Experimental analysis of the cyclic response of CFT-SHS members

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ABSTRACT: Aiming at the evaluation of the ultimate behaviour of Concrete Filled Tubular (CFT) members subjected to bending moment, an experimental program is currently in progress at the Materials and Structures Laboratory of the Department of Civil Engineering of Salerno University. In particular, Square Hollow Section CFT beam-columns are being investigated, both under monotonic and cyclic loading conditions. The three point bending scheme is adopted for testing specimens, where an hydraulic actuator is used for the transmission of the transverse load at midspan under displacement control. An adequate measurement system is adopted during the tests. In particular, in addition to the main transverse displacement at midspan, longitudinal deformations along the cross section perimeter are measured by means of strain gauges, aiming to the experimental evaluation of moment-curvature relationships as well as force-displacement curves for each specimen. This paper presents the structural details of six specimens and the corresponding experimental results. The members have shown high ductility and energy dissipation capacity.

1 INTRODUCTION

During last years, the use of composite structures is widely increased due to the their characteristics of stiffness, resistance and ductility. At the same time, researchers have more and more focused their attention on the design issues concerning the behaviour of such structural typology.

Composite members belong to two different categories: Concrete Filled Tubular (CFT) and Concrete Encased Composite (CEC) members. Several advantages characterize the first typology if compared with the second one. In fact, the distribution of structural materials (i.e. the location of steel along the perimeter and of concrete in the core) allows to increase both the flexural capacity of the section and the stiffness of the member. Furthermore, the steel profile provides transversal action on the concrete core assuring confinement effects which significantly improve concrete resistance and ductility. Moreover, the inward buckling of steel plates is avoided due to the presence of the concrete, even if outward buckling is still possible. Finally, the steel tube prevents concrete spalling.

The behaviour of a composite member is governed by the interaction between the two materials constituting the section. An accurate modeling of such interaction is necessary to predict the ultimate response of composite members. In particular, in the following Sections these topics will be briefly examined, in order to point out all the parameters involved. The presented experimental program is also aimed at the validation of a fiber model to predict the ultimate behaviour of CFT members.

2 INTERACTION BETWEEN STEEL AND CONCRETE

In CFT members, the interaction between steel and concrete is mainly responsible of three effects: the confinement of concrete; the bi-axial stress state of steel; the local buckling modes of the steel profile. Due to all these aspects, the response of the cross section is different from the sum of those corresponding to the steel tube and the concrete column, separately.

2.1 Confinement of concrete

The effect of confinement of the concrete core is due to the fact that the Poisson ratios of the two materials are different. In fact, as far as the axial deformation increases the Poisson ratio of concrete increases, becoming greater than that of steel. At this state, the steel tube avoids the lateral expansion of concrete, so that radial compressive stresses arise in concrete. Under these conditions, concrete is subjected to a tri-axial stress state, while the steel profile is subjected to a bi-axial stress state (Shams & Saadeghvaziri 1999; Elremaily & Azizinamini 2002).

As it is known, the confined concrete exhibits an increase of both resistance and ductility if compared to the unconfined material. In order to well describe such behaviour, several analytical formulations are available in the technical literature to obtain an appropriate constitutive law. Within them, the most accredited is probably the one proposed by Mander et al. (1988), in which the compressive strength of confined concrete is calibrated on the basis of the lateral
effective stress acting on the material. In particular, starting from the in-plane equilibrium of the cross section, the lateral effective stress \( f_{lx} \) is related to the hoop stresses \( \sigma_\theta \) arising in the steel profile (Fig. 1).

According to this model, the entire stress-strain relationship is obtained depending on the moduli of elasticity and on the ultimate strain level of concrete. The softening branch of the material constitutive law is also provided.

2.2 Bi-axial stress state in steel profile

As previously discussed, due to the lateral expansion of concrete, the steel profile is subjected to additional stresses acting along the mid-thickness line of the cross section. This stress state is superimposed to the stresses acting in the longitudinal direction of the member, so that a bi-axial stress state occurs.

With reference to the von Mises yield criterion (Fig. 2), the resulting values of the yield stress in tension and in compression become different. In particular, since the hoop stresses \( \sigma_\theta \) are tensile stresses, the compressive stress at yielding \( f_{yc} \) is smaller than the uni-axial nominal yield stress \( f_y \), while the tensile stress \( f_{yt} \) at yielding is greater than \( f_y \).

