Confinement of Concrete Filled Steel Tubular Columns - New Frontiers

Abdelqader Najmi *

ABSTRACT

A new type of composite columns consists of tubular sections, filled with concrete has been investigated. Special link connectors are used to join the steel shell with the concrete inside. The method of connecting the two materials brings the level of confinement of concrete in this type of columns to values never reached before. The level of confinement reached surpasses by far the values obtained in spirally reinforced concrete columns when considering ultimate axial load. Concrete strength can be increased by more than 50%. These composite columns attain large axial strains at ultimate load of the order of 2% and more, such strains are well outside the plastic strains of steel and almost (6-10) times the concrete compression crushing strain of 0.3%. The resulting integrity of the cross section goes beyond preventing elastic local buckling of the steel shell; to sustaining the stiffness of the cross section in the plastic zone. Ultimate loads of connected composite concrete columns take place by the plastic buckling of the steel tube, and not by the crushing of concrete.

Keywords: Composite columns, Concrete, Steel, Confinement, U-shaped links.

INTRODUCTION

Concrete in compression is usually characterized with a stress-strain relationship obtained from uniaxial standard compression tests. However, most concrete structural elements are subjected to a multi-axial stress state. A uniaxial stress state represents only one of an infinite number of multi-axial stress conditions to which an element of concrete in a structure may be subjected throughout the loading history of the structure; see Kotsovos (1987). The response of concrete varies widely for different stress states and it is therefore important to know how the concrete behaves for different multi-axial stress states. As an example, Kotsovos shows the variation of the peak axial compressive stress sustained by a concrete cylinder with increasing confining pressure. It was noted that a small confining pressure of about 10 percent of the uniaxial cylinder compressive strength was sufficient to increase the load-bearing capacity of the specimen by as much as 50 percent.

Both the strength and ductility of concrete are increased substantially under conditions of tri-axial compression. Richart et al. (1928) found the following relationship for the strength of concrete cylinders loaded axially to failure while subjected to confining fluid pressure $f_c$:

$$ f_c' = f_c + \lambda f_l $$

(1)

The value of $\lambda$, the lateral stress coefficient = 4.1, however, Balmer (1949) suggested a range between 4.5 and 7.0. $f_c'$ is the enhanced axial stress, and $f_l$ is the compressive cylinder strength of concrete.

A new model was proposed for concrete confined by spiral reinforcement based on concrete-transverse steel interaction. The two main parameters were concrete strength and lateral stress lateral strain relationship that represents the response characteristics of the transverse steel to the lateral expansion of concrete. Assa et al. (2001) modeled a confinement mechanism and limited the lateral expansion of the confined concrete with the

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maximum lateral expansion capacity.

Assa et al. reached some relationships expressed in the following equation:

\[ f_{cc}^* = f_c^* + 3.36 f_1 \quad (2) \]

Abdel Fattah (2008) gives a literature review on the subject of confined concrete in circular columns

Composite columns may be steel-reinforced concrete columns (SRC) or concrete-filled steel tubes (CFT). The LRFD Specification does not provide detailed requirements for reinforcing bar spacing, splices, tie areas, concrete strength and other structural provisions. Designers normally follow the ACI 318 Code for situations not clearly covered by the LRFD Specification. Residual stresses in the steel are ignored. Failure occurs when the extreme fiber compressive strain in concrete reaches a limiting value of 0.003.

The contribution of each component of an axially loaded column to its overall strength is difficult to determine. The AISC Specification considers a stress of 0.95\(f_c^*\) for the concrete component of a round HSS filled with concrete to account for the effects of concrete confinement when plastic distribution is attained in pure compression (AISC Steel construction manual July 2006). The Steel code also requires the cross-sectional area of the steel HSS used shall comprise at least 1% of the total composite area, and the maximum \(b/t\) ratio for a rectangular HSS used as a composite column shall be equal to \(2.26\sqrt{Fy/Fc}\). \(b\) is the overall outside dimension of the rectangular HSS. \(t\) is the thickness of its wall. The maximum \(D/t\) ratio for a round HSS filled with concrete shall be 0.15\(E/F_y\).

The concrete fill adds stiffness and compressive strength to the tubular column and reduces the potential for local buckling. The tube acts as a formwork for the concrete, thus saving in the construction cost. The structural benefits require stress transfer between the steel and the concrete in order to ensure their composite action. Both the shear connectors and the natural bond transfer stresses between the two materials. It is believed that the bond strength has a significant effect on the behavior of the CFT columns; see Johansson and Gylltoft (2002).

