Performance of fly ash and silica fume self-compacting concrete filled steel tube stub columns under axial compression

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Abstract. Experimental research on the performance of fly ash and silica fume self-compacting concrete filled steel tube (FSS CFST) stub columns under axial compression were conducted. The main parameters varied in the tests are steel tube’s diameter to thickness ratio, D/t (from 20 to 33), and yield strength (from 275.83 MPa to 350.51 MPa). The self-compacting concrete grade used as infill is M60 concrete with 50% and 10% addition of fly ash and silica fume, as partial replacement of Portland cement. The performance of FSS CFST stub columns is examined via the axial compression capacity, load-shortening response, and mode of failure of 12 specimens. The results indicate that the ultimate axial load capacity of the columns increases as the D/t ratio decreases. The FSS CFST columns have a maximum concrete contribution ratio of 3.72, a strength index greater than unity, and a ductility index of 2.431 and below. The predicted values obtained using Eurocode 4 closely estimate the test results with a mean of 0.927 and a standard deviation of 0.073. Conversely, the predicted values using ACI code are more conservative with a mean of 1.341 and a standard deviation of 0.089.

1. Introduction
Concrete filled steel tube (CFST) sections made from manufactured steel sections and concrete infills have been extensively used as vertical load-bearing components. CFST provides essential benefits such as enhanced strength, fire resistance, ductility, significant cost savings, and rapid construction [1–3]. To improve the compactness of the concrete infill in the steel tubes, self-compacting concrete (SCC) is used. The SCC will increase the working efficiency, compactness, and suitability for pumping and pouring the concrete core [4]. On the other hand, due to rapid industrial development, the amount of fly ash produced yearly has increased; the output is estimated to arrive at 400 million tonnes by 2020 [5]. Fly ash (FA) and silica fume (SF) have once been considered industrial by-products, contributing to contamination and secondary emissions, such as land use. Notwithstanding, Golewski [6] reveals that for sustainable and economic developments, FA and SF’s application as cementing materials rather than cement alone will lead to numerous advantages. Several researchers have established better awareness of the behavior of concrete filled steel tube (CFST) columns. Nevertheless, there is little knowledge about the impact of adding FA and SF on the behavior of CFST columns. Notably, the strength of ordinary Portland cement concrete is virtually affected by the cement replacement with FA and SF [7–9]. It is reported that the use of FA and SF in Portland cement concrete will boost early-age concrete compressive strength and produce concrete with much denser...
microstructure. The enhancement in the strength of FA - SF concrete is due to the filler influence of SF and its pozzolanic reaction. Moreover, the use of FA and SF permits the usage of SF, which is much more refined, with FA, each providing its benefit and as a blend allowing more mixture to be used while sustaining suitable fresh concrete properties [10]. In like manner, the application of these by-products would offer ecological benefit and help reduce Portland cement consumption while improving the concrete properties [11-12].

Herein are previous research that focused on the structural behavior of FA and SF CFST columns. Jegadesh et al. [1] conducted an experimental study on CFST columns' behavior under axial loading. A 25wt% of FA was added. The concrete is reinforced with polypropylene fibers at a varying amount of 0.5, 1.0, and 1.5wt%. The research results showed that the concrete with a 1.0wt% addition of fiber and 25wt% of FA gives the peak axial capacity of CFST columns. Moreover, the incorporation of FA and fibers in the concrete core improves the axial capacity of the CFST columns. Furthermore, Vinod Kumar and Rajamane [13] investigates the axial load behavior of CFST columns using cold-formed steel and geopolymer concrete. FA and blast furnace slag are used as supplementary cementing materials in the concrete. Results indicated that the geopolymer CFST column has high axial load resistance compared to control specimen. Likewise, Li et al. [14] explore the boundary between the steel tube and concrete core. They observed that the boundary between the concrete core and the steel tubes is rich in giant crystals and porous. This is because the steel tube has a strong affinity for water, which produces a massive flux of water to the steel tube surface and increases the water-cement ratio. The FA concrete as a concrete core raises the C-S-H content at the surface. It decreases the calcium hydroxide content, thus increasing the layer density through pozzolanic reactions with calcium hydroxide. Therefore, the bond strength increased since the actual surface area of C-S-H was greater than that of calcium hydroxide. The effect of SF on the steel-concrete bond has also been documented [15-16]. The pozzolanic reaction between the crystals of calcium hydroxide and SF contributes to the linked capillary porosity pathway in the interfacial zone, thereby strengthening the transition zone between concrete and steel. Moreover, the use of FA and SF has been documented by Sadrmontazi et al. [17] to improve the transition zone structure by micro-fillers or their deterrence of calcium hydrate crystal production. Apart from that, FA reduces concrete porosity because of the micro filler effect and prohibits the development of calcium hydrate crystals, thus increasing its compressive strength.

