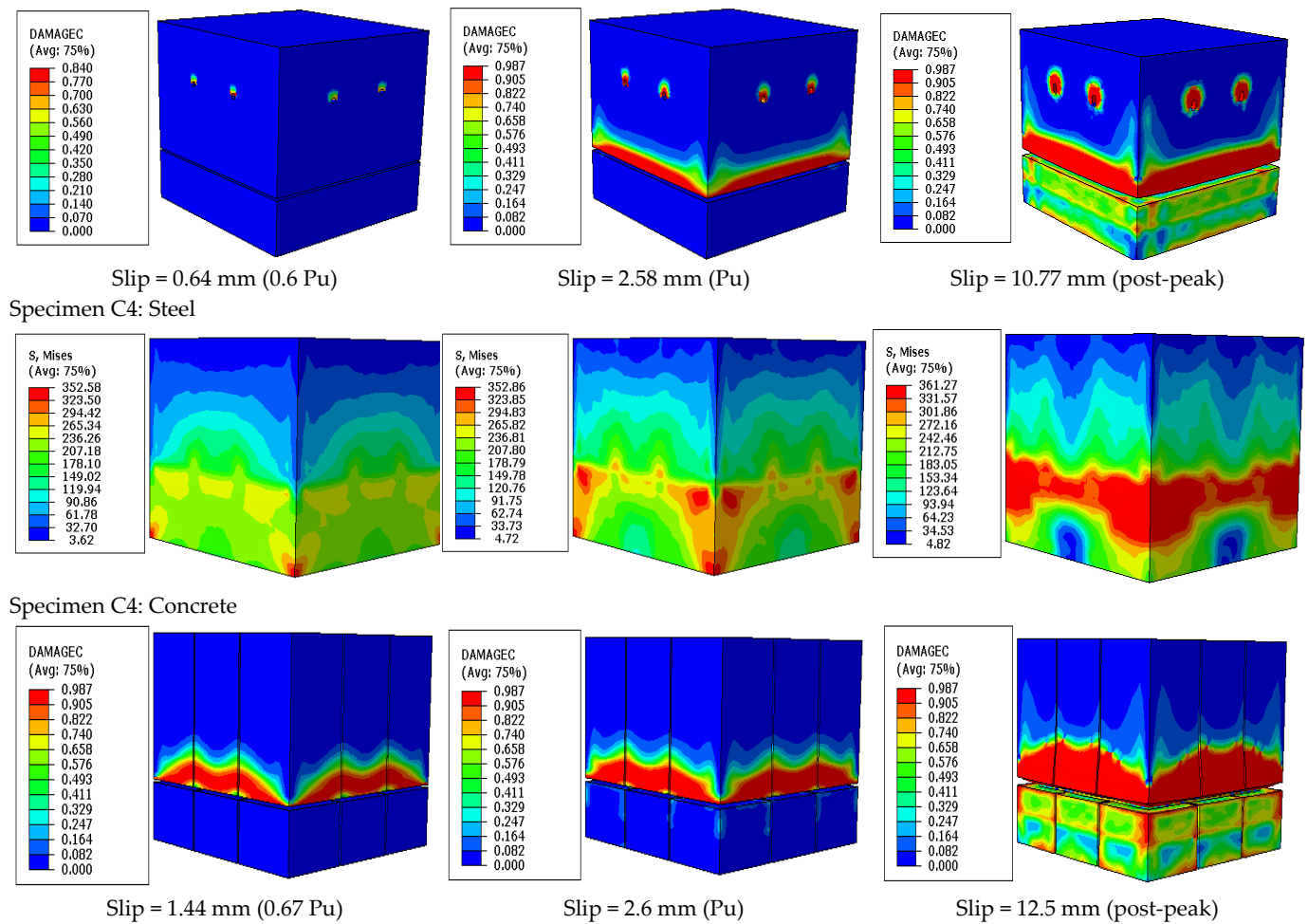


Figure 15. Damage in concrete and steel.

3.3. Distribution of the Load Along the Steel Length

The distribution of the load for each model along the steel length is presented for several peak load levels in Figure 16, with the main parameter being the shear stress transfer. The concrete supported the total load at the top of the specimen, and the steel tube supported the whole load at the end. One can observe that a large portion of the load transfer at any load level moved towards the steel tube and occurred at the connector's position.