As a consequence, an asymmetrical stress-strain relationship has to be considered. In particular, Elremaily & Azizinamini (2002) suggest an average value of \( \sigma_\theta = 0.1 f_y \) with reference to CHS columns, regardless of the slenderness of steel profile. By assuming this value, the following values for the compressive and tensile yield stresses of steel are obtained: \( f_{yc} = -0.946 f_y \) and \( f_{yt} = 1.046 f_y \).

2.3 Local buckling of steel plates

As already discussed, the interaction between the concrete core and the steel profile has a great influence on the buckling modes of steel plates of CFT columns. In fact, inward deformations are prevented, so that only outward local buckling is possible. As all the plate elements are constrained to buckle outward, the rotation at corners is prevented, so that in case of SHS columns the local buckling of each plate requires the formation of yield lines at both edges. It means that buckling of compressed plates requires a number of yield lines greater than those needed in case of empty profiles. As a result, the internal work required for the development of the kinematic mechanism is greater, so that local buckling is delayed, even though not avoided.

Obviously, it can be observed that the slenderness ratio, i.e. the ratio between the width \( W \) of the tube plate elements and the thickness \( t \), is the parameter governing local buckling also in the case of composite columns. In fact, for slender sections, i.e. high values of \( W/t \), local buckling of the steel tube occurs at the peak load, while for thick sections, i.e. lower values of \( W/t \), the steel tube buckles after the peak load is reached. In both cases, however, ductile behaviour is observed even after the occurrence of local buckling (Shams & Saadeghvaziri 1999).

Due to the formation of bulges, a reduction of the flexural capacity of the beam-column could be expected. In fact, the steel buckled plate is responsible of a resistance decrease of the section. In addition, local buckling gives rise to the separation of steel and concrete. At this stage, however, the concrete maintains its compressive capacity until separation is complete. Consequently, even though the CFT column cannot sustain higher loads after the occurrence of local buckling it still maintains the load-carrying capacity.

From the analytical point of view, a useful approach to account for local buckling of compressed plates of SHS columns is constituted by the effective width approach (Mastrandrea et al. 2008).

3 EXPERIMENTAL PROGRAM

An experimental program has been almost completed at the Materials and Structures Laboratory of the Department of Civil Engineering of Salerno University.

The tests are devoted to SHS columns filled with concrete. In the following Sections all the structural
details of specimens, the testing devices and the experimental results are summarized.

3.1 Specimens

Nowadays, six CFT SHS columns have been tested; two additional tests will complete the planned experimental program. The main test parameter is represented by the width-to-thickness ratio $D/t$. Detailed information about the specimens are provided in Table 1.

Each specimen was manufactured from a rolled steel sheet, folded and welded along one longitudinal side. At the ends of the specimen, steel square plates (20 mm thick) were welded; one of them is holed, to allow concrete filling. The specimens were placed upright and filled with concrete, then waiting for its complete cure. In order to recover the little longitudinal shrinkage at the top of column, high strength epoxy was used to fill the gap and let the end surface be uniform between concrete and steel. The length of specimens is equal to 2400 mm.

The specimens to be tested have been located horizontally and fixed to the supports by means of bolted connections, to assure the continuity of the member. Therefore, the actual length of tested specimens, i.e. the distance between the supports, is equal to 3000 mm (Fig. 3).

3.2 Material properties

Three coupon specimens have been prepared for each steel tube. Coupon tests have been carried out to evaluate the average maximum strength $f_{su}$ and the actual stress-strain relationship. The ultimate stress is provided in Table 1. In addition, Figure 4 shows a typical stress-strain relationship.

Figure 4. Stress-strain diagram for steel (specimen S5).

Even though all the steel tubes were filled with the same concrete cast, for every specimen the material was tested in compression at the same date of each experimental test, in order to establish the actual concrete compression strength. To this scope, four 150 mm concrete cubes were taken for each specimen. The average compression strength results are given in Table 1 for each specimen.

3.3 Tests description

Three point bending tests under cyclic loading conditions have been performed. No axial load was applied to the specimens, because the experimental program is focused on the response of composite bridge piers. In fact, the axial load acting on bridge piers is a small part of the squash load (usually about 0.10 times the squash load). This means that the flexural behaviour governs the response of such structural members, while the axial load is not particularly significant. However, the second part of the planned experimental activity will be devoted also to the investigation of CFT members subjected to axial force and bending.