Given the preceding, FA concrete-filled steel tubes have strong potential for load-bearing, but little work is currently underway on FA and SF self-compacting (FSS) CFST columns. Therefore, it is essential to study the axial compression behavior and failure mechanism of steel tubular stub columns filled with FSS concrete. The presented research aims twofold: first, to conduct an experimental testing program to study and monitor the performance of FSS CFST columns. The performance of FSS CFST columns is examined via the axial compression capacity, the load-shortening response, and the mode of failure of stub columns. Three yield strengths and D/t ratios of steel tubes were considered. Secondly, the axial load of FSS CFST columns is compared with the one obtained using international design codes to validate the accuracy of the available design codes. The grade of SCC used is M60 concrete with 50% and 10% addition of FA and SF, respectively, as partial replacement of Portland cement.

2. Experimental program

2.1. Materials
The concrete mixes were made using CEM 1 ordinary Portland cement with a specific gravity of 3.15 and a strength of 42.5 MPa. The low calcium FA per ASTM C618-12a [18] Class F with a specific gravity of 2.1, collected at Johor Bahru from the Tanjung Bin power plant, was used to achieve the required workability and uniformity of SCC while cutting down on the likelihood of segregation and hydration heat. To account for the comparatively weak early FA concrete strength, densified SF with a bulk density of 550–650 kg/m³ and a specific gravity of 2.1–2.4 was used. As suggested by Muller et al. [19], 10% of SF was added. Class F FA comprises a composition of 70% silicon dioxide (SiO₂), aluminium oxide (Al₂O₃), and iron oxide (Fe₂O₃), while the total composition of calcium oxide (CaO) is less than 7%, according to ASTM C618-12a [18]. Natural river sand with a relative density (SSD)
of 2.65 kg/m$^3$, fineness modulus of 3.17, and water absorption of 1.15% was used as the mixture's fine aggregates. A well-graded 10 mm aggregate was used as a coarse aggregate, with a water absorption value and an SSD of 1.0% and 2.66 kg/m$^3$, respectively. The third-generation Superplasticizer based on polycarboxylate ether, termed Sika Viscocrete-2044, which meets ASTM C494 [18] was used to achieve the necessary consistency.

2.2. Specimens

A total of 12 circular specimens were formed and tested under compressive axial loads. The main parameters varied in the tests are (1) diameter to thickness ratio of tubes (from 20 to 33) and (2) yield strength of steel (from 275.83 to 350.51 MPa). The steel tubes were all manufactured from cold-formed steel; each tube was formed from a plate and then bent and welded into a tube. The tube ends were cut to the required length and machined. Steel coupon samples were tested in tension. A self-compacting concrete mix was designed for compressive cube strength ($f_{cu}$) at 28 days of 60 MPa. The modulus of elasticity of the concrete was estimated, the average value being 56,130 MPa. After mixing, the concrete was poured into the steel tubes. The casting of the hollow steel column was done in a vertically upright position. Six samples of circular hollow steel tubes were cast with the FSS, while the remaining six samples were filled with reference concrete. Since all the concrete studied herein were self-compacting, no additional compaction was needed as the concrete was self-flowing. The mix proportions of SCC are presented in Table 1, and the summary of the specimen details is given in Table 2.

### Table 1. Mix proportion of self-compacting concrete.

| Concrete mixes                  | Proportion in kg/m$^3$ | Super plasticizer |
|---------------------------------|------------------------|-------------------|
|                                  | W/B = 0.31             |                   |
| 100%OPC                         | 188 587.50 0 0 796.33 | 837.00 15.69      |
| 40%PC;50%FA;10%SF              | 188 235.00 293.75 58.75 651.37 837.00 | 15.28 |

The symbols used for naming the CFST specimens in Table 2 have the following meanings: letter ‘R’ and ‘P’ represent fly ash-silica fume concrete infill and Portland cement concrete infill. All the circular steel tubes are 350 mm in length. After casting the hollow steel tubes with SCC, the specimens could settle for 24 hours, as shown in Figure 1, and then cured with wet gunny sacks for 28 days.

