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Abstract

This study investigates the behavior of self-compacting concrete (SCC) specimens reinforced with small-diameter steel tubes in lieu of reinforcing bars. Twenty specimens were cast and tested. Four specimens were reinforced with normal steel bars (reference specimens) and the remaining 16 with steel tubes. All specimens contained steel helices with a pitch of either 50 mm or 75 mm. Deformed steel bars of 16-mm diameter were used in the four reference specimens as longitudinal reinforcement. Steel tubes of 33.7-mm outside diameter with 2-mm wall thickness (ST33.7) and steel tubes of 26.9-mm outside diameter with 2.6-mm wall thickness (ST26.9) were used as longitudinal reinforcement in the remaining 16 specimens. The specimens were divided into five groups with four specimens in each group. From each group, one specimen was tested under concentric load, one under 25-mm eccentric load, one under 50-mm eccentric load, and one under flexural load. Although the nominal yield tensile strength of the steel bars was 150 and 250 MPa greater than the nominal yield tensile strength of steel tubes ST33.7 and ST26.9, respectively, the results revealed that steel tube reinforced self-compacting concrete (STR SCC) specimens have ultimate load similar to reference specimens. Ductility of concentrically loaded STR SCC specimens was higher than that of the reference specimens. This study also reveals the challenges associated with using steel tubes as longitudinal reinforcement in STR SCC column specimens due to slip of steel tubes in concrete and explores remedial measures for the slip of steel tubes.

Keywords

columns, reinforced, longitudinally, behavior, steel, self-compacting, tubes, concrete

Disciplines

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Behavior of Self-Compacting Concrete Columns Reinforced Longitudinally with Steel Tubes

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Abstract

This study investigates the behavior of Self-Compacting Concrete (SCC) specimens reinforced with small diameter steel tubes in lieu of reinforcing bars. Twenty specimens were cast and tested. Four specimens were reinforced with normal steel bars (reference specimens) and the remaining 16 with steel tubes. All specimens contained steel helices with a pitch of either 50 mm or 75 mm. Deformed steel bars of 16 mm diameter were used in the four reference specimens as longitudinal reinforcement. Whilst steel tubes of 33.7 mm outside diameter with 2 mm wall thickness (ST33.7) and steel tubes of 26.9 mm outside diameter with 2.6 mm wall thickness (ST26.9) were used as longitudinal reinforcement in the remaining 16 specimens. The specimens were divided into five groups with four specimens in each group. From each group, one specimen was tested under concentric load, one under 25 mm eccentric load, one under 50 mm eccentric load and one under flexural load. Although

the nominal yield tensile strength of steel bar was 150 and 250 MPa greater than the nominal yield tensile strength of steel tubes ST33.7 and ST26.9, respectively, the results revealed that Steel Tubes Reinforced Self-Compacting Concrete (STR SCC) specimens have ultimate load similar to reference specimens. Ductility of concentrically loaded STR SCC specimens was higher than reference specimen. This study also reveals the challenges associated with using steel tubes as longitudinal reinforcement in STR SCC column specimens due to slip of steel tubes in concrete and explores remedial measures for the slip of steel tubes.

Keywords: Composite columns; self-compacting concrete; steel bars; steel tubes; stress-strain behavior; debonding.

Introduction

Performance of concrete column has been widely improved by using composite material systems such as encased sections and concrete-filled tubes (Shanmugam and Lakshmi 2001). Different combinations of encased section and steel section were widely studied. The characteristics of the composite columns are governed by the geometric configuration and strength of materials. Steel sections and concrete have been used to construct composite columns with different cross-sections. The synergies between steel and concrete in composite columns provide better performance in terms of high strength, stiffness, ductility as well as fire and seismic resistance (Sakino et al. 2004; Giakoumelis and Lam 2004 and Choi and Xiao 2010). Concrete-filled steel tube (CFT) columns consist of a steel tube and concrete infill. In some cases internal steel reinforcement is used to provide proper connections between the concrete members (Moon and Lehman 2013). Another advantage of using internal steel reinforcement is to enhance the strength of the concrete columns, because the concrete core of these columns is confined by both steel tube and internal confinement

reinforcement (Xiamuxi and Hasegawa 2012). Steel tube confines the concrete in a triaxial state. Therefore, a few stirrups can be used in the reinforced concrete column confined with thin-walled steel tube (Wang et al. 2015).

Composite columns offer considerable improvements over conventional columns reinforced with steel bars and are used in many structural applications. In traditional reinforced concrete columns, solid steel bars are used as longitudinal reinforcement. In this study, small diameter steel tubes were used as longitudinal reinforcement for concrete columns. Steel tubes that have the same cross sectional area as solid bars have higher second moment of area and radius of gyration than solid bars. Filling these steel tubes with concrete can further increase the yield and ultimate strength as well as the ductility of the concrete columns under axial compression load. This is because the concrete infill will contribute in delaying the local buckling and converts the failure mode of steel tube wall from inward to outward ANS/AISC 360-10 (AISC 2010). For eccentric and flexural loading, using concrete filled steel tubes in lieu of solid bars may increase the stiffness of the cracked concrete section of the specimens, as the second moment of area of the cracked cross section for the steel tubes reinforced specimen is slightly higher than that of steel bars reinforced specimen. This is mostly due to the presence of the confined concrete inside the steel tubes. In addition, a circular tube section provides better confinement of concrete than other tube sections because the tube wall resists the concrete pressure by membrane-type hoop stresses instead of the plate bending (Schneider 1998). Since, the diameter of these steel tubes is quite small to be filled with the normal concrete, high flowability concrete is needed to completely fill the tube without segregation. The Self-Compacting Concrete (SCC) is considered a suitable option.