However, numerous tests have been carried out within this area showing uncertainty about the effect of bond strength. In addition, the stress transfer is not well understood; see Shams and Saadeghvaziri (1997), and Kilpatrick and Rangan (1999). The bond transfer between the steel tube and the concrete fill depends on the radial displacements due to the pressure of the wet concrete on the shell and the shrinkage of the concrete core, together with the rugosity of the interior surface of the tube. It was shown that the average bond stress for rectangular tubes is 70% smaller than the average bond stress for circular tubes. Large diameter tubes with large \(d/t\) ratios revealed that shrinkage of the concrete could lead to very little bond stress capacities. \(d\) represents the diameter of the tube and \(t\) is the thickness of the tube. However small diameter tubes with small \(d/t\) ratios developed large bond stresses (Roeder, et al. 1999).

In a composite column, both the concrete and steel carry axial load. The load is distributed between the steel and concrete in accordance with compatibility of strain. The SSRC (1998) report stipulated that columns with a ratio of encased steel area of \(A_s/A_y > 0.04\) should be designed as composite and those with \(A_s/A_y < 0.04\) should be designed as reinforced concrete. Accordingly, the AISC-LRFD Specification limits its definition and requirements for composite columns to members having \(A_s > 0.04A_y\).

For short encased columns the resulting squash load is:

\[ P_a = 0.85 f_c A_y + f_{ys} A_x + f_{ys} A_z \quad (3) \]

Lakshmi and Shanmugam (2002): A “short column” refers to a compression member that can attain its ultimate capacity, known as squash load, without overall buckling. The column must be straight and subjected to axial load. In the early stages of loading, the Poisson’s ratio for concrete is lower than that for steel and the steel tube has no restraining effect on the concrete core.

Objectives: To use special U –links to enhance the structural role of the concrete inside the steel tubular section, and to prove experimentally its effectiveness on the global behavior of the tubular columns.
EXPERIMENTAL INVESTIGATION

1. Steel Tubes:
   In order to test the “lateral separation-confining hypothesis”, initially, four different groups of test specimens were investigated. All specimens in the same group were cut from the same steel tube, and have the same length. They also have the same concrete-fill, and only differ in the number of U-links installed in the tubes. The test specimens were labeled such that the group and the specimen could be identified. For example “C1-40”: C1 defines group C1 and 40 refers to U-link spacing of 40 millimeters.

1. Square Test Specimens – Group-1
   Three test specimens of square cross-section, with \(L/B = 3.79\) were tested under concentric axial compression. Table 1 summarizes the dimensions and material properties of the specimens, with \(A_s\) and \(f_y\) representing the cross-sectional area and the yield strength of the steel tube, respectively; \(f_{c'}\) denoting the compressive cylinder strength of the concrete. L denotes the total height of the specimen, and B denotes the overall width of the square cross-section.

2. Circular Tubes Specimens – Group-2
   Four test specimens of circular cross-section, with \(L/d = 3.03\) were tested under concentric axial compression. Table 2 gives the dimensions and material properties of the specimens; \(d\) denotes the external diameter of the circular cross-section.

3. Circular Tubes Specimens – Group-3
   Table 3 gives the dimensions and material properties.

4. Rectangular Tubes Specimens – Group 4
   Table 4 gives the dimensions and material properties.

### Table 1. Group-1 Square Steel Tube Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>B (mm)</th>
<th>t (mm)</th>
<th>L/B</th>
<th>(A_s) (mm(^2))</th>
<th>(f_y) (MPa)</th>
<th>(f_{c'}) (MPa)</th>
<th>(A_s) (\frac{A_s}{A_{total}})</th>
<th>Links</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-00</td>
<td>140</td>
<td>3.77</td>
<td>3.79</td>
<td>2056</td>
<td>320</td>
<td>30</td>
<td>0.105</td>
<td>-</td>
</tr>
<tr>
<td>C1-40</td>
<td>140</td>
<td>3.77</td>
<td>3.79</td>
<td>2056</td>
<td>320</td>
<td>30</td>
<td>0.105</td>
<td>40</td>
</tr>
<tr>
<td>C1-30</td>
<td>140</td>
<td>3.77</td>
<td>3.79</td>
<td>2056</td>
<td>320</td>
<td>30</td>
<td>0.105</td>
<td>30</td>
</tr>
</tbody>
</table>