![Figure 1. Steel tubes set-up after casting.](image-url)
The speed was then decreased to 0.005 mm/sec and kept constant until the load point equal to 90% of the peak load, which was already on the descending branch of the load vs. strain curve, was reached. In the final part of the test, the rate was then raised to 0.01 mm/sec, then to 0.02 mm/sec. It took about 20-30 minutes for the whole process. The ultimate loads ($N_{\text{EXP}}$) obtained from the results of the test are presented in Table 2. At every increment, the displacement estimates and the strain readings were monitored by a computer-controlled data acquisition framework.

### Table 2. Specimen details and member capacities of CFST stub columns.

| Sample Type | Depth-D (mm) | Thickness-t (mm) | D/t ratio | Length-L (mm) | $L/D$ | Steel yield strength-$f_y$ (MPa) | Cylinder Strength of Concrete-$f_c$ (MPa) | $N_{\text{EXP}}$ (kN) | CCR | $SI$  | $DI$  |
|-------------|--------------|-----------------|-----------|--------------|------|-------------------------------|--------------------------------|----------------|-----|------|------|
| C5AR        | 100          | 5.0             | 20.00     | 350          | 3.50 | 350.51                        | 62.23                         | 1286.30        | 2.46 | 1.40 | 1.847 |
| C5BR        | 100          | 5.0             | 20.00     | 350          | 3.50 | 350.51                        | 62.23                         | 1243.70        | 2.38 | 1.35 | 2.257 |
| C4AR        | 100          | 4.0             | 25.00     | 350          | 3.50 | 320.36                        | 62.23                         | 1109.20        | 2.87 | 1.39 | 1.897 |
| C4BR        | 100          | 4.0             | 25.00     | 350          | 3.50 | 320.36                        | 62.23                         | 1133.70        | 2.93 | 1.42 | 1.818 |
| C3AR        | 100          | 3.0             | 33.33     | 350          | 3.50 | 275.83                        | 62.23                         | 937.20         | 3.72 | 1.37 | 2.431 |
| C3BR        | 100          | 3.0             | 33.33     | 350          | 3.50 | 275.83                        | 62.23                         | 898.20         | 3.56 | 1.31 | 1.928 |
| C5AP        | 101          | 5.0             | 20.20     | 350          | 3.47 | 350.51                        | 62.64                         | 1176.70        | 2.23 | 1.26 | 2.316 |
| C5BP        | 100          | 5.0             | 20.00     | 350          | 3.50 | 350.51                        | 62.64                         | 1165.70        | 2.23 | 1.26 | 2.146 |
| C4AP        | 101          | 4.0             | 25.25     | 350          | 3.47 | 320.36                        | 62.64                         | 1022.20        | 2.62 | 1.25 | 2.600 |
| C4BP        | 100          | 4.0             | 25.00     | 350          | 3.50 | 320.36                        | 62.64                         | 975.20         | 2.52 | 1.21 | 3.336 |
| C3AP        | 100          | 3.0             | 33.33     | 350          | 3.50 | 275.83                        | 62.64                         | 826.70         | 3.28 | 1.20 | 2.638 |
| C3BP        | 100          | 3.0             | 33.33     | 350          | 3.50 | 275.83                        | 62.64                         | 859.70         | 3.41 | 1.25 | 3.325 |

2.3. Test set-up and instrumentation

To track the axial and hoop strains on CFST column, two strain gauges at right angles to each other in the transverse and longitudinal directions were glued at the mid-height of the column. The steel tube's outer surface was scrub with sandpaper to remove any paint and then cleaned with Acetone chemical to remove rust and dust before gluing the strain gauges. Figure 2(a) and (b) illustrate the sample preparation and the strain gauges' location with their orientation. A pair of linear variable displacement transducers (LVDTs) was evenly spaced on each side of the circular column in diametrically opposite positions, as shown in Figure 2(c). The LVDTs were used to monitor the overall axial shortening at symmetric locations. For each specimen, axial deformations were obtained from the mean of the LVDTs. The data logger records and stores all data from the LVDT, load cell, and strain gauges. The axial compressive test of the CFST column specimens was performed using a 2000 kN capacity TINIUS OLSEN Universal Testing Machine. The load actuator is servo-controlled for consistent load increments. The axial compressive load was applied only to the core concrete through the bearing plates. The CFST specimens were loaded axially at an initial rate of 0.01 mm/sec until the load applied exceeded 70% of the expected load resistance. The speeds were then decreased to 0.005 mm/sec and kept constant until the load point equal to 90% of the peak load, which was already on the descending branch of the load vs. strain curve, was reached. In the final part of the test, the rate was then raised to 0.01 mm/sec, then to 0.02 mm/sec. It took about 20-30 minutes for the whole process. The ultimate loads ($N_{\text{EXP}}$) obtained from the results of the test are presented in Table 2. At every increment, the displacement estimates and the strain readings were monitored by a computer-controlled data acquisition framework.