The SCC is an innovative material that can be used in complex forms and members that contain congestion of reinforcement, without needing vibration as it is able to compact under its own weight (Goodier 2003; De Schutter 2005 and EFNARC 2005). Han and Yao (2004) showed that the SCC is very suitable for CFT columns due to rheological properties of the SCC. Roeder et al. (2010) indicated that the empty tube could be filled with concrete rapidly by using SCC, without requiring any vibration. Muciaccia et al. (2011) explained that the use of SCC in-filled steel tube columns provides ease to construct difficult shapes and it reduces the cost as no vibration is required. It was also found that the use of SCC in lieu of normal concrete did not affect the design requirements of the CFT columns. Use of SCC can help in investigating new types of composite concrete columns in order to gain better stiffness, ductility and strength of concrete columns.

The above review of literature shows that no study has yet been conducted on the behavior of concrete columns reinforced with longitudinal small diameter steel tubes. It is expected that use steel tubes filled with SCC as reinforcement will decrease the overall buckling of longitudinal reinforcement and consequently increase the ductility of the column. Also, steel tubes will effectively confine the infill concrete resulting in an increase of the axial compressive strength. Hence, significant research investigations are needed to explore the behavior of columns reinforced with small diameter steel tubes filled with SCC (STR SCC column). This study provides a basis for using steel tubes instead of the conventional steel reinforcing bars as a new method for reinforcing concrete columns. It also reveals the problems associated with the use of plain steel tubes as longitudinal reinforcement in STR SCC column specimens due to the slip of steel tubes in concrete. Remedial measures for the problem associated with the slip of steel tubes have also been investigated.

Experimental Program

A total of 20 circular SCC specimens reinforced longitudinally with steel bars and tubes were cast and tested at the High Bay Laboratory of the University of Wollongong, Australia. All specimens have the same dimensions: 240 mm in diameter, 800 mm in height and a height to diameter ratio of 3.3. The dimensions of the tested specimens were chosen to be suitable to the condition and capacity of the available testing facilities in the laboratory. It is noted that vertical support member with height-to-diameter (L/D) ratio greater than or equal to 2.5 is considered as a column in Canadian Standards CAN/CSA S6-06 (CSA 2006). Moreover, ACI 318-11 (ACI 2011) defined column as a member mainly used to support the axial compressive load with height-to-least lateral dimension ratio more than 3.

The SCC mix with a nominal compressive strength of 50 MPa was used in casting the specimens. The maximum aggregate size used in the SCC mix was 10 mm. The mix proportion of the SCC used in this study is shown in Table 1. All longitudinal steel tubes (33.7 mm and 26.9 mm outside diameters) were also filled with the SCC during the casting of the specimens. The specimens were divided into five groups with four specimens in each group. From each group, one specimen was tested under concentric load, two specimens were tested under 25 and 50 mm eccentric loads, and the last specimen was tested as a beam under flexural load (four-point load bending). The first group contained specimens reinforced with steel bars and considered as reference specimens and the remaining specimens were reinforced with steel tubes. All specimens were reinforced transversally with plain R10 (10 mm diameter plain steel bars) steel helices with a nominal tensile strength of 250 MPa. The specimens in the first group were reinforced longitudinally with six deformed steel bars of 16 mm diameter and nominal tensile strength of 500 MPa. The specimens in the second and third groups were reinforced longitudinally with six steel tubes of 33.7 mm outside diameter,

2 mm in thickness and nominal tensile strength of 350 MPa (ST33.7). The remaining specimens in the fourth and fifth groups were reinforced longitudinally with six steel tubes of 26.9 mm outside diameter, 2.6 mm in thickness and nominal tensile strength of 250 MPa (ST26.9). Steel helices with 50 mm pitch were used in reinforcing the specimens transversally in the first, second and the fourth groups. The pitch of the steel helix satisfies ACI 318-14 (ACI 2014) requirements. The specimens in the third and fifth groups were reinforced transversally with 75 mm pitch steel helices. The specifications and reinforcement details of tested specimens are shown in Table 2 and Figure 1.

Table 2 shows the group names and specimen labels. The groups are labelled in two parts and the specimens are labelled in three parts. In the first part of the group names and specimen labels, the letters (N and ST) refer to the type of the longitudinal reinforcement in the specimen. The letter N refers to deformed steel bars and the letters ST refer to the steel tubes. The number 16 refers to deformed steel bars of 16 mm diameter with a nominal tensile strength of 500 MPa. The number 33.7 refers to the steel tubes with 33.7 mm outside diameter and 2 mm wall thickness and with a nominal tensile strength of 350 MPa. The number 26.9 refers to the steel tubes with 26.9 mm outside diameter and 2.6 mm wall thickness and with a nominal tensile strength of 250 MPa. In the second part of the group names and specimen labels, the set of a letter and numbers refer to the helix pitch. H50 represents a 50 mm pitch and H75 represents a 75 mm pitch of the transverse reinforcement. In the third part of the specimen label refers to the type of the loading condition. The C refers to the concentric loading, E25 refers to the 25 mm eccentric loading, E50 refers to the 50 mm eccentric loading and F refers to the flexural loading (four-point load bending). For example, ST33.7-H50-E50 refers to the specimen that reinforced longitudinally with six ST33.7 steel tubes and transversally with a pitch of 50 mm helix and tested under 50-mm eccentric load.