### Table 2. Group-2 Filled Circular Steel Tube Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>d (mm)</th>
<th>t (mm)</th>
<th>L/d</th>
<th>(A_s) (mm(^2))</th>
<th>(f_y) (MPa)</th>
<th>(f_{c'}) (MPa)</th>
<th>(A_s) (\frac{A_s}{A_{total}})</th>
<th>Links</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2-00</td>
<td>165</td>
<td>4.19</td>
<td>3.03</td>
<td>2361</td>
<td>355</td>
<td>23.7</td>
<td>0.11</td>
<td>-</td>
</tr>
<tr>
<td>C2-40</td>
<td>165</td>
<td>4.19</td>
<td>3.03</td>
<td>2361</td>
<td>355</td>
<td>23.7</td>
<td>0.11</td>
<td>40</td>
</tr>
<tr>
<td>C2-30</td>
<td>165</td>
<td>4.19</td>
<td>3.03</td>
<td>2361</td>
<td>355</td>
<td>23.7</td>
<td>0.11</td>
<td>30</td>
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<tr>
<td>C2-20</td>
<td>165</td>
<td>4.19</td>
<td>3.03</td>
<td>2361</td>
<td>355</td>
<td>23.7</td>
<td>0.11</td>
<td>20</td>
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</tbody>
</table>

### Table 3. Group-3 Filled Circular Steel Tube Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>d (mm)</th>
<th>t (mm)</th>
<th>L/d</th>
<th>(A_s) (mm(^2))</th>
<th>(f_y) (MPa)</th>
<th>(f_{c'}) (MPa)</th>
<th>(A_s) (\frac{A_s}{A_{total}})</th>
<th>Links</th>
</tr>
</thead>
<tbody>
<tr>
<td>C3-00</td>
<td>219.5</td>
<td>4.79</td>
<td>2.74</td>
<td>3241</td>
<td>430</td>
<td>27.1</td>
<td>0.086</td>
<td>-</td>
</tr>
<tr>
<td>C3-20</td>
<td>219.5</td>
<td>4.79</td>
<td>2.74</td>
<td>3241</td>
<td>430</td>
<td>27.1</td>
<td>0.086</td>
<td>10</td>
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</tbody>
</table>

### Table 4. Rectangular Steel Tube Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>d (mm)</th>
<th>t (mm)</th>
<th>L/d</th>
<th>(A_s) (mm(^2))</th>
<th>(f_y) (MPa)</th>
<th>(f_{c'}) (MPa)</th>
<th>(A_s) (\frac{A_s}{A_{total}})</th>
<th>Links</th>
</tr>
</thead>
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<tr>
<td>C4-00</td>
<td>220</td>
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<td>2.54</td>
<td>3371</td>
<td>495</td>
<td>26.7</td>
<td>0.076</td>
<td>-</td>
</tr>
<tr>
<td>C4-10</td>
<td>220</td>
<td>5.25</td>
<td>2.54</td>
<td>3371</td>
<td>495</td>
<td>26.7</td>
<td>0.076</td>
<td>10</td>
</tr>
<tr>
<td>C4-20</td>
<td>220</td>
<td>5.25</td>
<td>2.54</td>
<td>3371</td>
<td>495</td>
<td>26.7</td>
<td>0.076</td>
<td>10</td>
</tr>
</tbody>
</table>
Table 4. Group - 4 Filled Rectangular Steel Tube Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>W (mm)</th>
<th>B (mm)</th>
<th>t (mm)</th>
<th>$A_t$</th>
<th>$f_y$ (MPa)</th>
<th>$f_c$ (MPa)</th>
<th>$A_s$</th>
<th>$A_{tot}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>REC-00</td>
<td>104.3</td>
<td>251.2</td>
<td>5.78</td>
<td>3976</td>
<td>367</td>
<td>23.4</td>
<td>0.152</td>
<td>-</td>
</tr>
<tr>
<td>REC-45</td>
<td>104.3</td>
<td>251.2</td>
<td>5.78</td>
<td>3976</td>
<td>367</td>
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<td>REC-35</td>
<td>104.3</td>
<td>251.2</td>
<td>5.78</td>
<td>3976</td>
<td>367</td>
<td>23.4</td>
<td>0.152</td>
<td>35</td>
</tr>
</tbody>
</table>

Ratio L / W = 5.77

U-Shaped Links:

Parallel legs of the U-shaped link are welded at their ends to opposite steel section walls alternately. The arrangement of U-shaped links requires the parallel legs to be welded to the four faces of the tube, following a clockwise order as shown in Fig.1. The three-legged link will confine the compressed concrete much greater than the standard rectangular links. Due to the tendency of separation between the concrete core and the steel tube, each pair of links welded to opposite faces of the steel tube, becomes part of a squeezing mechanism all along the height of the column, as opposite faces of the tube move further apart, as the axial compression load increases. The "table" of each U-Shaped link compresses the concrete in one direction, while the opposite "table" of the U-Shaped link compresses the concrete in the reverse direction. A minimum of two opposite links action is essential to confine the concrete over their spacing. Therefore, confinement in two perpendicular directions requires the use of four links, consequently, the number of links in the square column should be the multiple of four to achieve uniform confinement. In a rectangular section, to achieve uniform confinement, different spacing of U-links on adjacent sides must be used. The use of multiple cells could be considered, if the adjacent sides’ length ratio is large.