Figure 3 provides a schematic view of the test setup. The ends of the specimen are free to rotate in-plane due to the connection to pin jointed bearings. Therefore, a simply supported scheme is adopted to test the beam-column. The distance between the two pins, i.e. the actual length of specimens, is equal to 3000 mm. However, the results can be easily extended to a cantilever beam-column scheme (which is relevant for bridge piers) by considering an half member. One of the two supports presented slotted hole, in order to allow free sliding of the specimen in the longitudinal direction.

![Figure 3. Test setup.](image)

Table 1. Summary of test information.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$D \times t$ mm</th>
<th>$D/t$</th>
<th>$f_{su}$ MPa</th>
<th>$f_{cu}$ MPa</th>
<th>$P_u$ kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2</td>
<td>300 × 6.0</td>
<td>50.0</td>
<td>604</td>
<td>32.1</td>
<td>778</td>
</tr>
<tr>
<td>S4</td>
<td>300 × 6.0</td>
<td>50.0</td>
<td>604</td>
<td>32.1</td>
<td>750</td>
</tr>
<tr>
<td>S5</td>
<td>220 × 7.4</td>
<td>29.7</td>
<td>604</td>
<td>43.1</td>
<td>490</td>
</tr>
<tr>
<td>S6</td>
<td>220 × 4.8</td>
<td>45.8</td>
<td>444</td>
<td>43.9</td>
<td>240</td>
</tr>
<tr>
<td>S7</td>
<td>220 × 4.8</td>
<td>45.8</td>
<td>444</td>
<td>38.9</td>
<td>272</td>
</tr>
<tr>
<td>S8</td>
<td>220 × 7.8</td>
<td>28.2</td>
<td>554</td>
<td>38.8</td>
<td>508</td>
</tr>
</tbody>
</table>
The flexural loading has been applied, in the middle of the member, by means of an hydraulic actuator having 2500 kN capacity in tension and 3000 kN in compression. The connection between the specimen and the actuator is constituted by means of a rigid stub located in the middle of the specimen. The stub is realized by means of two rigid plates on the horizontal sides of the specimen, joined by means of steel bars. Therefore, loads both in tension and in compression can be applied to the specimen. The length of the rigid stub is equal to 460 mm.

Several measuring devices have been adopted to record displacements and deformations. The measure of the displacement at force location has been carried out by means of the LVDT of the hydraulic actuator and also by means of displacement transducer. Additional displacement transducers located along the length of the specimen have been adopted to measure its deflected shape. In addition, strain gauges have been applied on the vertical lateral sides of the steel box, in two control sections, one on the right side and one on the left side of the rigid stub. These sections are those characterized by the occurrence of the maximum value of the bending moment. They are placed at 1250 mm distance from each support center of rotation. In particular, longitudinal strain gauges were applied on the webs at 55 mm distance from the top and the bottom flanges of the tube, respectively. In this way, deformations close to the maximum values are measured. Finally, in order to evaluate transversal deformations, two strain gauges have been applied also transversally, one for each control section.

Tests have been carried out under displacement control, with a maximum imposed displacement of 140 mm by means of the piston of the hydraulic actuator. The displacement history for all the specimens, with exception of specimen S4, is characterized by a slow ramp in compression until the maximum piston stroke (140 mm) is reached, in order to reach the maximum monotonic displacement allowed by the testing equipment. After an unloading branch, a cyclic displacement history has been applied. The typical displacement history applied to the specimens is depicted in Figure 5 with reference to specimen S5.

Only in the case of specimen S4 a different displacement history has been applied. In fact, after the first loading phase up to the maximum piston stroke and the corresponding unloading, the test has been interrupted and the specimen has been disconnected from the actuator. Successively, piston has been regulated at the middle of its stroke, and the connection with the specimen has been re-established. The following cycles have been still carried out exploiting the maximum stroke (140 mm), but cycling in the range ±70 mm.

Experimental tests have been continued until either the specimen fails due to the fracture of the steel tube at the corners with concrete leakage, or at least a 50% loss of the lateral load resistance is observed.

4 TEST RESULTS AND DISCUSSION

All the specimens have shown a ductile response. The results are herein summarized by means of force-displacement curves and moment-curvature relationships.