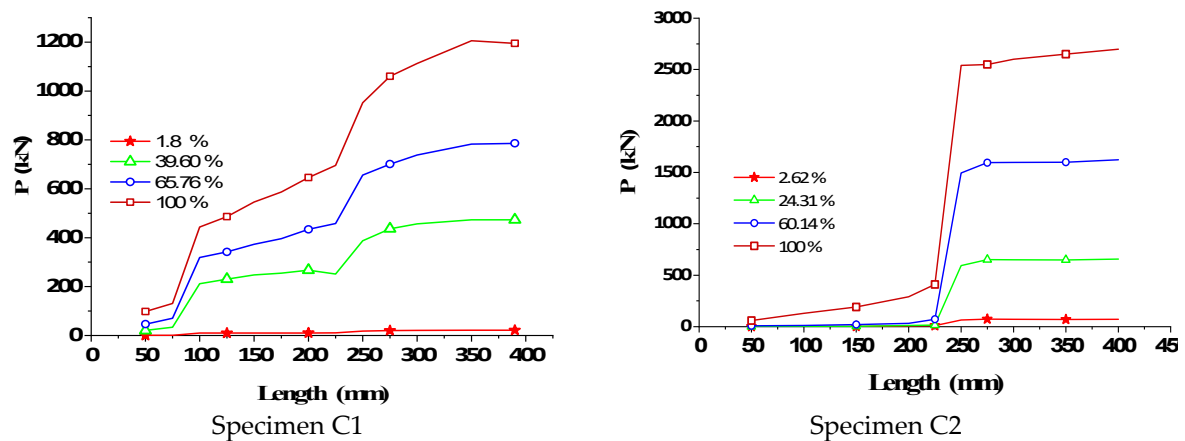


Figure 16. Cont.

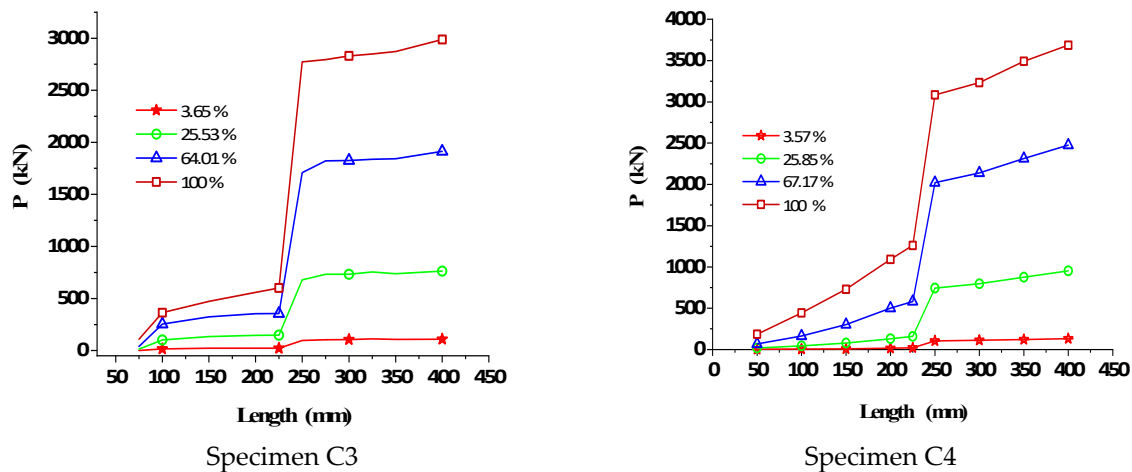


Figure 16. Load–length of the steel at different peak load levels.

For specimen C1, a larger portion of the load was transferred through the two positions of the two rows of the connectors, where the upper one was more effective than the lower row. For specimen C2, more than 3/4 of the total load was transferred at the circular rib position.

For specimen C3, 4/6 was transferred by the circular rib positioned at 250 mm at the peak load. Meanwhile, the studs ensured transferral of 1/6 of the total load, and the remaining 1/6 of the total load was transferred with natural interaction.

In addition to the performance of the four corners of the circular rib in transferring the load for specimen C4, the presence of vertical ribs limited the deformation of the circular rib mainly in the central parts, so the sliding was very limited between the steel tube and the concrete core, significantly increasing the bond strength.

About 55% of the total load was transferred by the circular rib positioned at 250 mm from the top, and the presence of the vertical rib along the column gave a significant load transfer capacity at the top and bottom.

The relationship of each connector to the peak load P_u is illustrated in terms of the rate of load transmitted compared to the total load in Figure 17.

