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Abstract
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AXIAL COMPRESSION TESTS ON HYBRID FRP-STEEL-CONCRETE COLUMNS WITH HIGH-STRENGTH STEEL PLATES

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ABSTRACT
The applications of high-strength steel (HSS) products to civil engineering structures have been limited by elastic local buckling, by the perceived lack of ductility, and by the perceived difficulties of welding such steels. Against this background, the authors have recently proposed a new hybrid column (i.e. High Strength Steel Plate-Concrete Filled FRP Tube or HSSP-CFFT) consisting of an outer FRP tube, a concrete in-fill and a number of encased high-strength steel plates that are connected to each other by bolted angle brackets at discrete elevations. The new column offers an ideal opportunity for the use of HSS plates in construction, with their high yield stresses being fully utilized and without welding (and therefore without welding residual stresses). In this paper, the rationale for the new column form together with its expected advantages is first explained. Results from a series of axial compression tests are then presented to confirm some of the expected advantages. The results demonstrated that the concrete in the tested specimens was very effectively confined, and that buckling of the steel plates was prevented by the encasing concrete up to and beyond the rupture of the FRP tubes, leading to full structural utilization of the construction materials and very ductile column responses.

KEYWORDS
High-strength steel, FRP, concrete, column, confinement, buckling.

INTRODUCTION
The use of high-strength materials is often preferred when reductions in structural self-weight and/or size are important. A significant number of studies (e.g. Rasmussen and Hancock 1995; Ban et al. 2013) have thus been conducted on high-strength steel (HSS) products, but their applications in civil engineering structures are rather limited at present. HSS typically refers to steel with a yield stress not lower than 450 MPa, although the HSS grades available vary in different countries (Li et al. 2013). A major limiting factor for HSS is that its high yield stress cannot be fully utilized due to buckling under compression (e.g. in columns). Efforts to prevent local buckling of structural columns include filling a steel tube with concrete (i.e. concrete-filled tube or CFT) or encasing a steel section (e.g. I-section) in concrete (Uy 2001; Pocock 2006). However, these techniques have had limited success for HSS sections: in the former columns, local outward buckling of the steel tube still occurs; in the latter columns, the concrete typically crushes under compression at an axial strain between 0.2% to 0.3%, which may be much lower than the yield strain of HSS. As a result, in both forms of columns, the HSS component can still buckle well before the yield strain is reached. Even when buckling occurs close to the yield strain, the subsequent response of the column is non-ductile, which is highly undesirable for structures designed for seismic resistance. Furthermore, fabricated steel sections require welding of the plate components, which adds to the fabrication cost especially for high strength steel.

Against the above background, the authors recently proposed a novel solution to the problem of local buckling by fully encasing high strength steel plates in concrete that is cast in an FRP confining tube. The resulting structural member, referred to hereafter as concrete-filled FRP tube with HSS plates (or HSSP-CFFT), consists of an outer FRP tube, a concrete in-fill and a number of encased high-strength steel plates (Figure 1). In the novel column, the FRP tube offers mechanical resistances primarily in the hoop direction to confine the concrete and to enhance the shear resistance of the column, while the steel plates act as the main longitudinal reinforcement. One novel aspect of the column is that no welding is involved to fabricate the high strength steel sections, thus reducing the fabrication and transportation costs. Importantly, it obviates the perceived difficulties with welding high strength steels among the structural engineering profession. The individual steel plates can be connected to each other by angle brackets at discrete locations to facilitate their placement inside the FRP tube prior to casting concrete. By doing so, the angle brackets also serve as (1) shear connectors between the steel plates and concrete for improved composite action; and (2) restraints to the individual plates leading to reduced buckling lengths. The other novel aspect of the column is that it allows the configurations of the steel plates to be optimized for particular applications, which may not be in the shape of traditional sections such as I-section.
The novel column may be constructed in-situ or precast, with the FRP tube acting as the stay-in-place form. The section form shown in Figure 1 consists of a circular outer tube and four steel plates, but many different combinations are possible (e.g. the outer tube can be square or rectangular, while more steel plates can be used to form a polygonal shape).

![Figure 1 Typical cross-section of HSSP-CFFT](image)

The main advantages of HSSP-CFFTs include: (1) excellent ductility, as the concrete is well confined by the FRP tube and local buckling of the steel plates are constrained by the concrete; (2) excellent corrosion resistance, as the FRP tube is highly resistant to corrosion while the steel plates are protected by the FRP tube and the concrete; and (3) ease for construction, as the FRP tube act as a permanent form for casting concrete, and the welding of high-strength steel plates is avoided in the column. In addition, the full exploitation of the high yield stress of HSSP leads to significantly lighter columns that are cheaper to construct, as well as increased floor space in building construction.