Material Properties

Self-compacting concrete was prepared according to EFNARC (2002) method. A number of concrete mixes were investigated to achieve self-compacting concrete. The properties of fresh concrete were tested according to ASTM C1610-14 (ASTM 2014b); ASTM C1611-14 (ASTM 2014c) and ASTM C1621-14 (ASTM 2014d). For the water/powder ratio of 0.4, column segregation test according to ASTM C1610-14 (ASTM 2014b), slump flow test according to ASTM C1611-14 (ASTM 2014c) and J-ring test according to ASTM C1621-14 (ASTM 2014d) were carried out. The test results were found to be satisfactory. All the specimens were cast using ready-mixed self-compacting concrete supplied by a local company. The average compressive strength of the SCC was 57 MPa at 28 days. Two different steel bars were used in this study, deformed steel N16 bars (16-mm diameter deformed bars with 500 MPa nominal tensile strength) for longitudinal reinforcement in the first group (reference specimens) and plain mild steel R10 bars (10-mm diameter plain bars with 250 MPa nominal tensile strength) for transverse reinforcement in all specimens. Three samples of N16 deformed bars and R10 plain bars were tested in accordance with Australian Standard AS 1391-2007 (AS 2007). Results of testing revealed that the N16 steel bar experienced strain hardening behavior with an obvious yield stress. However, the yield stress of the R10 plain bar was not easily identified and was determined using the 0.2% proof stress. The average tensile strengths and elastic modulus of N16 bars were 556 MPa and 200 GPa, respectively. The average tensile strength and elastic modulus of R10 bars were 400 MPa and 195 GPa, respectively.

Two different steel tubes were used to reinforce remaining specimens in the longitudinal direction: steel tubes ST33.7 and ST26.9. Three samples of each ST33.7 and ST26.9 tubes were tested in accordance with ASTM A370-14 (ASTM 2014a). Steel plugs were designed

based on the inner diameters of the steel tubes to suit the 29.7 and 21.7 mm steel tubes in tensile testing. Yield stresses of both steel tubes were not easily identified with a strain plateau and were instead determined using the 0.2% proof stress. The average tensile strength and elastic modulus of ST33.7 were 450 MPa and 196 GPa, respectively. The average tensile strength and elastic modulus of ST26.9 were 355 MPa and 192 GPa, respectively.

The bond stress between different longitudinal reinforcement and SCC was determined using pull-out tests according to RILEM (1983). For each reinforcing type, three specimens were tested on the 28th day. The average bond stress of N16 steel bars was 19 MPa at failure. The specimens with N16 steel bar failed due to pull-out. The average bond stress of ST33.7 and ST26.9 steel tubes filled with the SCC was 1.5 and 0.75 MPa, respectively at failure. The specimens with SCC filled ST33.7 and ST26.9 failed due to slip.

Specimen Preparation and Testing

Formwork

PVC pipes with a 240 mm inner diameter were used as formwork molds for the concrete column specimens. The PVC pipe was cut for a length of 800 mm. Twenty pieces of PVC pipe molds were prepared. These pipe molds were then fixed to a non-absorbing plywood base plate. The formwork was constructed to keep both the top and bottom of the molds in a fixed plumb position. The formwork was secured to a rigid pallet onto which the base plate was mounted as shown in Figure 2.

Longitudinal and Transverse Reinforcement

The internal longitudinal reinforcement bars and steel tubes were prepared and cut at 760 mm in order to provide 20 mm clear cover at the top and bottom of the reinforcement cage. In

addition, the transverse steel helices were prepared by forming helices with a 200 mm outer diameter and pitches of 50 and 75 mm. In constructing the reinforcing cages, the longitudinal reinforcement was tied with equal spacing to the inside of the helices following the test matrix in Table 2 and Figure 1.

To construct the reinforcement cages, two 12 mm thick circular plastic templates were made. The diameters of the holes in the templates were designed to suit various tube and bar sizes as well as to maintain the inner diameter of the helix as 180 mm. Circular template 1 was designed to hold both ST26.9 and ST33.7 steel tubes, whilst circular template 2 was designed to hold the N16 bars. Figure 3 shows the schematics of circular templates. Each circular template contained six half circles holes at the circumference to hold the longitudinal reinforcement. These holes were created at equal distances on the circumference of the circular template. The two identical circular templates were setup parallel to each other at the top and bottom by using a threaded steel rod. Hex steel nuts, washers and lock washers were used to fix these two circular templates at required distance to the threaded steel rod. Figure 4(a & b) shows the setup of circular templates for different longitudinal reinforcement. In addition, two aluminum forms were designed as a spacer for holding the helices at the desired pitch, as shown in Figure 4(c). The aluminum forms have grooves to adjust the spacing of helices at 50 and 75 mm. All longitudinal steel bars or steel tubes were tied together with the helix by using tie wire. Figure 5 shows 20 fabricated reinforcing cages.

Testing of the Specimens

The Denison 5,000 kN compression testing machine was used for testing all the specimens. Of the 20 specimens, 15 were tested as columns in compression tests. The top and bottom of the column specimens were wrapped by two layers of CFRP sheets with 100 mm overlap to

avoid premature failure of the concrete during axial compression tests. Also, the top and bottom end surfaces of the columns were capped with high strength plaster to ensure a uniform distribution of the applied loads. The plaster was poured into the loading head molds at the bottom end first and left to cure for 30 minutes before the column specimen was turned over and similarly the plaster was poured into the top loading head mold in order to cap the top end.