For rectangular and square tubular sections, the nominal U-link spacing used in confinement calculations is double the spacing between two consecutive U-links. For Circular tubular sections, it is dependent on the U-link arrangement.

Fig. 2 shows the arrangement of the links in typical sections and the calculations of concrete confinement.

Spacing of ordinary links in reinforced concrete columns is often reduced to prevent brittle failure and to increase the ductility of concrete. However, using ordinary links causes the formation of a natural plane of separation between the confined concrete core and the unconfined concrete core, and thus increasing the risk of a premature spalling of concrete cover, see Claeson (1998), Razvi and Saatcioglu (1999) and Johansson and Gylltoft (2002). It seems not reliable to depend on the natural bond strength to get full composite action, see Johansson and Gylltoft (2002).

There are three important reasons for using the U-shaped link in concrete filled steel tubes (CFT), first to control the confinement of the concrete, and second to play the role of a mechanical shear connector that also restrains and braces the steel tube walls, and finally, eliminates the spalling of the concrete cover. The testing of the specimens revealed the following:

1. In the initial stage of loading, Poisson’s ratio of concrete is lower than that of steel; therefore, the steel tube expands faster in the radial (lateral) direction than the concrete core, i.e., the steel does not restrain the concrete core. Provided the bond between the steel and concrete does not break, the initial circumferential steel hoop stresses are compressive and the concrete core is under lateral tension. At this stage and later, the concrete core and the steel tube are stressed tri-axially. The bond strength has no influence on structural behavior because there is no relative
movement between the concrete core and the steel tube.

Further increase in the axial load will cause the differential lateral strain to exceed the tensile strain of the concrete sustained by the bond. Once the bond-breakage takes place; the two legs of the link start to restrain the separation between the walls of the tube and the concrete core. The size of lateral separation is dependent on the stage of loading. The U-link and one of the walls of the tube will restrain the lateral expansion of compressed concrete. However, due to its larger lateral expansion, this wall itself will move away from its opposite wall, and each of these two walls will take with it the U-link welded to it. This will initiate a squeezing mechanism in their direction. The tensile forces exerted on the walls of the steel tube by the welded legs of the links will induce transverse compressive (hoop) stresses in the steel shape. Four U-links will confine compressed concrete in two directions.

The use of U-links imposes tri-axial stress systems on both steel tube and the concrete core, as shown in Fig. 3; where an element contains a welded U-link will bear the shown stresses and forces.

The U-shaped link will induce favorable confining stresses \( f \) on the concrete core as shown in fig. 3.

2. The ultimate load of the column was reached when the plastic buckling of the steel tube took place. Specimen C1-00 (with no links) exhibited the buckling of the steel walls on three distinct heights of the specimen, and the ultimate strain recorded was 0.39%. The other specimens, exhibited the buckling of the steel walls, but with the steel walls forming a spiral bulging form, and the strains recorded were 1.03% and 1.08% respectively. The Load deflection curve did not show a descending curve. The signs of failure occurred at values nearest to the ultimate load, when buckling of the steel walls started to show clearly.

3. The exact split of the load between the structural concrete inside the steel tube and the tube itself is dependent on the stage of loading. At the initial stage of loading, the elastic modular ratio that is based on the tangential moduli of the two materials controls the split. Later, the load-split is dependent on the instantaneous secant moduli ratio; this ratio is variable in accordance with the stress-strain curves of the two materials, assuming strain compatibility hypothesis. It is customary in design codes to focus on the load-split at ultimate load. The load split between the concrete core and the steel section is normally calculated at ultimate load. Considering the steel tube to be fully yielding, the concrete core will carry the rest of the axial force. Table 4 gives all the results in brief.

Fig. 4 shows the Load-Strain relationships for the specimens in group-1. The maximum-recorded strain was about 1% but still smaller than the actual strain attained, due to the stoppage of measurements. In Fig. 5, the same curves were plotted for group-2, and the actual strains were measured. These strains reached values of about 3%. Group-3 of circular sections of larger diameters, were tested and similar curves were plotted in Fig. 6. Fig. 7 shows the load-strain curves for rectangular specimens in group - 4. The U-links have an important role in sustaining the integrity of the cross-sections. The rectangular tube bulges in both directions, at strains of about 0.7% nearly at the same depth. The implication of this type of failure is extremely important. Although, the product of \( B \times e_u = W \times e_w \approx \text{constant} \), where \( e_u \) and \( e_w \), are the link spacing on the sides of the rectangle , the confinement in the longer side direction of the cross section is larger. This is because the participation of the walls in the longer dimension of the cross section is larger. None of the different specimens in groups 1, 2 and 3 that are provided by U links experienced a crushing type of failure in the structural body of the concrete. In fact, when the tested specimens were cut open by removing strips of the steel shell, the concrete surface was smooth and took the shape of the body of the steel shell, and showing no cracking at all.
U-links are welded in a clockwise manner. Link 1 on face A, then link 2 on face B, downwards to faces C and D in multiples of 4 links.