![Figure 2](image)

**Figure 2.** Sample preparation (a) location of strain gauges scrub clean before gluing (b) Schematic view of the orientation of strain gauge and (c) LVDT arrangement for CFST circular stub column.
3. Results and discussions

3.1. Compressive strength of CFSTs
An improvement in the ultimate strength of FSS CFSTs is observed compared with Portland cement self-compacting (PCS) CFSTs, as illustrated in Table 2. The ultimate axial load capacities of FSS CFSTs are range between 898.20 kN to 1286.30 kN, while those of PCS CFSTs are range between 826.70 kN to 1176.70 kN. FSS CFSTs has higher compressive strength than PCS CFSTs. The incorporation of FA and SF in concrete effectively reduces the porosity of concrete. It improves the transition zone structure due to the materials’ micro filler effect or their inhibition of calcium hydrate crystal growth [19], therefore, enhancing the ultimate axial capacity of FSS CFSTs.

3.2. Concrete Contribution Ratio
The concrete contribution ratio (CCR) was used to analyse the concrete core's contribution in the CFST stub columns. CCR is defined as follows:

\[
CCR = \frac{N_{\text{exp}}}{A_{\text{eff}} f_y} 
\]

where \( N_{\text{exp}} \) is the test load resistance, \( f_y \) is the steel tube's yield strength, and \( A_{\text{eff}} \) is the steel tube’s effective area corresponding to CEN EN1993-1-5 provisions [20]. For each of the CFST columns, the CCR is measured, and the values are listed in Table 2. The CCR of the CFSTs increases with a decrease in the thickness of steel tubes. The CFST columns with thin-walled have the highest CCR values since the concrete core prevents the steel tube from local buckling, which is likely to occur in these cases. Similarly, the CCR decreases for thicker steel tubes because of the increase in the steel tube's cross-sectional area. Ibáñez et al. [21] reported similar observations. Specimen C3AR and C3BR have the highest CCR value of 3.72 and 3.56, respectively. There is an inherent improvement in the ultimate capacity of CFST columns due to FA and SF's addition.

3.3. Strength index (SI)
The strength index (SI) was used to compare the circular CFST column’s confinement effect under axial load. SI is a suitable measure for composite action and confinement assessments in CFST columns. SI is the ratio between experimental load resistance and the theoretical cross-sectional capacity. A strength index defined by Ibáñez et al. [21] was used herein to quantify the section strength as follows:

\[
SI = \frac{N_{\text{exp}}}{A_s f_y + A_f f_c} 
\]

where \( N_{\text{exp}} \) is the ultimate experimental load, \( A_s \) and \( A_f \) are the cross-sectional area of the steel tube and concrete respectively, \( f_y \) is the steel tube's yield strength and \( f_c \) the concrete strength. The strength index was calculated for each column, and the values were summarized in Table 2. Generally, the strength index improves with the increase in the sectional dimension. The SI value greater than unity indicates that the cross-section can reach the yield load. The improvement in load-carrying capacity for stub columns is believed to result from the strain hardening of steel tube. The scale impact of concrete and steel from the Di ratio between these specimens is insignificant to the columns’ strength. It can be observed that all the CFST columns indicate SI greater than unity. This is due to the influence of core concrete confinement, which results in higher sectional capacities than the total of all components. Compared with PCS CFST columns, all CFST columns infill with FSS concrete has a greater SI index. This can be due to the beneficial effect of concrete volume change rising from free lime hydration in FSS concrete. As the volume extension of concrete was constrained by the steel tube, the microstructure was modified and improved the steel tube's confinement.
3.4. Ductility index
Ductility frequently refers to a structural member or section's ability to withstand plastic deformation beyond the elastic limit while maintaining a sufficient capacity for bearing the load before complete failure. The ductility index is used to assess the ductility of axially loaded CFST columns. A ductility index (DI) defined by Zhong Tao [22] is used.

\[
DI = \frac{\varepsilon_{85\%}}{\varepsilon_y} \tag{3}
\]

where \(\varepsilon_{85\%}\) is the actual axial strain when the load falls to 85% of the peak load, \(\varepsilon_y\) is equal to \(\varepsilon_{75\%}/0.75\), and \(\varepsilon_{75\%}\) is the nominal axial strain when the load reaches 75% of the peak load in the pre-peak stage. The values of \(\varepsilon_{85\%}\) and \(\varepsilon_y\) are obtained from axial load-axial strain curves. The ductility indexes (DI) so determined are given in Table 2. In general, the CFST columns with reference concrete demonstrate better ductility. This is attributed to the brittleness of FSS concrete. The CFST with FSS concrete exhibited a ductility index of 2.431 and below, while reference CFST columns showed member ductility up to 3.336.