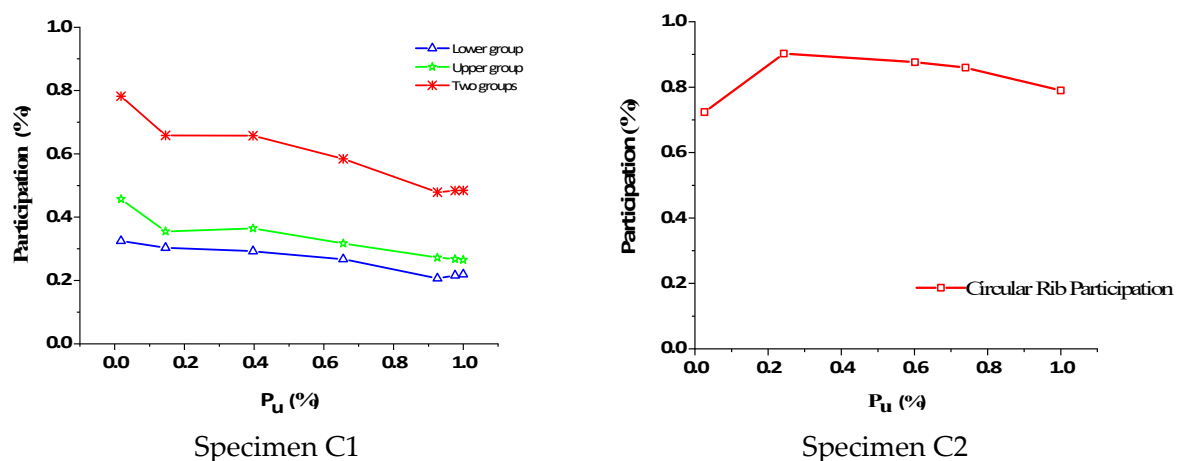


Figure 17. Cont.

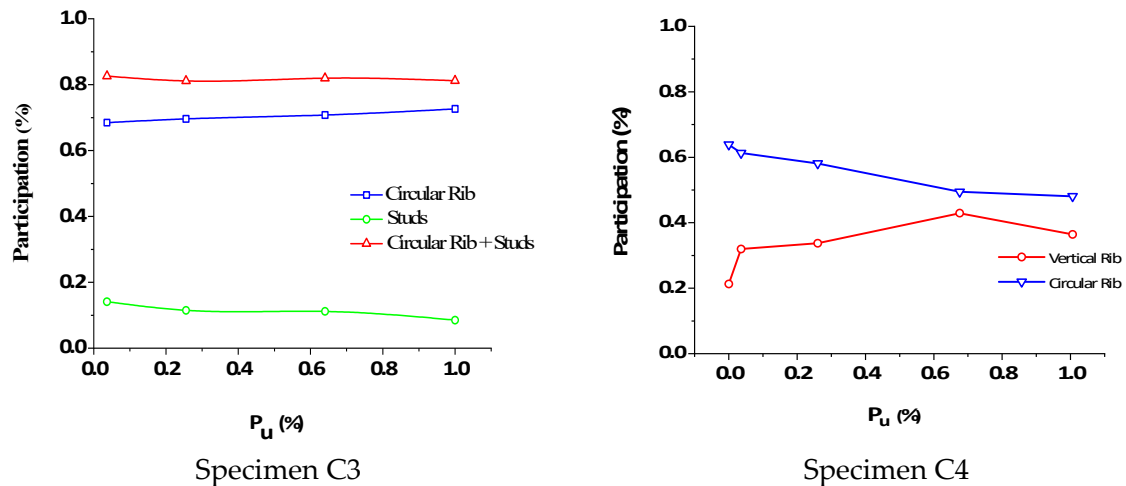


Figure 17. Connectors' participation ratios for all specimens.

3.4. Performance of Connectors

For specimen C1, the studs were found to be very effective in transferring the load at the first slip. Between 60% to 80% of the total load was transmitted by the two groups. At peak load, the participation reduced to lower values. The upper group was more effective than the lower group. After the peak load, the studs reached their strength limit and failure of the connection with the steel tube occurred at the welded section.

For specimen C2, the circular rib transferred approximately 80% of the total load at the first slip, while less than 20% of P_u was transferred by the natural bond between the concrete core and the steel tube. At 0.25 P_u , this value increased to a very high value of 90%. After this step, the load transferred decreased to about 74% at P_u .

The circular rib and studs in specimen C3 transferred a constant load up to the peak bond of 82%. The contribution of the circular rib was 4.5 times greater than the contribution of the studs at peak load. This value was mainly due to the phase of failure of the studs. At post-peak load, the circular rib continued to transfer the load alone.

The circular rib in specimen C4 was more effective at increasing the ultimate bond strength compared to the vertical rib. The contribution of the circular rib tended to decrease with increasing load, contrary to the vertical rib which recorded an increase. The combination of these two connectors was the most effective method to increase load capacity compared to the other models.

Figure 18a shows the total load transferred by each group of connectors. Specimens C2, C3, and C4 were very efficient, with an average participation rate of more than 80%, followed by specimen C1 which recorded at the first slip a very high participation value of more than 75% before showing a rapid decrease after the peak bond strength at a slip value of 7 mm.

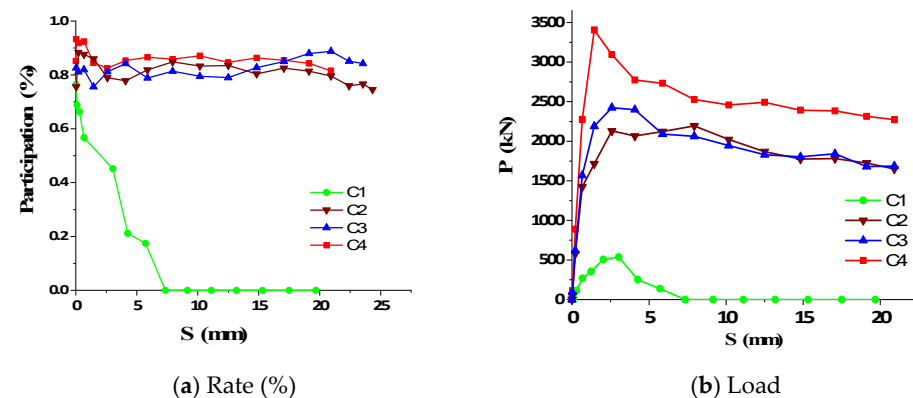


Figure 18. Connectors' transferred load.