Fig. 1: Arrangement of U-Links
Specimen C2 - 20

\[ A_c = 19,265 \text{ mm}^2 \]
Average confinement of concrete along the specimen is obtained from considering 3 stirrups.

Area of concrete confined = Internal Diameter x 3 spacings
Spacing = 20 mm

Specimen Capacity

\[ f_y = 355 \text{ MPa} \]
\[ f_c = 23.7 \text{ MPa} \]
\[ A_s = 2,361 \text{ mm}^2 \]
\[ A_c = 19,265 \text{ mm}^2 \]

Un-confined Load:
\[ f_y \times A_s + 0.85 f_c \times A_c = 838 + 388 = 1226 \text{ kN} \]
Failure Load (confined) = 1947 kN

Confinement of Concrete

Links: diameter = 8 mm \( f_y = 335 \text{ MPa} \)
From figure: 3 legs confine width of concrete = internal diameter = 156.62 mm
\[ 3 \times 49 \times 335 = 60 \times 156.62 \times f \]
\[ f = 5.24 \text{ MPa} \]

Effect of lateral stress \( f \)

Failure load - steel shell load =
\[ 1947 - 838 = 1109 \text{ kN} \]
Max. Stress = \[ 1109 \times 1000 / 19,265 \]
\[ = 57.57 \text{ MPa} \]
\[ 57.57 = 0.85 f_c + \lambda f \]
\[ \lambda = 7.14 \]

welded ends of a U-link

"Table" ends of a U-link

Fig. 2 Typical Arrangement of U-Links in HSS Sections and Confinement Analysis of concrete
Fig. 3. Tri-axial stress systems on both steel and concrete elements in vicinity of welded link.
Fig. 4: Axial Load - Average Axial Strain Group C1

Fig. 5: Axial Load - Average Axial Strain Group C2
Fig. 6: Axial Load - Average Axial Strain
Group C3

Axial Strain %

Fig. 7: Axial Load - Average Axial Strain
Group C4 (Rectangular cross-sections)

Average Strain %
II. Built-up Tubes:

The practical construction of tubular columns with fixed links inside by welding, can be implemented for large dimensions, where the welders can access the locations inside the tubes. Fig. 8 shows the marking used to locate the welding points of the U-links. The procedure of construction is explained by the figures 9 to 11. Figures 12 and 13 emphasize the importance of sound structural welding. Figures 14 to 17 show the testing of four specimens at Newmark Laboratories at Illinois University. The welding of links inside smaller dimensions of tubes cannot be implemented for practical considerations. The construction of built-up tubes from steel plates enables the designers to build all desired dimensions and thicknesses, in addition to have the links welded to the folded plates prior to welding them longitudinally to end up with the desired tube shape.

Two groups of built-up tubes were tested:

**Group A:** Table (6) shows the dimensions of the specimens resulted from welding two angles formed by folding 6 and 8 mm plates. L is the finished height after machining the two ends of the specimen; the cross section is a rectangle of dimensions D and W. The designation L6 35 14 w indicates that two sides of the cross section are the two legs of an angle that is formed by folding a rectangular plate explained in figures 8 to 11. The 6 refers to the thickness of the plate, 35 is the spacing of the links, and 14 is the diameter of the link, w indicates that the two angles are structurally welded together along their height. Two groups: A1 and A2 were tested; the test results are shown in figures 12 and 13. Limited confinement was achieved. The failure of all specimens in this group was due to the failure of the welds, especially the two longitudinal welds that should sustain the hoop tension forces generated at larger loads. In-group A1, the welds were inadequate and a premature failure took place, giving no room for concrete confinement. In-group A2, the longitudinal welds, were under reinforced, and barely carried the forces generated in the steel shell at their particular location. The outcome of less stiff longitudinal strips resulted in limited concrete confinement. Engineering measurements were limited to recording the axial load plus the manual measurements of the axial strains. Tests of group A were conducted at the structural laboratory of the Jordan University of Science and Technology.

**Group B:** Table (7) shows the dimensions of the four built-up tubular sections that were constructed at Newmark Structural Engineering Laboratory at the University of Illinois at Urbana-Champaign; the lab is equipped with the necessary measuring devices so that all sorts of engineering measurements are possible. All components of the tubular cross section were strain-gaged: the strains of the steel shell, the strains in the U-links inside the specimens and the strains of the concrete inside the tubular cross section. All of these measurements require electronic power supply during the testing. Moreover, dial gages, which do not require an electric power supply, were also used to measure axial deflection. All measurements were recorded continuously and simultaneously during the testing. High definition camera was dedicated and focused on the specimens while the load was progressing.

