Performance of Steel Tube Strengthened Recycled Concrete Exposure to High Temperature and Reinforced by CFRP

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ABSTRACT: This paper presents the results of experimental investigations on steel tube strengthened recycled concrete (STSRC) exposed to high temperatures and reinforced by CFRP. The failure mode, bearing capacity and deformability of the columns were investigated to analyze the effects of CFRP, temperatures (300°C, 500°C and 700°C), and recycled coarse aggregate (rCa) replacement ratios (0%, 50% and 100%) on mechanical performance. The results indicate that the higher exposed temperatures, the more cracks occurred and the lower ultimate load carrying capacities for the specimens, and the failure mode and load capacity could be increased greatly by external strengthened with CFRP sheets. However, the higher rCa replacement ratio could induced a smaller cracks and higher load capacity, and the specimens with 50% rCa ratio gave the highest load capacity when exposed to high temperature. In addition, the ABAQUS software was employed to conduct finite element analysis (FEA) of STSRC reinforced, and a comparison of results calculated using the proposed model exhibits good agreement with an experimental results.

1. INTRODUCTION

It was well known that steel reinforced concrete is one of the most popular composite materials currently used in construction over the last 40 years [1,2], due to its excellent static and fatigue resistance properties [3–6]. Moreover, in the past, there have been numerous research studies which have investigated recycled aggregate concrete structures such as columns or beams when subjected to compression bending and other stress phenomena [7–10]. Based on previous research, the composite structure of steel tube strengthened recycled concrete (STSRC) was gradually developed. The bearing capacity and failure mode of STSRC when subjected to static of fatigue loads has attracted much recent research attention [11–13].

It is generally accepted that recycled concrete structures may be exposed to high temperatures. Much previous research has reported that concrete loses approximately 25–80% of its strength when exposure to temperatures reaching 700°C. However, recycled concrete exhibits better compressive strength, flexural properties, and elasticity modulus than normal concrete in a certain high temperature range [14–17]. Based on studies of RAC properties at elevated temperatures, RAC maintains greater proportion of its strength than NAC since the surface cracks on recycled coarse aggregate can release energy during water evaporation at high temperatures.

Generally, RAC structures exposure to high temperatures are able to be repaired by FRP due to the only partial damage to its form [18,19]. Yaque and Bailey [20,21] conducted tests on post-heated reinforced concrete repaired by FRP under axial compression. It was reported that due to the superior tensile properties of FRP, the axial capacity of post-heated reinforced concrete columns was significantly improved.

Based on previous research [22–24], a new composite structure of steel tube strengthened recycled concrete (STSRC) was proposed in previous research by the authors. In practice, STSRC may behave differently under static or fatigue loads after exposure to high temperatures (300°C, 500°C or 700°C), in terms of the failure mode, bearing capacity stiffness and fatigue properties. The columns exposed to high temperatures and repaired by CFRP demonstrated good resistance to fire. CFRP can be a suitable reinforcement
material to repair post-heated RAC in numerous engineering applications. ABAQUS software [25–27] was used to conduct analysis of STSRCs subjected to axial compressive after exposure to 500°C and repaired by CFRP. FEA modelling was then used to investigate the stress state of each partial failure location. The experimental research provides necessary parameters for the calculation of ultimate bearing capacity, as well as the compressive strain curves of the entire process.

2. EXPERIMENT WORK

2.1. Materials

The Table 1 and 2 describe the material properties of the coarse aggregate and steel tubes which were used to make experimental specimens. Concrete mixes were designed with the grades of compressive strength according to the Chinese Standard [28]. The mix was comprised of ordinary Portland cement 32.5R, normal fine sand and gravel with natural aggregate or recycled aggregate size between 2.36 mm and 19 mm. The recycled aggregate was made from waste concrete of grade C30. The aggregate gradation curve shown in Figure 1 was obtained according to the “standard of recycle coarse aggregate GBT25177-2010”.

The strengthening material of CFRP sheets have a nominal thickness of 0.11 mm per ply, were externally boned onto the surface of STSRC by using a two-part epoxy mixed in a weight ratio of 2:1 and then cured at room temperature. The tensile strength, elastic modulus, mass per unit and ultimate elongation of CFRP sheets applied are 3496 MPa, 242 GPa, 299 g/m² and 1.71%, respectively.