According to Figure 18b, the circular and vertical ribs more efficiently increased the peak load, mainly due to the significant participation of the effective section. Figure 19 shows the load transfer by one side of the circular rib (C2) for different peak load levels, and one vertical rib (C4) at peak load. At P_u , the corners of the circular rib were very effective at increasing the peak bond compared to the central part. The lower side of the upper rib and the two sides of the lower vertical rib transferred greater value compared to the other sections.

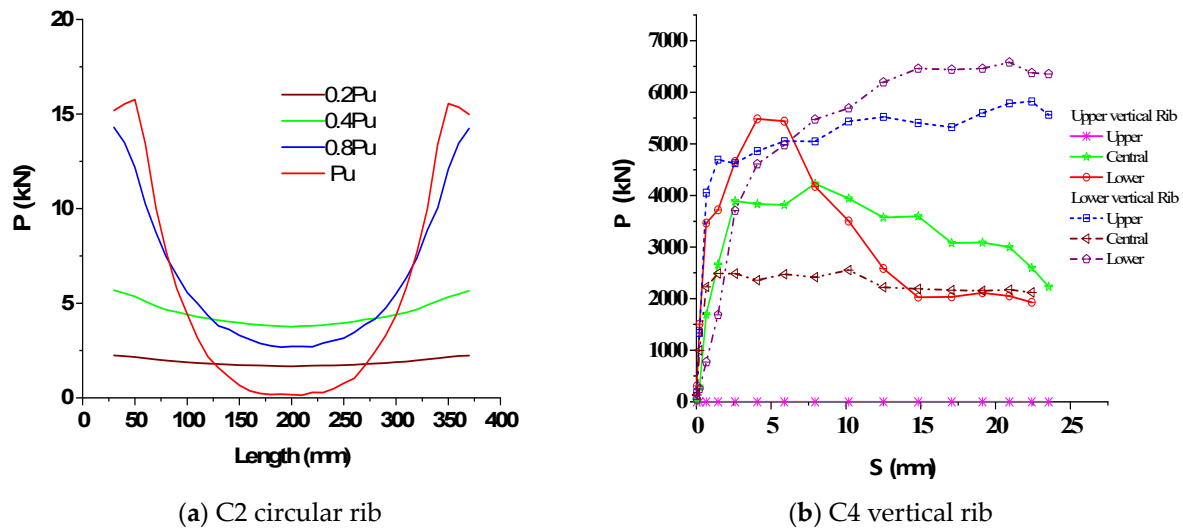


Figure 19. Load transferred along the length of the circular and vertical ribs.

The load transferred by each connector at the peak loads is summarised in Table 4.

Table 4. Summary of the parameters investigated relating to the connectors.

Specimen	Connectors	S_u (mm)	P_u Specimen (kN)	P_{cu} (kN)	τ_{cu} (MPa)	Participation (%)
C1	Studs (Upper)	3.03	1164.8	312.55	0.59	26.83
	Studs (lower)			255.94	0.48	21.97
C2	Circular rib	2.58	2698.7	2131.06	4.01	78.97
C3	Studs	2.58	2987.45	234.83	0.44	7.86
	Circular rib			2170.15	4.079	72.64
C4	Circular rib	2.60	3686.79	1823.42	3.43	49.46
	Vertical rib			1583.04	2.98	42.94

According to Figure 20 that presents the curves' bond strength transferred for the circular and vertical ribs, Figure 21 illustrates the ultimate bond strength for each connectors, the circular ribs of specimens C2 and C3 were 1.2 times more effective at increasing the peak bond strength compared to the circular rib of specimen C4 and 1.37 times more effective than the vertical rib. The circular rib of specimen C4 transferred more than 1.15 times the load transferred by the vertical rib in the same specimen. The upper row of specimen C1 was more efficient than the lower row by more than 1.22 fold.

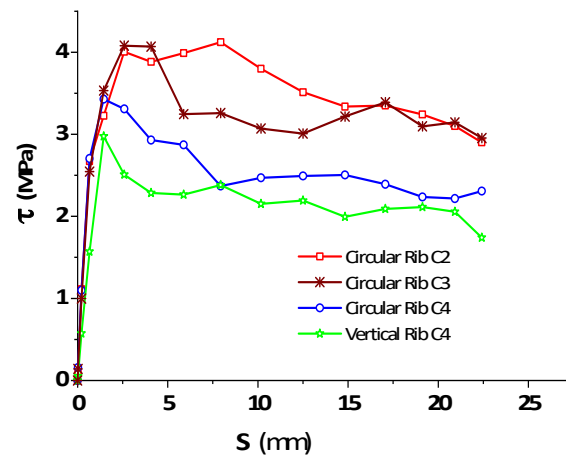


Figure 20. Bond strength curves of circular and vertical ribs.