For applying the eccentric loading on the column, steel loading heads manufactured at the University of Wollongong were used. The loading heads were used to transfer the load generated by the machine into the column specimen causing lateral deflection (δ). Steel loading heads consisted of two parts: a circular steel plate adapter and steel plate with ball joint, as shown in Figure 6(a). A steel ball joint was installed on a steel plate to induce eccentricities (e) of 25 mm and 50 mm to the top and bottom loading heads. Concentric loading however was applied directly through the circular steel plates attached at both ends of the column specimen without any steel ball joints. To measure the axial deformation of the column specimens, two linear variable differential transformers (LVDT) were used. The LVDTs were connected directly to the machine loading heads at opposite corners. A laser optical displacement sensor (LODS) was attached at the mid-height of the test specimen to measure lateral displacement under eccentric loading. In addition, before casting, strain gauges were attached on the two opposite longitudinal reinforcements (bars and tubes) to measure axial strain. The LVDTs, LODS and strain gauges were connected to the data logger which recorded measurements and saved on the control computer. Figure 6(b) shows the typical test setup of column specimens.

Five specimens were tested as beams under flexural load (four-point load bending). Flexural loading system consisted of two circular steel loading rigs at the top and bottom of the beam specimens. These rigs were manufactured at the University of Wollongong to suit the Dension testing machine and 240 mm round specimens. The specimens were tested by using a four-point load bending arrangement with the loads applied at $L/3$ distance from the supports, where L is the span length. To measure the mid span deflection (δ) of the beam specimen the same LODS was placed centrally underneath the rig. A slot at the bottom support of the rig allowed the LODS to optically measure the deflection. The LODS was connected to the data logger which recorded the results. Figure 7 shows the schematic details of the four-point load bending test arrangements. All tests were carried out at the displacement controlled loading rate of 0.3 mm/min and the data was recorded at every 2 seconds.

Experimental Results and Analysis

Column Specimens under Concentric Load

Five specimens were tested under concentric axial compression until failure. Table 3 reports the test results. Figure 8 shows the axial load-axial deformation diagrams of the five concentrically tested specimens. For all specimens, initial failure of the concrete cover started with cracks after the maximum load was reached. As the loading continued to increase the buckling of longitudinal reinforcement occurred at the mid-height of the column specimens. Finally, the specimens failed by fracture of the steel helices at the mid-height. The failure modes of the specimens are shown in Figure 9. The average strains in the longitudinal steel bars and steel tubes indicated that steel bars and tubes yielded at the maximum axial load. The axial deformation corresponding to the first helix fracture in Specimen N16H50C was 20 mm, while in Specimen ST33.7H50C and Specimen ST26.9H50C were 33.5 and 36 mm,

respectively. Specimen N16H50C had a lower axial deformation at the first fracture because the N16 steel deformed bar buckled earlier than the steel tubes. Consequently more pressure was applied on the steel helix and caused the yielding and fracture of steel helices. Due to the tensile strengths being different, the force contribution of steel tubes ST33.7 and ST26.9 in Specimen ST33.7H50C and Specimen ST26.9H50C were less than N16 steel bars in the reference Specimen N16H50C by 19.9% and 37%, respectively. Nevertheless, Specimen ST33.7H50C had similar yield and maximum load as the reference Specimen N16H50C. Specimen ST26.9H50C had only 5% less maximum load than the reference specimen. Circular steel tubes provided confinement to the concrete and increased the compressive strength of concrete which resulted in enhancing the capacity of the columns. Besides, the ultimate axial deformation at failure for Specimen ST33.7H50C and Specimen ST26.9H50C were higher than the reference Specimen N16H50C (Table 3).

For the increase of the pitch of helices from 50 to 75 mm in the specimens reinforced with steel tubes ST26.9, the maximum axial load reduced by 6%. Whereas, increasing the pitch of helices from 50 to 75 mm in the specimens reinforced with steel tubes ST33.7, the maximum axial load reduced by only 3.5%. The reason for the differences in the reduction of maximum axial load is associated with the slenderness (L/D) ratio for steel tubes. The slenderness of steel tube ST26.9 is higher than the slenderness of steel tube ST33.7. From the test results, it was observed that specimens reinforced with ST33.7 steel tubes had the yield and maximum load capacity greater than specimens reinforced with ST26.9 steel tubes. This is because the ST33.7 has higher nominal tensile strength than ST26.9. Besides, the larger internal diameter of the ST33.7 tubes allowed for a large concrete to be filled inside the tube which contributed in increasing the capacity of the specimens as the confined concrete inside the tube positively contributed to the compressive strength of the specimen.

To study the behavior of circular column specimens reinforced longitudinally with steel tubes, it is important to investigate the ductility of the specimens, which provides an indication of the post peak axial load-axial deformation behavior. In this paper, the ductility was calculated as the ratio of the area under the curve up to the ultimate deformation to the area under the curve up to the yield deformation (Hadi 2009). In order to specify yield deformation, the secant line was drawn from the origin to the point of 0.75 times of the first peak load. Then, the yield deformation was specified on the ascending load-deformation curve as corresponding to the intersection point of the extension secant line and a horizontal line from the first peak load (Foster and Attard 1997).