The size of the holes drilled in the rectangular plate in the positions will vary in size and should be larger than the size of the diameter of the link by a tolerance accepted in welding, normally of the order of 1.5 mm. The distance to the edge of each hole to the end of the bent plate should also account for the drilled hole, not to interfere with the bent corner of the angle.

Figures 14 to 17 show the relationship between the applied axial loads versus:

1. The average strains attained in the concrete inside the tube (embedment strain gage).
2. The average strain attained in the U-links (Average rebar).
3. The average strain attained in the axial direction of the steel tube (Average axial).
4. The average strain attained in the transversal direction of the steel tube (Average transverse).
5. The average overall strain attained in the steel tube (Average overall).

Two links are needed to squeeze the concrete between four pitches, and one link is used to confine the concrete between two pitches. Thus, the confining stress is:

\[
\sigma_i = \frac{A_i \times f}{B \times e}
\]

where:

- \( A_i \) = Area of one leg
- \( e \) = pitch
Applying this equation at ultimate, for the three sizes of the used links:

\[
\sigma_3 = \frac{A \times f_y}{B \times e} = \frac{\pi \times \left(\frac{3}{8}\right)^2 / 4 \times 100}{8 \times 1.5} = 0.92 \text{ ksi (6.34 MPa)} \quad \cdots \quad d = 3/8 \text{ in. (9.53 mm)}
\]

\[
\sigma_3 = \frac{A \times f_y}{B \times e} = \frac{\pi \times \left(\frac{1}{2}\right)^2 / 4 \times 100}{8 \times 1.5} = 1.64 \text{ ksi (11.31 MPa)} \quad \cdots \quad d = 1/2 \text{ in. (12.7 mm)}
\]

\[
\sigma_3 = \frac{A \times f_y}{B \times e} = \frac{\pi \times \left(\frac{5}{8}\right)^2 / 4 \times 100}{8 \times 1.5} = 2.56 \text{ ksi (17.65 MPa)} \quad \cdots \quad d = 5/8 \text{ in. (15.9 mm)}
\]

The level of confinement is subject to control. The above equation shows the effect of the diameter of the link, which increases the confinement proportional to its area. The larger the grade of the steel of the links, the larger is the confinement. The increase of the section dimension reduces the confinement of concrete; however, larger dimensions may require the subdivision of the cross section to more than one cell, by introducing partitioning plates. If the partitioning plates are used, it is possible to control the dimension \( B \) in the equation.

The role of the vertical spacing of the links in confining the concrete inside the tube, can be increased by reducing its dimension; however, this dimension should not hinder the installment of the links. A minimum of \( e = 30 \text{ mm} \) should set the lower limit for practical purposes. The largest extra force carried by concrete due to confinement equals 1582 kips (7037 kN) minus 1222 kips (5435 kN) which equals 360 kips (1602 kN). These results demonstrate the effectiveness of the system.

Table 5. Summary of the results.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Square Specimens Group-1</th>
<th>Circular Specimens Group-2</th>
<th>Circular Specimens Group-3</th>
<th>Rectangular Specimens Group-4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C1-00</td>
<td>C1-40</td>
<td>C1-30</td>
<td>C2-00</td>
</tr>
<tr>
<td>Spacing of U-Links (mm)</td>
<td>-</td>
<td>40</td>
<td>30</td>
<td>-</td>
</tr>
<tr>
<td>Ultimate Load (kN)</td>
<td>1104</td>
<td>1568</td>
<td>1620</td>
<td>1230</td>
</tr>
<tr>
<td>( f_{\text{max}} / f'_c )</td>
<td>0.85</td>
<td>1.73</td>
<td>1.83</td>
<td>0.86</td>
</tr>
<tr>
<td>( f_{\text{max}} / 0.85 f'_c )</td>
<td>1.0</td>
<td>2.03</td>
<td>2.15</td>
<td>1.0</td>
</tr>
</tbody>
</table>

\( f'_c = 30 \text{ MPa} \)
\( A'_c = 17540 \text{ mm}^2 \)
\( f'_{c, \text{(shell)}} = 320 \text{ MPa} \)
Dia. of U-link = 8 mm
\( f'_{c, \text{(link)}} = 335 \text{ MPa} \)

\( f'_c = 23.7 \text{ MPa} \)
\( A'_c = 19265 \text{ mm}^2 \)
\( f'_{c, \text{(shell)}} = 355 \text{ MPa} \)
Dia. of U-link = 8 mm
\( f'_{c, \text{(link)}} = 335 \text{ MPa} \)