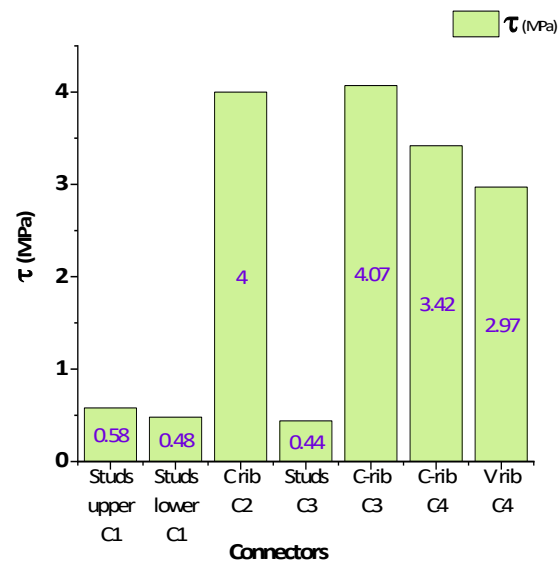


Figure 21. Peak bond strength of connectors.

3.5. Parametric Study

Several parameters related to CFST columns and the characteristics of connectors have an important impact on bond strength. Finite element analysis enables a parametric study by considering a wide range of parameters. The properties of the studied parameters is summarised in Table 5.

Table 5. Properties of the studied parameters.

<i>N</i>	Circular Rib Parameter	Properties	P_u (MN)	τ_u (MPa)	Ratio
1	Thickness (mm)	3	2.36	4.44	0.88
2		4	2.69	5.06	1
3		5	2.83	5.32	1.05
4	Stainless Steel (MPa)	$f_y = 352$	2.66	5.00	0.99
5		$f_y = 369$	2.69	5.06	1
6		$f_y = 398$	2.73	5.13	1.01
7	Position (mm)	$L = 50$	2.58	4.85	0.96
8		$L = 100$	2.69	5.06	1
9		$L = 150$	2.72	5.11	1.01
10		$L = 200$	2.7	5.08	1
11		$L = 250$	2.67	5.02	0.99

Figure 22 shows a graph of bond vs. slip for different circular rib thicknesses. The increase in the bond strength is proportional to the thickness of the circular rib. A thickness of 5 mm resulted in a bond strength of 5.32 MPa. This value was the largest for the square-shaped column and presented an increase of 16.54% compared to the circular rib of 3 mm in terms of maximum bond strength. This increase was observed until the end of the test. Figure 23 shows the bond strength for steel tubes with three yield strengths of 352, 369, and 398 MPa. The increase in bond strength is relative to the increase in yield strength of the circular rib connectors, but this increase is small. The effect of the position of the circular rib connectors on bond strength is shown in Figure 24 for five positions: 50, 100, 150, 200, and 250 mm. A maximum value of 5.11 MPa was recorded at the middle position of 150 mm, showing an increase of more than 5% compared to the lowest value of 4.85 MPa at the base of the columns at position 50 mm. The two limit positions had the lowest values.

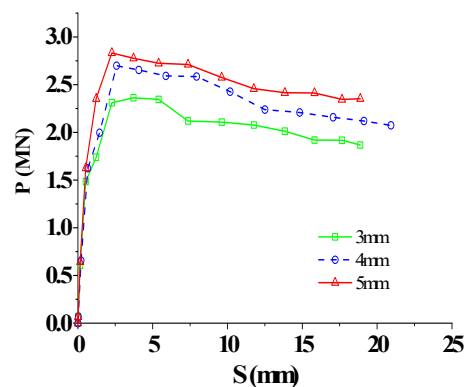


Figure 22. Load–slip curves for circular ribs of varying thickness.

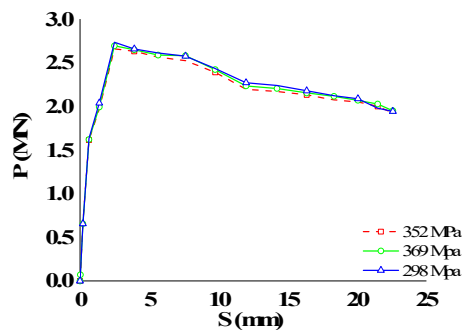


Figure 23. Load–slip curves for several stainless steel yield strength values.