For concentric load, the ductility of the column specimens reinforced with steel tubes were higher than the column specimens reinforced with steel bars for the same pitch of helix. The ductility of Specimen ST33.7H50C and Specimen ST26.9H50C were 30% higher than the reference specimen (Table 3). In spite of increasing the pitch of helix from 50 to 75 mm in the Specimen ST26.9H75C, the ductility of Specimen ST26.9H75C was 11% higher than the reference specimen. However, the Specimen ST33.7H75C has ductility lower than the reference Specimen by 18%. This is mainly because Specimen ST33.7H75C has higher yield deformation than the reference specimen by about 13% (Table 3).

Column Specimens under Eccentric Load

Ten specimens were tested under eccentric loading, five of which under 25 mm eccentric load and five under 50 mm eccentric load. Test results of specimens under 25 mm eccentric loading are given in Table 4. Figure 10 shows the axial load-axial deformation and axial load-lateral deformation diagrams of the specimens under 25 mm eccentric loading. For all specimens tested, the initial failure occurred as a result of cracks in the concrete cover once

the specimens reached their maximum load. This was followed by a small drop in the maximum load due to spalling of concrete cover. After that, reference Specimen N16H50E25 showed an increasing load. The load increased to a value less than the first maximum load. On the other hand, specimens reinforced with steel tubes showed a steady reduction in the load-deformation curve after spalling of concrete cover. This is because the debonding between steel tubes and concrete took place during the testing. For specimens reinforced with steel tubes, the failure occurred as results of a buckling of the longitudinal steel tubes in the compression region. While the failure of the reference column specimen occurred due to the fracture in one of the longitudinal bars in the tension region combined with buckling in the longitudinal bars in the compression region, as shown in the Figure 11. After the completion of the testing, by drilling holes in the specimens, the slip in both steel bars and steel tubes was measured using a measurement tape. It was found that the slip in steel tube was 20 mm while no slip was observed in N16 steel bars.

To overcome the slip of steel tubes for column specimens reinforced with steel tubes under eccentric loading, 4 mm thick washers with two different inner diameters 34 and 27 mm were welded to both ends of ST33.7 and ST26.9 steel tubes, respectively. The washers were cut to three quarters of the shape. An initial attempt to prevent debonding was done on Specimen ST33.7H75E50 and Specimen ST26.9H75E50. At first, the locations of steel tubes were identified by mildly drilling the concrete cover. Secondly a ring of the concrete cover was removed carefully by using a chisel and a hammer in order to reveal the ends of the steel tubes. Thirdly, the washers were fitted and properly welded around the ends of the tubes. Finally, high strength plaster (compressive strength of 70 MPa) was used to fill the concrete cover. Figure 12 shows details of the remedial measure for the slip of steel tubes.

Results of testing Specimen ST33.7H75E50 and Specimen ST26.9H75E50 indicated that slip still occurred even after the repair. This is because the concentrated stresses on the washers led to bending of washers and tearing the tubes near the welding area at both ends of the tube in the tension region. To solve this, for the remaining Specimen ST33.7H50E50 and Specimen ST26.9H50E50, two washers were welded at both ends of the longitudinal steel tubes in the tension region of each specimen (Figure 13). This was done in an attempt to distribute the tensile strain over four anchor points and prevent the first washer from bending by supporting it by the second washer. This attempt was also used to increase the resistant capacity of steel tube wall before tearing at the ends and overall slip.

Test results of specimens under 50 mm eccentric loading are reported in Table 5. Figure 14 shows the axial load-axial deformation and axial load-lateral deformation diagrams of the specimens tested under 50 mm eccentric loading. For all specimens tested, the initial failure occurred as a result of cracks within the concrete cover once the specimens reached their maximum load. This was followed by a small drop in the maximum load due to spalling of concrete cover. After that, reference Specimen N16H50E50 showed an increasing load. The load increased to a value less than the first maximum load. Axial load then steadily decreased until the failure occurred due to yielding of the longitudinal bars in the tension region combined with buckling of the longitudinal bars in the compression region. Also, Specimen ST33.7H50E50 experienced a similar yield point to the reference specimen after initial spalling of concrete cover but the tube tearing and failure did not occur at the center. Similarly for Specimen ST26.9H50E50, two washers were welded at both ends of the longitudinal tubes which were supposed to be in the tension region during testing. However, during testing the tubes with washer were placed inadvertently in the compression region rather than in the tension region resulting in a steady reduction in the load-deformation curve

after spalling of concrete cover. As in Specimens ST26.9H75E50 and ST33.7H75E50, the tubes tore around the single washers after reaching the maximum load causing a slip with no signs of ductile behavior. However, placing the tubes with double washers in the compression region resulted in a higher load carrying capacity compared to the Specimen ST33.7H50E50. This indicated that the slip may also occur in tubes in compression loads as well as in tension under 50 mm eccentric loads. After spalling of concrete cover, the tube was compressed and slip in the compression region, as a result, the double washers prevented the slip in this case.

For 25 mm eccentric load, the ductility of the column specimens reinforced with steel tubes were lower than the column specimens reinforced with steel bars due to slip of steel tubes. For 50 mm eccentric load, the ductility of the column specimens reinforced with steel tubes were improved due to the use of washers, but still lower than the column specimens reinforced with steel bars.