In particular, force-displacement curves represent the relationships between the lateral load and the deflection at the middle of the specimens. They are directly provided by the actuator data. Instead, moment-curvature relationships are obtained by means of deformations measured through the strain gauges, so that they are referred to the sections adjacent to the rigid stub. Since four measures are available (two control sections and two webs for each one), the moment-curvature relationships have been obtained considering the average values of the deformations. However, the moment-curvature relationship has been evaluated only for the first loading branch, because due to great deformations, strain gauges went off-scale.

In Figures 6–9 the force-displacement curves for specimens S2, S4, S5 and S6 are shown, while in Figures 10–13 the experimental moment-curvature relationships for specimens S2, S5, S6 and S8 are depicted. The maximum lateral loads achieved for all the tests is reported in Table 1 (\(P_u\)).

As shown by the force-displacement curves, each specimen has shown a satisfactory ductility. In fact, hysteresis cycles are large and stable, without sudden loss of strength. Many cycles are necessary to achieve 50% loss of strength. In particular, as far as the ratio \(D/t\) increases the number of cycles to fracture decreases. This is due to local buckling, which affects more and more the global behaviour as the local slenderness of the plate elements constituting the steel profile increases.

In all the cases, bulges formed at both top and bottom flanges of steel tubes, close to the rigid stub,
Figure 6. Specimen S2 lateral load-deflection curve.

Figure 7. Specimen S4 lateral load-deflection curve.

Figure 8. Specimen S5 lateral load-deflection curve.

Figure 9. Specimen S6 lateral load-deflection curve.

Figure 10. Specimen S2 moment-curvature relationship.

Figure 11. Specimen S5 moment-curvature relationship.

Figure 12. Specimen S6 moment-curvature relationship.

Figure 13. Specimen S8 preliminary comparison between experimental and analytical moment-curvature relationships.
where the bending moment assumed the maximum value. Since the first displacement ramp was downward the first bulge always formed on the top flange, i.e. the steel plate in compression. Then, the bulge formed also on the other flange when the lateral displacement was reversed.

Regarding the small slips occurring in same cases at load reversal points (Figs. 6–7), they are probably due to the settlement of the connecting system of the rigid stub embracing the specimen.

However, with reference to local buckling it can be underlined that, despite of buckling, no significant softening branch is observed in force-displacement curves. Conversely, referring to the first loading branch, the maximum lateral load is almost sustained without any loss of strength. This means that CFT columns exhibit high global ductility and are able to absorb a great amount of input energy.

With reference to transversal strain-gauges, applied on the upper part of the webs to evaluate the deformations due to the interaction between the steel tube and the concrete core, even if the corresponding longitudinal stress field was in compression, they measured tensile deformations. This agrees the phenomenon described in Section 2.2. It means that, due to the interaction between the steel tube and the confined concrete, a bi-axial stress state arises in the steel plate elements, being hoop stresses in tension.

Finally, in Figure 13 a preliminary comparison between the experimental moment-curvature relationship and the one carried out by means of a numerical procedure is depicted in the case of specimen S8. In particular, the above mentioned numerical procedure has been developed by the same authors by means of a fiber model (Mastrandrea et al. 2008) accounting for concrete confinement according to Mander et al. (1988), bi-axial stress state of steel according to Elremaily & Azizinamini (2002) and local buckling by means of the effective width approach. The comparison is encouraging, being the analytical curve close to the experimental one. However, further calibrations to improve the accuracy of the numerical procedure and the validation of the fiber model, exploiting test results, are still in progress.

5 CONCLUSIONS

In this paper, an experimental program dealing with concrete filled tubular beam-columns with square hollow steel sections has been presented.

In particular, six specimens up-to-now have been tested according to three point bending scheme with the aim to evaluate the ultimate response of the member under flexural cyclic loading conditions. No axial load has been applied.

Test results have been reported and discussed, showing satisfactory performances of CFT beam-columns under seismic loading conditions. In fact, high resistance and ductility have been recognized, while local buckling does not significantly affect the global behaviour of tested specimens.

At present time, numerical procedures are being calibrated by the Authors in order to evaluate moment-curvature relationship and force-displacement curve for CFT columns (Mastrandrea et al. 2008). However, the preliminary comparison between the numerical fiber model and test results are encouraging.

Experimental tests will be used to validate such fiber model, providing the designers with useful tools to foresee the behaviour of steel-concrete composite beam-columns.

ACKNOWLEDGEMENT

This work has been supported by Italian research grant DPC RELUIS 2005–2008.

REFERENCES