As a first step in developing the new concept into a practical construction technology that can be routinely designed by structural engineers, a series of axial compression tests on HSSP-CFFTs were conducted. In parallel, tests were conducted on corresponding CFFTs without steel plates for comparison. This paper presents results from the axial compression tests which demonstrate some of the expected advantages of the new columns.

**EXPERIMENTAL PROGRAM**

**Test Specimens**

A total of 10 specimens were constructed and tested, comprising 8 HSSP-CFFT specimens and 2 CFFT specimens without steel plates. All specimens had a nominal diameter of 203 mm (i.e. inner diameter of the FRP tube) and a height of 600 mm. For the HSSP-CFFT specimens, two configurations of steel plates were adopted, one with a single plate placed along a centre line of the cross-section while the other consisting of four plates connected together by bolted angle brackets to form a square (referred to as “square plate configuration” hereafter). The widths of the steel plates were so designed that the concrete cover had a minimum thickness of around 20 mm. As a result, the steel plates used in the single-plate specimens had a width of 160 mm, while those used in the square-plate specimens had a width of 85 mm or 95 mm, depending on the thickness of the plates. The steel plates had a yield stress of around 450 MPa or 290 MPa. Although the steel plates with a yield stress of 290 MPa were not HSS plates, they were used in the present study for comparison, and all the specimens are referred to herein as HSSP-CFFT specimens for simplicity. The other main test variables included the thickness of the steel plates and that of the FRP tube. The details of all the specimens are summarized in Table 1. For ease of reference, each specimen is given a name (see Table 1), which starts with a letter “S” to represent “specimen”, followed by a number (0, 1 or 4) to represent the number of steel plates in the specimen; this is then followed by a letter (from “A” to “C”) to represent the type of steel plates (for HSSP-CFFT specimens only). The second number in the specimen name defines the thickness of the FRP tube.

**Material Properties**

Two types of prefabricated glass FRP (GFRP) tubes were used in the present study. These GFRP tubes were all formed by a filament-winding process with a nominal fiber volume fraction of 59% and fiber winding angles of
±75 degrees to the longitudinal axis of the tube. The only difference between the two types of GFRP tubes was in their nominal thicknesses, which were 1.5 mm and 3 mm, respectively. According to the manufacturer data, the tubes had elastic moduli equal to 33 GPa in the hoop direction and 12 GPa in the longitudinal direction.

The 10 HSSP-CFFT and CFFT specimens were divided into two testing groups (see Table 1) and were tested at two different times. The specimens of Group 1 were tested after 32-34 days of curing while the others were tested two weeks later. As a result, the compressive strengths of the concretes in the two testing groups were slightly different. Plain concrete cylinders of two dimensions, namely, 150 mm x 300 mm and 100 mm x 200 mm, were tested to determine the compressive strength of concrete. The 150 mm cylinder tests showed that the concrete in Group 1 had a compressive strength of 33.1 MPa while that in Group 2 had a compressive strength of 35.9 MPa. The compressive strengths obtained from the 100 mm cylinder tests were 44.7 MPa and 48.0 MPa for Groups 1 and 2, respectively.

Tensile tests on three steel coupons were conducted for each type of steel plates. The measured yield stresses of Plates A, B and C were found to be 290 MPa, 450 MPa and 455 MPa, respectively.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Configuration of steel plates</th>
<th>Steel Plates</th>
<th>Thickness of FRP tubes (mm)</th>
<th>Testing group</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Yield stress (MPa)</td>
<td>Thickness (mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>----------</td>
<td>-------------------</td>
<td>----------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S0-3</td>
<td>N/A</td>
<td>3</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>S0-1</td>
<td>1.5</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>S1A-3</td>
<td>Single</td>
<td>A</td>
<td>290</td>
<td>4.95</td>
</tr>
<tr>
<td>S4A-3</td>
<td>Square</td>
<td>A</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>S4A-1</td>
<td>Square</td>
<td>A</td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>S1B-3</td>
<td>Single</td>
<td>B</td>
<td>450</td>
<td>5</td>
</tr>
<tr>
<td>S4B-3</td>
<td>Square</td>
<td>B</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>S4B-1</td>
<td>Square</td>
<td></td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>S1C-3</td>
<td>Single</td>
<td>C</td>
<td>455</td>
<td>10</td>
</tr>
<tr>
<td>S4C-3</td>
<td>Square</td>
<td></td>
<td>3</td>
<td>1</td>
</tr>
</tbody>
</table>