Flexural Specimens

The remaining five specimens were tested under four-point load bending as beams. Test results of specimens under flexural loading are reported in Table 6. Figure 15 shows the load-midspan deflection diagrams of the specimens tested under flexural loading. It is noted that span to depth ratio of the test specimen is relatively short and hence the failure may not truly represent pure bending condition. However, for uniformity and consistency, specimen dimensions have been kept the same as other specimens tested under concentric and eccentric axial compression. It was expected that the steel tubes in the tension region for the specimens under flexural would be subjected to higher tensile stresses compared to the specimens under 25 and 50 mm eccentric loads, with high possibility that slip would occur. To prevent slip of steel tubes inside these specimens, single washer and double washers were also used in the

compression and tension regions, respectively. Because of the relatively large cross-section area of the specimens, precautions were taken to avoid any bearing and shear failures. Bearing plate spacers, 50 mm width were used on the testing rigs. Also, CFRP sheets were used to wrap the specimen ends leaving only one third in the middle.

For all specimens, the failure mechanism was consistent. The initial failure occurred as a result of cracks within the concrete cover in the middle when maximum load was reached. This was followed by yielding and then fracture of the farthest bar or tube in the tension region. Afterward, the other bars or tubes in the tension region either fractured or kept yielding for a while and then fractured depending on the stresses distribution and corresponding generated cracks. The failure modes of specimens tested under flexural loading are shown in Figure 16. However, for specimens reinforced with steel tubes, the initial slope of load-midspan deflection diagrams in the specimens reinforced with tubes of a 350 MPa nominal tensile strength (ST33.7) was steeper than other specimens reinforced with tubes of a 250 MPa nominal tensile strength (ST26.9). It is clear that the large diameter ST33.7 tube have a higher second moment of area. This greater second moment of area provides a higher stiffness combined with the higher nominal tensile strength of steel in the ST33.7. Also, the effect of tensile strength on the maximum load was higher than the effect of the pitch. The lowest value of the maximum load was in Specimen ST26.9H75F and then Specimen ST26.9H50F.

For flexural load, the ductility of the specimens reinforced with steel tubes was lower than the specimens reinforced with steel bars. The ductility of Specimen ST33.7H50F and Specimen ST33.7H75F were less than the reference specimen by 29% and 33%, respectively (Table 6). The reason for this is that the tensile strength of steel tubes was lower than the tensile

strength of the steel bars. In addition, the effect of tensile strength longitudinal reinforcing steel on the ductility of specimens was higher than the effect of the spacing of the pitch. The lowest value of the ductility was observed in Specimen ST26.9H75F and Specimen ST26.9H50F (Table 6).

Interaction Diagrams

The experimental axial load-bending moment (P-M) interaction diagrams of all specimen types are shown in Figure 17. Four points that are concentric, 25 mm eccentric, 50 mm eccentric and flexural loading were used to draw the P-M interaction diagrams. The first point consisted of maximum axial load for specimens tested under concentric load. The second and the third points consisted of the maximum axial load for specimens tested under 25 and 50 mm eccentric loads and the corresponding bending moments, respectively. The corresponding bending moments at the mid-height of the columns under 25 and 50 mm eccentric loads were calculated as in Equation 1.

$$M = P_{Max}(e + \delta) \quad (1)$$

Where M and δ represent moment and lateral deformation corresponding to the maximum axial load (P_{Max}), respectively, and e represents the loading eccentricity. The fourth point consisted of only pure bending moment for the specimen tested under four point bending. The pure bending moments at mid span of the beams were calculated as in Equation 2.

$$M_p = (P_{Max}l)/6 \quad (2)$$

Where M_p represents pure bending moment corresponding to the maximum load (P_{Max}) of the tested beam specimen, and the l represents span length, or the distance between the supports.

Specimens of Group ST33.7H50 showed similar axial load-bending moment interaction behavior to the reference specimens of Group N16H50. At the concentric load and 25 mm eccentric load, the axial load and bending moment capacities of specimens of Group ST33.7H50 were similar to the axial load and the bending moment capacities of the reference specimens of Group N16H50. At 50 mm eccentric load, the maximum axial load of Specimen ST33.7H50E50 was lower than the maximum axial load of reference specimen by 5%. This is because the use of a single washer for the longitudinal steel tubes was insufficient to prevent the slip of tubes in the compression region of the specimen. For flexural load, the bending moment of Specimen ST33.7H50F was 3.4% lower than the bending moment of N16H50F due to the lower tensile strength of longitudinal tube reinforcement compared to steel bars. Increasing the pitch of the helices in the specimen from 50 mm to 75 mm resulted in the reduction of the axial load carrying capacity and bending moments under concentric, eccentric and flexural loads of Group ST33.7H75.

At the concentric load and 25 mm eccentric load, specimens of Group ST26.9H50 showed lower axial load carrying capacity compared to the specimens of Group ST33.7H50 because the lower tensile strength and small outside diameter steel tubes. However, at 50 mm eccentric load, placing the steel tubes with double washers in the compression region of the Specimen ST26.9H50E50 resulted in a higher axial load carrying capacity and corresponding bending moment compared to the Specimen ST33.7H50E50. Under flexural loads, the bending moment of the Specimen ST26.9H50F was 23.7% and 21% lower than the bending moment of reference Specimen N16H50F and Specimen ST33.7H50F, respectively. Similar to the specimens of Group ST33.7H75, increasing the pitch of the helices in the specimen from 50 mm to 75 mm resulted in the reduction of the axial load carrying capacity and bending moments under concentric, eccentric and flexural loads of Group ST26.9H75. This

indicates that the use of steel tubes with similar tensile strength of the steel bars may show higher axial load carrying capacity and corresponding bending moment in STR SCC specimens. However, the slip of steel tubes should be prevented. Adequate use of washers may be considered a viable solution.