\( f'_c = 27.1 \text{ MPa} \)
\( A'_c = 37840 \text{ mm}^2 \)
\( f'_{c, \text{(shell)}} = 430 \text{ MPa} \)
Dia. of U-link = 10 mm
\( f'_{c, \text{(link)}} = 480 \text{ MPa} \)

\( f'_c = 23.4 \text{ MPa} \)
\( A'_c = 22220 \text{ mm}^2 \)
\( f'_{c, \text{(shell)}} = 367 \text{ MPa} \)
Dia. of U-link = 10 mm
\( f'_{c, \text{(link)}} = 480 \text{ MPa} \)
### Table 6. Group A- square tubular columns (using welded angled plates)

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>Specimen Dimensions</th>
<th>Links</th>
<th>Specimen &amp; links weight</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>L cm</td>
<td>D cm</td>
<td>W cm</td>
<td>T mm</td>
</tr>
<tr>
<td>L6 - - w</td>
<td>51.7</td>
<td>14.6</td>
<td>14.7</td>
<td>6</td>
</tr>
<tr>
<td>L6 35 14 w</td>
<td>51.7</td>
<td>14.3</td>
<td>14.4</td>
<td>6</td>
</tr>
<tr>
<td>L8 - - w</td>
<td>51.5</td>
<td>14.6</td>
<td>14.6</td>
<td>8</td>
</tr>
<tr>
<td>L8 35 14w</td>
<td>51</td>
<td>14.3</td>
<td>14.5</td>
<td>8</td>
</tr>
</tbody>
</table>

### Table 7. Group B- square tubular columns (using welded angled plates)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>B (in)</th>
<th>t (in)</th>
<th>L/B</th>
<th>$A_s$ (in$^2$)</th>
<th>$f_y$ (ksi)</th>
<th>$f_s$ (ksi)</th>
<th>$A_s/A_{con}$</th>
<th>Nominal Spacing (in)</th>
<th>dia. (in)</th>
<th>$f_{y,calc}$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Control)</td>
<td>8</td>
<td>0.5</td>
<td>2.44</td>
<td>13.5</td>
<td>46</td>
<td>6.64</td>
<td>0.211</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(5/8 &quot;rebar&quot;)</td>
<td>8</td>
<td>0.5</td>
<td>2.44</td>
<td>13.5</td>
<td>46</td>
<td>6.64</td>
<td>0.211</td>
<td></td>
<td>1.5</td>
<td>5/8 100</td>
</tr>
<tr>
<td>(3/8 &quot;rebar&quot;)</td>
<td>8</td>
<td>0.5</td>
<td>2.44</td>
<td>13.5</td>
<td>46</td>
<td>7.84</td>
<td>0.211</td>
<td></td>
<td>1.5</td>
<td>3/8 100</td>
</tr>
<tr>
<td>(1/2&quot;rebar&quot;)</td>
<td>8</td>
<td>0.5</td>
<td>2.44</td>
<td>13.5</td>
<td>46</td>
<td>7.84</td>
<td>0.211</td>
<td></td>
<td>1.5</td>
<td>1/2 100</td>
</tr>
</tbody>
</table>

### ANALYSIS AND RECOMMENDATIONS

Figure 18 shows the simplified approach that used to explain the role of the connecting U-link. By ignoring the bond between the steel shell and the concrete core, the equal axial strain in both materials will translate itself laterally by unequal amounts. The Poisson’s ratio of steel equals 0.3, while the concrete Poisson’s ratio equals 0.15. Both materials will have this ratio increased to 0.5 once plasticization commences. The crushing compression strain of the concrete enhances by the lateral confinement, and therefore, the point of bifurcation that decides the plasticization point will be numerically larger than the value of 0.003 given by the ACI Code. S is the bifurcation point of the steel tube, while C is the bifurcation point for the concrete core. In fact, beyond the value of $\varepsilon_{cup}$, the plastic compression strain of concrete, the concrete will still strain axially, but begins to expand laterally by a ratio of 0.5. Fig. 18 shows that $\varepsilon_{cup}$ is larger than $\varepsilon_{cup}$, its value is dependent on the confining conditions. The compression axial strain of the concrete beyond $\varepsilon_{cup}$ continues to increase without a real gain in the carrying capacity of the specimen. However, during this stage of loading the specimen enters a phase of “plasticity”, the sustained load becomes coupled with an increasing strain, reflecting the degradation of the secant stiffness of the cross section. Contrary to the ordinary composite columns, the collapse of the specimens occurs due to the plastic buckling of the steel shell, while the concrete core remains confined. Point C defines the state of maximum confinement; and beyond point C, the confinement is almost steady and constant.