3.5. Failure modes
During testing, the stub columns failed by concrete crushing and steel tube yielding. The concrete core prevented the occurrence of the inward buckling. Bulges at the mid of the columns are simple proofs of local outward buckling. The steel walls were pushed out by the concrete core in this crushing failure mode. The typical specimen failure mode is displayed in Figure 3.

![Figure 3. Typical failure modes of control and fly ash-silica fume CFST columns.](image)

Crippling form of failures appeared in the FSS CFST specimen C3AR and C3BR with steel tube thickness of 3 mm at higher shortening. Likewise, in circular column C4BR with steel tube thickness of 4.0 mm.

3.6. Axial load-deformation curves
Axial load versus deformation curves describe the ductility, stiffness, and toughness behavior of CFST columns. The axial load-deformation curves can be used to apprehend the behavior of the concrete core. Figure 4 presents the axial load-deformation curves of FSS CFSTs and control. There are some standard features about the column failure behavior in this category, dissimilarities in steel properties and concrete cause diverse reactions in the load-shortening curves.
Specimen C4AP has a very smooth load-shortening curve, as presented in Figure 4(a), which indicates the high potential for deformation of 62.64 MPa control, with maximum axial deformation of 13.65 mm. The SI of C4AP is 1.25, which means that this circular column has a strong confinement effect. Similarly, column C4BP has a smooth curve, as illustrated in Figure 4(b). This specimen has a sound level of deformation capacity with a maximum axial shortening of 9.74 mm and SI of 1.21. Specimen C3AP and C5BP have a load-shortening curve with distinct features. The transition from peak zone to post-peak zone is rapid for these specimens. This behavior can be attributed to the high strength concrete's brittle nature, as Ibañez et al. presented comparative perceptions [23]. These specimens, C3AP and C5BP, exhibited a maximum axial deformation of 4.13 and 3.72 mm, respectively.

![Figure 4. Axial load-deformation curves of CFST columns (a) Series A and (b) Series B.](image)

The transition from the linear region to the collapse region of the load-shortening curve of specimens C3BP and C5AP is very rapid. The ultimate load capacity of sample C5AP was recorded to be 1176.70 kN, which corresponds to the peak axial load for the category of the control. However, the specimen demonstrates a brittle failure with a maximum axial deformation of 0.61 mm. Conversely, specimen C3BP recorded a maximum axial deformation of 3.29 mm, and its curve has an irregular behavior in the zone of collapse. The failure behavior follows a similar pattern for all the circular CFSTs with an inelastic softening. The ascending branch of the axial load-deformation curve for each series seems consistent. Almost all the specimens fail in a brittle pattern, and the descending branch of the axial load-deformation curves fall sharply.

3.7. Analysis of axial load-strain relation
The axial load versus axial and hoop strains based on the strain gauge readings at mid-height of the CFST stub columns are shown in Figure 5(a) to (c). The hoop strain is presented as positive in the figures, while the axial strain is negative. The strain in circular specimen C5AP infill with reference concrete develops rapidly during loading. On the contrary, strain in column C5AR infill with FSS concrete did not show much increase at the initial stages of loading, as presented in Figure 5(a). The axial strain in C5AP is 0.0199, which is greater than 0.00389 in C5AR. At the same time, the hoop strain in C5AP is 0.0162, which is less than 0.0173 in column C5AR. Column C5AR has greater axial load-bearing capacity.