**Preparation of Specimens**

The preparation process of the HSSP-CFFT specimens included the following steps: (1) preparation of steel plates, which involved cutting of the steel plates to the desired dimensions, attaching strain gauges on the steel plates and bolting the steel plates to form the square plate configurations; (2) fabrication of the mould, which consisted of an outer FRP tube with a centered steel plate configuration fixed to a wooden frame with a waterproof base plate; (3) casting the concrete; (4) strengthening of local regions which included the two regions each of a 75 mm height near the column ends and the regions near the holes drilled through the FRP tubes for passing the electric wires of the strain gauges on the steel plates. A similar preparation process was adopted for the CFFT specimens except that Step (1) was not needed and Steps (2) and (4) were simpler because of the absence of steel plates.

**Test Set-Up and Instrumentation**

For each specimen, four strain gauges with a gauge length of 20 mm or 30 mm were installed at the mid-height of the FRP tube to measure the hoop strains. For each steel plate in the HSSP-CFFT specimens, two axial strain gauges with a gauge length of 10 mm were attached on the two faces respectively at the mid-height. In addition, two linear variable displacement transducers (LVDTs) were used to measure the total axial shortening of each specimen. All compression tests were carried out using a 500 ton Denison compression testing machine with a displacement control rate of 0.5 mm/min. All test data, including the strains, loads, and displacements, were recorded simultaneously by a data logger.
TEST RESULTS AND DISCUSSIONS

General Behaviour

All the specimens displayed continuous load-shortening behavior with a monotonically increasing load until the ultimate failure due to rupture of the FRP tube under hoop tension, except Specimen S4C-3 for which the test was terminated when the load approached the capacity of the testing machine. The rupture of FRP tube generally initiated from a hole which was drilled for passing the electric wires, although that region was locally strengthened. This means that the hoop rupture strain of FRP recorded in these tests may be well below that could be reached in a real column with an intact FRP tube, but the effect of the small holes on the overall behavior of the column before the ultimate failure is believed to be minor.

Axial Load-Shortening Behaviour

The key test results of all the ten specimens are summarized in Table 2. In this table, \( P_c \) is the ultimate load obtained in the test, \( P_s \) is equal to the squash load of the steel plates, and \( P_{co} \) is equal to the unconfined concrete strength times the net cross-section area of concrete.

Table 2. Key test results of specimens under concentric compression

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ultimate load ( P_c ) (kN)</th>
<th>Squash load of steel plates ( P_s ) (kN)</th>
<th>Unconfined strength of concrete section ( P_{co} ) (kN)</th>
<th>( \frac{P_s}{P_c} )</th>
<th>Hoop rupture strain ( \varepsilon_{h,rup} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>S0-3</td>
<td>3394.0</td>
<td>0</td>
<td>1165.2</td>
<td>2.91</td>
<td>0.0176</td>
</tr>
<tr>
<td>S0-1</td>
<td>2250.2</td>
<td>0</td>
<td>1165.2</td>
<td>1.93</td>
<td>0.0164</td>
</tr>
<tr>
<td>S1A-3</td>
<td>2981.8</td>
<td>229.7</td>
<td>1136.6</td>
<td>2.18</td>
<td>0.0120</td>
</tr>
<tr>
<td>S4A-3</td>
<td>3774.3</td>
<td>545.5</td>
<td>1097.4</td>
<td>2.29</td>
<td>0.0167</td>
</tr>
<tr>
<td>S4A-1</td>
<td>3140.8</td>
<td>545.5</td>
<td>1097.4</td>
<td>1.91</td>
<td>0.0233</td>
</tr>
<tr>
<td>S1B-3</td>
<td>3030.8</td>
<td>360</td>
<td>1136.4</td>
<td>2.02</td>
<td>0.0153</td>
</tr>
<tr>
<td>S4B-3</td>
<td>4191.1</td>
<td>855</td>
<td>1096.8</td>
<td>2.15</td>
<td>0.0170</td>
</tr>
<tr>
<td>S4B-1</td>
<td>3486.5</td>
<td>855</td>
<td>1096.8</td>
<td>1.79</td>
<td>0.0202</td>
</tr>
<tr>
<td>S1C-3</td>
<td>3700.1</td>
<td>720</td>
<td>1107.6</td>
<td>2.02</td>
<td>0.0135</td>
</tr>
<tr>
<td>S4C-3</td>
<td>4646.1</td>
<td>1530</td>
<td>1042.8</td>
<td>1.81</td>
<td>0.0158</td>
</tr>
</tbody>
</table>