Theoretical axial load-bending moment (P-M) interaction diagrams of all specimen types were determined by layer-by-layer integration method. The cross-section of concrete is assumed to consist of small parallel layers with a small thickness (t) and a variable width (b_n), as shown in Figure 18. The number of layers (n) is calculated by dividing the diameter of the cross-section (D) on the thickness of each layer. The cross-section was divided into 240 small layers by assuming the thickness of each layer equal to 1 mm. Based on the assumptions that plane sections remain plane after bending, the strain ($\epsilon_{c,n}$) in the center of each layer can be calculated according to the linear distribution of strain, as a function of the depth of neutral axis (d_n) (Figure 18). The d_n is represented the distance from the extreme concrete compressive fiber to the neutral axis ($N.A$). After calculating the strain in each concrete layer, the corresponding stress value ($f_{c,n}$) on the center of each layer is calculated according to the stress-strain model for unconfined concrete in Aslani and Nejadi (2012). The tensile strength of concrete is ignored in the calculations. The unconfined concrete strength is taken as 83% of 28-day cylinder compressive strength according to AS 3600-2009 (AS 2009). The ultimate strain in extreme concrete compressive fiber has been considered as 0.003 according to AS 3600-2009 (AS 2009).

The force reaction in the center of each concrete layer ($P_{c,n}$) is calculated by multiplying the stress in each layer by the corresponding area of concrete in each layer ($A_{c,n}$). The moment for each layer is calculated by multiplying the force in each layer by the distance from center

of each layer to the centerline of the cross-section. The total force of the concrete cross-section is calculated as the summation of the forces acting on the strips. The tensile forces are considered negative while the compressive forces are positive. In addition, the total moment of the concrete cross-section is calculated as the summation of the moments with respect to the centerline of the cross-section.

Figure 19 compares the theoretical and the experimental P–M interactions for the specimens tested in this study. It was found that the theoretical results were in good agreement with the experimental results. The experimental bending moment of specimens reinforced with N16 steel bar, ST33.7 steel tube and ST26.9 steel tube tested under four-point load bending was relatively greater than the calculated bending moment. This might be because specimens tested as beam had shear span shorter than twice the effective depth of the concrete cross-section.

Conclusions

Use of small diameter circular steel tubes filled with Self-Compacting Concrete (SCC) in lieu of conventional steel bars were investigated in reinforcing the concrete column. Hence, a new reinforcing method of the concrete column specimens was proposed in this paper. Problems associated with the use of steel tubes as longitudinal reinforcement in columns due to slip of steel tubes have been highlighted. The following conclusions can be drawn.

1. As the tensile strength of steel bars and steel tubes are different, the force contribution of ST33.7 steel tubes in Specimen ST33.7H50 was found to be less than the force contribution of N16 steel bars in the reference Specimen N16H50 by 19.9%. In spite of less force contribution of steel tubes, the column Specimen ST33.7H50 had similar yield and maximum load as the reference Specimen N16H50 under concentric and 25 mm

- eccentric axial compression. The yield loads of the Specimens N16H50 and ST33.7H50 were 2505 and 2500 kN, respectively. The maximum loads of the Specimens N16H50 and ST33.7H50 were 2734 kN and 2729 kN, respectively. Hence, using steel tubes with similar tensile strength to the steel bars might result in a higher maximum load capacity.
2. For concentric load and 25 mm eccentric load, specimens reinforced with ST33.7 steel tubes had maximum load capacity greater than specimens reinforced with ST26.9 steel tubes. For 50 mm eccentric load, however, Specimen ST26.9H50E50 showed maximum load higher than Specimen ST33.7H50E50 due to the slip of ST33.7 steel tubes.
  3. In spite of the cross sectional area of ST33.7 and ST26.9 steel tubes are the same, increasing the pitch of helices from 50 to 75 mm resulted in a higher reduction in the maximum axial load of specimens reinforced with ST26.9 tubes compared to specimens reinforced with the ST33.7 tubes.
  4. Under flexural loading, the maximum loads of the Specimens ST33.7H50F and ST26.9H50F were 7% and 25%, respectively, lower than the reference Specimen N16H50F. In addition, the initial slope of load-midspan deflection diagrams in the specimens reinforced with ST33.7 tubes (nominal tensile strength=350 MPa) was steeper than the specimens reinforced with ST26.9 tubes (nominal tensile strength=250 MPa).
  5. The ductility of Steel Tubes Reinforced Self-Compacting Concrete (STR SCC) specimens was significantly influenced by the slip of steel tubes under eccentric loading. However, the STR SCC specimens showed a higher ductility than the reference Specimen N16H50C under concentric loading.
  6. Welding a single washer at the ends of the steel tubes to prevent slip of steel tubes in concrete was insufficient and resulted in tearing of the tubes near the welding area at both ends of the tube in the tension and compression regions. In contrast, welding double washers at the ends of the tubes prevented the slip of longitudinal steel tubes in concrete.