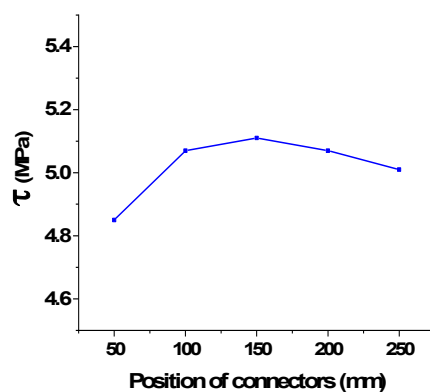


Figure 24. Load–slip curves for several positions of circular rib.

3.6. Discussion

For the stud connectors (C1), it was found that failure occurred at the welded section, suggesting that the welds were the weakest points. Indeed, these were found to be subjected to very high stresses, leading to the studs twisting prior to weld failure.

The circular rib (C2) and circular rib with studs (C3) connectors were found to exhibit similar performances, with C3 exhibiting an increased peak bond strength. This is evidence that the introduction of studs with a circular rib leads to improved peak bond strength, which is an important point to note in designing such components.

The results have shown that combining circular ribs with vertical ribs (C4) gives the optimal solution, leading to increased bond strength, reaching almost 7 MPa. This is an important finding that will assist the design process when choosing which configuration would give the optimal solution. The reason for C4's better performance can be explained by the fact that, in addition to the rigidity of the four corners of the circular rib against deformation and their ability to transfer the load, the presence of vertical ribs limited the deformation of the circular rib mainly in the central parts; consequently, the slippage between the steel tube and the concrete core was very limited, and the bond strength was considerably increased.

All models (C1 to C4) underwent concrete damage, especially in the upper part around the connectors. C1 appeared to show the most degradation of the concrete (crushing), and therefore, more attention needs to be paid when designing stud connectors to prevent failure of the welds and crushing of the concrete. For the steel tube, local buckling was the common feature in all types of connectors.

4. Conclusions

This paper presented the results of a numerical study on the performance of different connectors for load transfer in square CFST columns. The main conclusions are given below.

The effectiveness of the connectors in improving bond strength was demonstrated for each specimen. Specimen C4 provided a transferral rate of 92%, specimens C2 and C3 provided a transferral rate of 79% of the total load, and specimen C1 recorded a very high participation value of more than 75% at the first slip, followed by a rapid decrease at P_u .

The presence of vertical ribs in specimen C4 limited the deformation of the circular rib mainly in the central parts. Sliding was very limited between the steel tube and the concrete core, and the bond strength was significantly increased.

The circular rib in specimen C4 was more effective at increasing the peak bond strength, with an increase of 13.16%, compared to the vertical rib.

The circular ribs in specimens C2 and C3 were 1.2 times more effective at increasing the peak bond strength compared to the circular rib of specimen C4.

The corners of the circular rib were very effective at transferring the load and increasing the peak bond compared to the central part, which lost its ability to transfer load at peak bond strength.

The lower side of the upper vertical rib and the two sides of the lower vertical rib transferred higher load values compared to other sections.

The circular rib reached its maximum strength from the four inside corners after the peak bond strength, and the central part underwent buckling in the elastic phase before reaching the yielding strength limit. Meanwhile, the studs reached their peak strength at the welded section at peak load P_u .

The degradation due to compression in the concrete was largely influenced by the type and geometry of the connectors. The surface of the steel tube near the connector exhibited local buckling with complex shapes.

The proposed model showed a high ability to simulate the important parameters that affect the performance of high-strength concrete-filled steel square columns with different shear connectors. This is important in assisting manufacturers and designers in making decisions on the types of connectors with the most effective performance.

The 5% difference observed between the peak axial loads in the experimental and numerical tests due to modelling (mesh size, friction coefficient, material, and boundary conditions modelling) did not affect the superposition of axial load capacity slip curves between experimental and numerical tests.

5. Limitation of the Study

The findings from this research may be further refined by choosing a finer mesh where needed—for example, in areas with the highest stress concentration. Using accurate material properties can also lead to more representative and accurate results: properties such as the elastic modulus of the steel tube and connectors, as well as the correct characteristics of the concrete, can all affect the results, and so there is scope for further improvements. Accurate modelling of the concrete–steel interface is also an important factor that can affect the results obtained. Finally, the boundary conditions, regarding how the different elements are joined together and how the structure is supported, also have a bearing on the findings.

6. Recommendations for Future Work

The results obtained from the present numerical study can be extended to study the load transfer and bond strength for other types of connectors. Additionally, the model can be further refined and developed to predict local and global buckling, and their interaction, for such components. This should include cases where the load is excentric, as is normally the case in columns. It is also important to study the influence of concrete strength and recycled aggregate concrete on the performance of connectors.

Author Contributions: Conceptualisation, M.B.E., A.N.T.I. and M.S.; methodology, M.B.E.; software, M.B.E. and A.N.T.I.; validation, M.B.E., A.N.T.I. and M.S.; formal analysis, M.B.E., A.N.T.I. and M.S.; investigation, A.N.T.I. and M.S.; resources, M.B.E. and A.N.T.I.; data curation, M.B.E. and A.N.T.I.; writing—original draft preparation, M.S.; writing—review and editing, M.S. and A.N.T.I.; visualisation, A.N.T.I. and M.S.; supervision, A.N.T.I.; project administration, M.B.E., A.N.T.I. and M.S. All authors have read and agreed to the published version of the manuscript.