In some cases, the strain can cause yielding to the U-link. The values of $\varepsilon_{cup}$ attained by the confined concrete range from 0.003 for poor densities of U-links or when the welds used to keep the section integrated, fail, to 0.03 or more for good densities of U-links. The steel stress $f_s$ in the U-links begins to pick up values as the load grows up. For a rectangular or square tube, the confining stress $f$ can be calculated from $2 \times A_s \times f_s = B \times (2S) \times f$, which gives:

$$f = \frac{A_s \times f_s}{B \times S}.$$  

A similar formula is derived for a circular tube, where three legs participate in confining the concrete core:
Confining stress value $f$ in the tests range between (10-40) % of $f_c$, high values of $f$ in excess of 15% $f_c$ are easily obtained by proper detailing. Large dimensions of the cross-section, requires smaller spacing of the U-links to achieve appreciable confinement. The value of tensile stress in the U links $f$, ranges from (100 - $f_c$) MPa and is dependent on the properties of the cross section and the spacing of the links. Production of connected composite columns could be produced in workshops or steel factories, by welding the U-links to plates before bending them into the desired shape and then inducing seam welding that closes the section.

CONCLUSION

The role of the U-shaped links has tremendous effect on the performance of the concrete-filled steel tubes:

1. The strength capacity of the concrete core appreciates with confinement. The average ultimate stress for confined concrete doubles for properly designed specimens.
2. The ultimate strain rose from 0.3% linked to unconfined concrete to more than (6-10) times this value for properly confined concrete; such values are larger than ten times the yield strain of steel. This brings the ultimate strains to the levels of strain hardening of the steel.
3. Properly confined specimens do fail by the plastic buckling of the steel shell, and not by the crushing of the concrete core.
4. The compression crushing strain of concrete $\epsilon_{cu}$ transforms to $\epsilon_{rup}$ to reflect the maximum compression strain of concrete at which compressed concrete “plastic” behavior commences.
5. Further research needs to explore more types of CFT columns subjected to eccentric loading, and explore the multi-cell construction of such columns.

Acknowledgements:

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The links are inserted into their position, welded to face 1 first, then at a higher level (pitch height), the link is welded to face 2, the procedure follows a helical climbing up pattern.

Fig. 10: U links are welded to Bent Plates at the prescribed locations.
After welding the links to the pertinent angle plates, the two angles are welded through lines BC and AD to form a tube.

**Fig. 11:** Tubular cross section with U Links

**Link Total Length = 422 mm**

\[
L = 2 \times 144 + 96 + 2 \times 18.85 = 421.7 \text{ mm}
\]
**Fig. 12:** Limited Axial Strain due to the Failure of welds.

**Fig. 13:** Inadequate welds forced limited concrete confinement.
Figure 4.14: Load - Axial Strain, Steel, Tube, Axial Strain of Concrete (Imbedment), Transverse Strain, Overall Strain.
Fig. 18: Simplified model of the loading strain in the link.
REFERENCES


ACI 318M-08 –Building Code Requirements for Structural Concrete. 2008.


الأعمدة الحركية المؤلفة من مقاطع فولاذية معبأة بالخرسانة

عبدالقادر النجمي

ملخص

يطغت البحث إلى دراسة نوع جديد من الأعمدة المركبة المؤلف من مقاطع فولاذية معبأة بالخرسانة. يتم استخدام نوع خاص من الكانات لربط جسم المقطع الأنبوبي الفولاذي مع الخرسانة داخله. وتبين من التجارب المخبرية أن طريقة الربط المستخدمة أدت إلى وصول الحصر للخرسانة في هذا النوع من الأعمدة لمستويات عالية جدا، لم يتم تسجيل مثل لها من قبل. فقد فاقت مستويات الحصر التي تم الحصول عليها قيم الخصر للخرسانة داخل الأعمدة الحلزونية بكثير عند الحمل الأقصي، وتجاوزت مقاومة الخرسانة بأكثر من 50%. وحققت هذه الأعمدة عند الحمل الأقصي انفعالات تجاوزت 2% وهذه الالتباسات العالية بالتأكيد أعلى من الأفعالات المرافقة لجهد الخضوع للفولاذ، وتساري (6-8) أضعاف انفعال الضغط الأقصي للخرسانة (0.3%). وتبين كذلك أن أداء المركب لهذه الأعمدة يمنع حصول الانبعاج للقشرة الفولاذية في المدى المرن للأحمال، ويحافظ على جسم هذه الأعمدة في المدى اللدن. عند الحمل الأقصي لا يحدث انهيار للخرسانة بتحطمها بالضغط، بل يكون الانهيار من خلال الانبعاج اللدن للمقطع الفولاذي.

الكلمات الدالة: الخرسانة، الفولاذ، أعمدة حركية، مقاطع.