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Table 1: Mix proportion of the SCC used in this study

Material	Quantity
Cement	280 kg/m ³
Fly ash	120 kg/m ³
Slag	50 kg/m ³
Fine aggregate	950 kg/m ³
Coarse aggregate	780 kg/m ³
Water	182 kg/m ³
High Range Water Reducer (HRWR)	3.375 l/m ³
Water/Powder ratio	0.4

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Table 2: Details of tested specimens

Group Name	Specimen Labels	Longitudinal Reinforcement				Transverse Reinforcement		Loading Modes
		No. of Bars or Tubes	Outside Diameter of Bars or Tubes (mm)	Thickness of Tubes, (mm)	Reinforcement Ratio, ρ_s (%)	Pitch (mm)	Reinforcement Ratio, ρ_s (%)	
N16-H50	N16-H50-C	6	16 (N16)	—	2.67	50	3.3	Concentric
	N16-H50-E25	6	16 (N16)	—	2.67	50	3.3	e = 25 mm
	N16-H50-E50	6	16 (N16)	—	2.67	50	3.3	e = 50 mm
	N16-H50-F	6	16 (N16)	—	2.67	50	3.3	Flexural
ST33.7-H50	ST33.7-H50-C	6	33.7	2	2.64	50	3.3	Concentric
	ST33.7-H50-E25	6	33.7	2	2.64	50	3.3	e = 25 mm
	ST33.7-H50-E50	6	33.7	2	2.64	50	3.3	e = 50 mm
	ST33.7-H50-F	6	33.7	2	2.64	50	3.3	Flexural
ST33.7-H75	ST33.7-H75-C	6	33.7	2	2.64	75	2.2	Concentric
	ST33.7-H75-E25	6	33.7	2	2.64	75	2.2	e = 25 mm
	ST33.7-H75-E50	6	33.7	2	2.64	75	2.2	e = 50 mm
	ST33.7-H75-F	6	33.7	2	2.64	75	2.2	Flexural
ST26.9-H50	ST26.9-H50-C	6	26.9	2.6	2.63	50	3.3	Concentric
	ST26.9-H50-E25	6	26.9	2.6	2.63	50	3.3	e = 25 mm
	ST26.9-H50-E50	6	26.9	2.6	2.63	50	3.3	e = 50 mm
	ST26.9-H50-F	6	26.9	2.6	2.63	50	3.3	Flexural
ST26.9-H75	ST26.9-H75-C	6	26.9	2.6	2.63	75	2.2	Concentric
	ST26.9-H75-E25	6	26.9	2.6	2.63	75	2.2	e = 25 mm
	ST26.9-H75-E50	6	26.9	2.6	2.63	75	2.2	e = 50 mm
	ST26.9-H75-F	6	26.9	2.6	2.63	75	2.2	Flexural

Table 3: Results of specimens tested under concentric loading

Specimen	N16H50C	ST33.7H50C	ST26.9H50C	ST33.7H75C	ST26.9H75C
Yield load (kN)	2505	2500	2375	2395	2275
Corresponding axial deformation (mm)	2.35	2.45	2.35	2.65	2.2
Maximum load (kN)	2734	2729	2598	2633	2443
Corresponding axial deformation (mm)	3.08	3.29	3.03	3.44	2.79
Ultimate axial deformation (mm)*	20	33.5	36	26.2	30.4
Ductility	16.96	22	22.84	13.91	18.91

* Ultimate axial deformation was defined by the fracture of the steel helices.

Table 4: Results of specimens tested under 25 mm eccentric loading

Specimen	N16H50E25	ST33.7H50E25	ST26.9H50E25	ST33.7H75E25	ST26.9H75E25
Yield load (kN)	1690	1685	1650	1675	1545
Corresponding axial deformation (mm)	2.6	2.4	2.5	2.5	2.35
Maximum load (kN)	1824	1820	1780	1782	1631
Corresponding axial deformation (mm)	3.11	2.93	3.24	3.18	2.86
Corresponding lateral deformation (mm)	2.49	2.42	2.95	2.04	2.47
Ultimate axial deformation (mm)*	26.3	20.2	20.7	20	20.2
Ductility	13.1	8.56	8.3	7.23	7.37

* Ultimate axial deformation was defined by 75% of the maximum load.

Table 5: Results of specimens tested under 50 mm eccentric loading

Specimen	N16H50E50	ST33.7H50E50	ST26.9H50E50	ST33.7H75E50	ST26.9H75E50
Yield load (kN)	1200	1145	1185	1150	1100
Corresponding axial deformation (mm)	2.5	2.4	2.5	2.4	2.35
Maximum load (kN)	1300	1233	1267	1240	1175
Corresponding axial deformation (mm)	2.78	2.84	2.9	2.92	2.73
Corresponding lateral deformation (mm)	2.94	2.87	3	3.5	3.24
Ultimate axial deformation (mm)*	20.7	14.7	20.7	18.4	25.1
Ductility	11.54	8.7	8.63	7.56	9.78

* Ultimate axial deformation was defined by 75% of the maximum load.

Table 6: Results of specimens tested under flexural loading

Specimen	N16H50F	ST33.7H50F	ST26.9H50F	ST33.7H75F	ST26.9H75F
Yield load (kN)	510	490	385	458	350
Corresponding midspan deflection (mm)	4.5	4.95	5.1	4.75	4.35
Maximum load (kN)	592	541	443	511	401
Corresponding midspan deflection (mm)	26.8	7.2	9.07	6.59	6.02
Ultimate midspan deflection (mm)*	44.1	47.5	39.8	37.9	23.9
Ductility	17.55	12.49	9.63	11.7	8.52

* Ultimate midspan deflection was defined by the fracture of the bar/tube.

Group N16-H50

Group ST33.7-H50

Group ST33.7-H75

Group ST26.9-H50

Group ST26.9-H75

Circular template 1

Circular template 2

(a) Loading heads

(b) Setup details of specimen

Cross-section divided
into n number of layers

Strain distribution

Corresponding stress
and force distribution