Funding: This research received no external funding.

Data Availability Statement: The original contributions presented in this study are included in the article. Further inquiries can be directed to the corresponding author.

Conflicts of Interest: The authors declare no conflicts of interest.

References

1. Falah, N.; Starossek, U. The interaction of steel tube and concrete core in concrete-filled steel tube columns. *Tubul. Struct.* **2010**, *XII*, 75–84.
2. Han, L.H.; Li, W.; Bjorhovde, R. Developments and advanced applications of concrete-filled steel tubular. *J. Constr. Steel Res.* **2014**, *100*, 211–228. [[CrossRef](#)]
3. Falah, N.; Starossek, U. Numerical Analyses of the Force Transfer in Concrete-Filled Steel Tube. *Struct. Eng. Mech.* **2010**, *35*, 241–256. [[CrossRef](#)]
4. De Nardin, S.; El Debs, A.L.H.C. Shear transfer mechanism in connections involving concrete filled steel columns under shear forces. *Steel Compos. Struct. J.* **2018**, *28*, 449–460. [[CrossRef](#)]
5. Tao, Z.; Song, T.Y.; Uy, B.; Han, L.H. Bond behavior in concrete-filled steel tubes. *J. Constr. Steel Res.* **2016**, *120*, 81–93. [[CrossRef](#)]
6. Qu, X.; Chen, Z.; Nethercot, D.A.; Theofanous, M. Push-out tests and bond strength of rectangular CFST columns. *Steel Compos. Struct. J.* **2015**, *19*, 21–41. [[CrossRef](#)]
7. Aly, T.; Elchalakani, M.; Thayalan, P.; Patnaikuni, I. Incremental collapse threshold for pushout resistance of circular concrete filled steel tubular columns. *J. Constr. Steel Res.* **2010**, *66*, 11–18. [[CrossRef](#)]
8. Wang, F.; Xie, W.; Li, B.; Han, L.H. Experimental study and design of bond behavior in concrete-filled steel tubes (CFST). *Eng. Struct.* **2022**, *268*, 114750. [[CrossRef](#)]
9. Chen, Y.; Feng, F.; Shao, Y.; Zhang, X. Bond-slip behaviour of concrete-filled stainless steel circular hollow section tubes. *J. Constr. Steel Res.* **2017**, *130*, 248–263. [[CrossRef](#)]
10. Eghbali, N.B.; Andamnejad, P. Structural performance of rigid shear connectors in concrete encased steel composite columns. *J. Struct.* **2023**, *54*, 348–368. [[CrossRef](#)]