Similarly, the early strain development of specimen C4AP during loading is greater than that of specimen C4AR, as illustrated in Figure 5(b). Both axial and hoop strains of 0.0617 and 0.0786 in column C4AP, respectively, are more significant compared to 0.0022 and 0.0139 in column C4AR. Moreover, column C4AR sustains a more significant axial load. Likewise, column C3AP exhibits larger early strain increment during loading than column C3AR, as shown in Figure 5(c). However, both axial and hoop strains of 0.0162 and 0.0339, respectively, in column C3AR are more extensive
The CFST columns infilled with FSS concrete withstand greater axial load. Notably, the early strain development in the CFST columns with thinner steel tube thickness is much higher. According to Kwan et al. [24], the hoop strain is interconnected to confining stress. The confinement on the concrete depends on the confined concrete hoop strain. Through the hoop strain component, the steel tube is being utilized in view of providing confinement to the infill concrete. Moreover, under low confinement, the load–strain curve demonstrates strain softening, while strain hardening is due to the strong confinement.

4. Compressive strength predictions based on existing design specifications

In this paper, Eurocode 4 (EC4) [25] and ACI [26] design standards are used to predict the ultimate axial load capacity of FSS CFST columns. EC 4 assumes the limit state design to provide safety and serviceability by applying some partial safety factors to material properties and load. The Australian Standards AS 5100 [27] and ACI use the same formulae for estimating the axial load resistance of the circular columns [28]. However, The ACI formulae do not account for the interaction and confinement efficiency between the steel tube and concrete core. On the contrary, the American code AISC [29] procedure, which is based on the design of structural steels, permits the utilization of either the allowable stress design or the limit state. Moreover, AISC increases the concrete stress to reflect the restraining hoop force. The ultimate axial load capacities of the specimens examined were compared with those predicted from EC 4 and ACI, as presented in Table 3. EC 4 gives the best prediction of the ultimate capacity of the CFST columns. The mean is 0.927, and the standard deviation is 0.073. On the contrary, the ACI forecast are very conservative because their calculation of the ultimate capabilities does not involve concrete confinement. The mean is 1.341, with a standard deviation of 0.089.
### Table 3. Comparison of experimental results with EC4 and ACI prediction.

| Specimen | $N_{\text{EXP}}$ (kN) | $N_{\text{EC4}}$ (kN) | $N_{\text{ACI}}$ (kN) | $N_{\text{EXP}}/N_{\text{EC4}}$ | $N_{\text{EXP}}/N_{\text{ACI}}$ |
|----------|------------------------|------------------------|------------------------|-------------------------------|-------------------------------|
| C5AR     | 1286.30                | 1261.94                | 866.38                 | 1.019                         | 1.485                         |
| C5BR     | 1243.70                | 1261.94                | 866.38                 | 0.986                         | 1.436                         |
| C4AR     | 1109.20                | 1111.08                | 780.00                 | 0.986                         | 1.422                         |
| C4BR     | 1132.70                | 1111.08                | 780.00                 | 1.019                         | 1.452                         |
| C3AR     | 937.20                 | 949.40                 | 691.73                 | 0.987                         | 1.355                         |
| C3BR     | 898.20                 | 949.40                 | 691.73                 | 0.946                         | 1.298                         |
| C5AP     | 1176.70                | 1430.10                | 881.73                 | 0.823                         | 1.335                         |
| C5BP     | 1165.70                | 1293.76                | 868.59                 | 0.901                         | 1.342                         |
| C4AP     | 1022.20                | 1161.16                | 794.51                 | 0.880                         | 1.287                         |
| C4BP     | 975.20                 | 1144.34                | 782.31                 | 0.852                         | 1.247                         |
| C3AP     | 826.70                 | 984.13                 | 694.13                 | 0.840                         | 1.191                         |
| C3BP     | 859.70                 | 984.13                 | 694.13                 | 0.874                         | 1.239                         |

### 5. Conclusions

This paper presents an experimental analysis on CFST stub columns with fly ash and silica fume self-compacting concrete under axial compressive load. Based on experimental results, the following conclusions were drawn.

- An improvement in the ultimate strength of the FSS CFST columns was observed compared with PCS CFSTs. The ultimate axial load capacities of FSS CFSTs are in the range of 898.20 kN to 1286.30 kN, while those of PCS CFSTs lie between 826.70 kN to 1176.70 kN.
- FSS CFST Specimen C3AR and C3BR have the highest CCR value of 3.72 and 3.56, respectively.
- All CFST stub columns indicate SI higher than unity.
- CFST columns with FSS concrete showed a ductility index of 2.431 and below, while reference CFST columns had member ductility up to 3.336.
- Nearly every CFST stub column fails via local buckling failure.
- The Eurocode 4 predicted values closely estimate the test results. The mean is 0.927, and the standard deviation is 0.073. Conversely, the ACI forecast are very conservative, with a mean of 1.341 and a standard deviation of 0.089.

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