Figure 2. Typical axial load-shortening curves

Typical axial load-shortening curves of the specimens with a 1.5 mm FRP tube are shown in Figure 2, where the axial shortenings were averaged from the two LVDT readings. It is evident that all the specimens had a bilinear axial load-shortening curve with two ascending branches. Because steel has a larger elastic modulus than
concrete, the two HSSP-CFFT specimens had a larger initial stiffness than their CFFT counterparts (Figure 2). The second linear branches of the curves of the three specimens, however, were approximately parallel to each other (i.e. had a similar slope) (Figure 2). Further examination of the curves reveals that for all the three specimens in Figure 2, the transition point between the two branches of the curve happened at a load which was approximately equal to \( P_{co} + P_s \) of the specimen. Considering the fact that the steel plates had yielded at the beginning of the second branch, as also evidenced by the readings of strain gauges attached on the steel plates, it can be deduced that the load increase in the second branch came mainly from the confinement effect of the FRP tube to the concrete, except that some load increase near the end of test may have been caused by the strain hardening of the steel plates. This also explains the approximately parallel second linear branches of the three curves in Figure 2, where the differences in the load of the three linear branches at an arbitrary shortening are approximately equal to the differences in the \( P_{co} + P_s \) values. It is also obvious that the HSSP-CFFTs reached ultimate loads that are significantly higher than the simple sum of the squash loads of the steel plates and the unconfined strength of the concrete (Table 2). It may be noted that the direct contribution of FRP tubes to the ultimate loads was very small.

Buckling of Steel Plates

In HSSP-CFFT specimens, the steel plates were restrained by the encasing concrete and the FRP tube. As a result, their propensity to buckle was reduced. The steel plates of all tested HSSP-CFFT specimens were taken out for examination. Figure 3a shows the plates taken out from the specimens with a single plate while Figure 3b shows those taken out from the specimens with a square plate configuration. It is evident from Figure 3a that the plates remained straight after test without any visible buckling deformation. However, for the plates which formed a square configuration, Figure 3b shows that most of them, especially those in specimens with a thin FRP tube, experienced significant buckling deformation. The only exception was the square plate configuration taken out from Specimen S4C-3 (rightmost specimen in Figure 3b). To further examine this issue, readings from the strain gauges attached on the steel plates in Specimen S4A-3 are shown in Figure 4 (the strain gauge readings from other specimens with a square plate configuration were similar). In this paper, a tensile strain is positive and a compressive strain is negative unless otherwise specified. Figure 4 shows that the four pairs of strain gauges recorded very similar strains at the beginning of test until a strain value which was approximately equal to the yield strain of the steel; after that the strain readings appeared somewhat erratic, which might be due to uneven yielding of the steel plates. Nevertheless, after a short period, all the eight strain gauges started to increase monotonically again at similar rates until the rupture of FRP tubes. There was no evidence that the two strain gauges of any pair (i.e. the two strain gauges attached on the two faces of a plate) displayed noticeable strain gradient over the thickness of a steel plate, suggesting that buckling deformation near the mid-height region was unlikely to occur before the rupture of the FRP tube. The fact that the strains of all the plates kept increasing (i.e. no unloading) suggests that buckling away from the mid-height region was also unlikely to occur before the FRP rupture. Considering also the fact that the four steel plates in Specimen S4C-3, whose test was terminated before the FRP rupture, did not show any visible buckling deformation, it may be reasonable to conclude that the buckling of steel plates shown in Figures 3b for specimens with a square plate configuration occurred only after the rupture of FRP tube, when the plates lost their buckling restraint. This conclusion is also
supported by the axial load-shortening curves shown in Figure 2, which suggests that the contribution of steel plates did not decrease before the FRP tubes ruptured.

CONCLUSIONS

This paper has presented the details of a newly proposed hybrid FRP-concrete-steel tubular column. The new column consists of an outer FRP tube, a concrete in-fill and a number of encased high-strength steel plates that are connected to each other by bolted angle brackets at discrete elevations. The new column, termed HSSP-CFFT (High Strength Steel Plate-Concrete Filled FRP Tube) in this paper, offers an ideal opportunity for the use of high-strength steel plates in construction, with the high yield stress of these plates being fully utilized and the perceived difficulties of welding high strength steels being obviated. No allowance needs be made for welding residual stresses, leading to simpler structural design procedures. The HSSP-CFFT column thus has many advantages compared to existing designs including its excellent corrosion resistance and ample ductility. This paper has also presented results from axial compression tests. The results demonstrated that the concrete in the tested specimens was very effectively confined, and that buckling of the steel plates with yield stresses up to 455 MPa was prevented by the encasing concrete up to and beyond the rupture of the FRP tubes, leading to full structural utilization of the construction materials and very ductile column responses.

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REFERENCES