11. Debnath, P.P.; Chan, T.M. Tensile behaviour of headed anchored hollo-bolts in concrete filled hollow steel tube connections. *Eng. Struct.* **2021**, *234*, 111982. [\[CrossRef\]](#)
12. Dong, H.; Chen, X.; Cao, W.; Zhao, Y. Bond-slip behavior of large high-strength concrete-filled circular steel tubes with different constructions. *J. Constr. Steel Res.* **2020**, *167*, 105951. [\[CrossRef\]](#)
13. Alatshan, F.; Osman, S.A.; Hamid, R.; Mashiri, F. Stiffened concrete-filled steel tubes: A systematic review. *Thin-Walled Struct.* **2020**, *148*, 106590. [\[CrossRef\]](#)
14. De Nardin, S.; El Debs, A.L. Shear transfer mechanisms in composite columns: An experimental study. *Steel Compos. Struct.* **2007**, *7*, 377–390. [\[CrossRef\]](#)
15. Dai, P.; Yang, L.; Wang, J.; Lin, M.; Fan, J. Bond stress-slip relationship in concrete-filled square stainless steel tubes. *Constr. Build. Mater.* **2022**, *326*, 127001. [\[CrossRef\]](#)
16. Ghannam, M.; Metwally, I.M. Numerical investigation for the behaviour of stiffened circular concrete. *J. Struct.* **2020**, *25*, 901–919. [\[CrossRef\]](#)
17. Soltanalipour, M.; Ferrer, M.; Marimon, F.; Albareda, A.; Casafont, M.; Iglesias, G. Experimental study on a highly efficient shear transfer system for square CFST. *J. Constr. Steel Res.* **2023**, *205*, 107905. [\[CrossRef\]](#)
18. Petrus, C.; Hamid, H.; Ibrahim, A.; Nyuin, J.D. Bond strength in concrete filled built-up steel tube columns with tab stiffeners. *Can. J. Civ. Eng.* **2011**, *66*, 915–922. [\[CrossRef\]](#)
19. Hasan, H.G.; Ekmekyapar, T. Bond-slip behaviour of concrete filled double skin steel tubular (CFDST) columns. *Mar. Struct.* **2021**, *79*, 103061. [\[CrossRef\]](#)
20. Qiao, Q.; Zhang, W.; Qian, Z.; Cao, W.; Liu, W. Experimental Study on Mechanical Behavior of Shear Connectors of Square Concrete Filled Steel Tube. *Appl. Sci. J.* **2017**, *7*, 818. [\[CrossRef\]](#)
21. Dong, H.; Chen, X.; Cao, W.; Zhao, Y. Bond behavior of high-strength recycled aggregate concrete-filled large square steel tubes with different connectors. *Eng. Struct.* **2020**, *211*, 110392. [\[CrossRef\]](#)
22. Xiao, Q.G.; Teng, J.G.; Yu, T. Behavior and Modeling of Confined High-Strength Concrete. *J. Compos. Constr.* **2010**, *14*, 249–259. [\[CrossRef\]](#)
23. dos Santos, L.R.; De Sousa Cardoso, H.; Caldas, R.B.; Figueiredo Grilo, L. Finite element model for bolted shear connectors in concrete-filled steel tubular columns. *Eng. Struct.* **2020**, *203*, 109863. [\[CrossRef\]](#)
24. Alemayehu, R.W.; Bae, J.; Ju, Y.K.; Park, J.P. Bond Behavior of Concrete-Filled Steel Tube Mega Columns with Different Connectors. *J. Mater.* **2022**, *15*, 2791. [\[CrossRef\]](#) [\[PubMed\]](#)
25. Chaves, M.D.F.F.; Xavier, E.M.; Sarmanho, A.M.C.; Ribeiro Neto, J.G. Study of bolts used as shear connectors in concrete-filled steel tubes. *Eng. Struct.* **2021**, *231*, 111697. [\[CrossRef\]](#)
26. Zheng, S.; Liang, Z.; Du, Y.; Song, Z. Advancements in interfacial bonding performance of concrete-filled steel tube members. *J. Constr. Steel Res.* **2024**, *213*, 108346. [\[CrossRef\]](#)
27. *Analysis User's Guide Volume V: Prescribed Conditions, Constraints & Interactions*; ABAQUS 6.14; The 3DEXPERIENCE Company: Waltham, MA, USA, 2014.
28. Lam, D.; Dai, X.; Han, L.H.; Ren, Q.; Li, W. Behaviour of inclined tapered and STS square CFST stub columns subjected to axial load. *Thin-Walled Struct.* **2012**, *54*, 94–105. [\[CrossRef\]](#)
29. Gupta, P.K.; Singh, H. Numerical study of confinement in short concrete filled steel tube columns. *Lat. Am. J. Solids Struct.* **2014**, *11*, 1445–1462. [\[CrossRef\]](#)
30. Lin, S.; Zhao, Y.G. Numerical study of the behaviors of axially loaded large-diameter CFT stub columns. *J. Constr. Steel Res.* **2019**, *160*, 54–66. [\[CrossRef\]](#)
31. Khizer, H.R.; Narayana, B.R.; Kumar, N.S. Numerical modeling of concrete composite steel tubes. *Int. J. Res. Eng. Technol.* **2014**, *3*, 187–194. [\[CrossRef\]](#)
32. Thai, H.T.; Uy, B.; Khan, M.; Tao, Z.; Mashiri, F. Numerical modelling of concrete-filled steel box columns incorporating high strength materials. *J. Constr. Steel Res.* **2014**, *102*, 256–265. [\[CrossRef\]](#)
33. Tao, Z.; Wang, Z.B.; Yu, Q. Finite element modelling of concrete-filled steel stub columns under axial compression. *J. Constr. Steel Res.* **2013**, *89*, 121–131. [\[CrossRef\]](#)
34. Lubliner, J.; Oliver, J.; Oller, S.; Onate, E. A plastic-damage model for concrete. *Int. J. Solids Struct.* **1989**, *25*, 299–326. [\[CrossRef\]](#)
35. Taşdemir, M.A.; Tasdemir, C.; Akyuz, S.; Jefferson, A.D.; Lydon, F.D.; Barr, B.I.G. Evaluation of strains at peak stresses in concrete: A three-phase composite model approach. *Cem. Concr. Res.* **1989**, *20*, 301–318. [\[CrossRef\]](#)
36. Popovics, S. A numerical approach to the complete stress-strain curve of concrete. *Cem. Concr. Res.* **1973**, *3*, 583–599. [\[CrossRef\]](#)
37. Mander, J.B.; Priestley, M.J.; Park, A.R. Theoretical Stress-Strain Model for Confined Concrete. *J. Build. Eng.* **1988**, *114*, 1804. [\[CrossRef\]](#)
38. American Concrete Institute. *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary*; American Concrete Institute: Farmington Hills, MI, USA, 2011.
39. Binici, B. An analytical model for stress-strain behavior of confined concrete. *Eng. Struct.* **2005**, *27*, 1040–1051. [\[CrossRef\]](#)

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