2.2. Properties of Recycled Concrete

In order to provide important parameters for investigation of the failure mechanism and ultimate bearing capacity of STSRC after exposure to high temperature, the strength development curve as shown in Figure 2 demonstrates R0, R50 and R100 with respect to the ratio of recycled coarse aggregate equal to 0%, 50% and 100% respectively. The average compressive strength \( f_{cu} \) was obtained from cube crush experiments on cubes with side lengths of 150 mm conducted at 3, 7, 14, 21, and 28 days. The strength development coefficient is described by \( \eta_{28} \). Strength at any tested point in time is indicated by \( f_{cu} \), while \( f_{28} \) expresses the strength at day 28. The equation is described as follows:

\[
\eta_{28} = \frac{f_{cu}}{f_{28}}
\]

Table 3 provides the basic material properties of recycled concrete including compressive strength, tensile spitting strength, breaking strength, elastic modulus, etc., which can be employed to investigate the effects of temperature and the recycle aggregate ratio for bearing capacity and finite element analyses. The recycled concrete replacement ratio of 50% exhibited better properties than the other two tested types of concrete.

2.3. Specimen Preparation

As shown in Figure 3, STSRC specimens were 400 mm in height, with a cross-section side length of 100 mm, in which was place a square tube with a side length of 50 mm. Table 4 describes ten STSRC specimens intended for axial compression experiments, in which the effects of replacement ratio, high temperature and CFRP replacement on the bearing capacity and failure mode are listed.
2.4. Method of Temperature Increase

According to the work reported by Yaqub [29], the rate of temperature increase adopted in this test was 5°C/min. During the heating process, the temperatures of 300°C and 500°C were retained for 30 minutes in order to avoid in conformity of temperature distribution throughout the specimen. When the target temperature was achieved (500°C or 700°C), the specimens were post-heated for three hours to simulate real fire situations. Specimens were then naturally cooled to room temperature.

3. RESULTS AND DISCUSSION

3.1. Changes on Concrete Surface

Crack viewer EMIC-1M (MMT-3) was used in the present experiment to measure crack width induced by temperature stress. The detailed crack distributions are shown in Figure 4. Tiny cracks occurred on the concrete surface with the specimen was heated to 300°C, with no observed change in colour and partial concrete fall out. When the temperature was further raised to approach 500°C, the colour of the concrete surface turned dark red and the cracks widened and lengthened. When the temperature was raised to 700°C, the cracks distributed on the entire surface widened and lengthened, and the specimen colour turned white. Additionally, concrete with a primary ingredient of mortar began to deconstruct as result of the high temperature.

The recycled aggregate concrete demonstrated better physical properties than normal concrete when exposed to identical temperatures. Higher recycled aggregate content resulted in shorter, narrower, and fewer cracks. Concrete specimens with a replacement ratio of 50% demonstrated the best temperature resistance. The maximum crack width was 3.5 mm, which appeared on the concrete with no aggregate when exposed to 700°C.

3.2. Damage Mechanism

The primary factors which induce concrete damage as result of exposure to high temperatures are discussed as follows.

1. Hydrated mineral dehydration in the inner spatial structure or lacuna could be destroyed when crystal water or bound water of the hydrated mineral escaped under exposure to high temperatures.
2. The chemical component modification of cement formed by high concentrations of aluminum or calcium compounds could break down or chemically react with one another under high temperature conditions. The concrete was destroyed when the inner stress induced by the chemical reactions approached the ultimate capacity of the specimen.

<table>
<thead>
<tr>
<th>Replacement Ratio (%)</th>
<th>Compressive Strength (MPa)</th>
<th>Exposure to High Temperature (°C)</th>
<th>Tensile Splitting Strength (MPa)</th>
<th>Breaking Strength (MPa)</th>
<th>Elastic Module (GPa)</th>
<th>Poisson’s Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>28.30</td>
<td>19.79</td>
<td>2.27</td>
<td>2.89</td>
<td>22.51</td>
<td>0.47</td>
</tr>
<tr>
<td>50</td>
<td>29.08</td>
<td>15.67</td>
<td>2.35</td>
<td>2.48</td>
<td>21.43</td>
<td>0.42</td>
</tr>
<tr>
<td>100</td>
<td>27.33</td>
<td>14.24</td>
<td>2.14</td>
<td>3.23</td>
<td>21.28</td>
<td>0.43</td>
</tr>
</tbody>
</table>
Meanwhile, water evaporated from the concrete and the cement lost its adhesive ability, resulting in concrete distribution.

3. The breakdown of recycled coarse aggregate was significant. It was observed that at higher temperatures, longer or wider cracks on the surface of concrete specimens containing recycled coarse aggregate was associated with lower concrete strength.

4. Adhesive breakdown between the mortar and aggregate. When the cracks became longer and wider under high temperatures, the cement mortar cushion began to suffer from tensile stress, while the aggregate suffered from compressive stress. The adhesive quality was destroyed by the complex stress relationship between the mortar and aggregate.

3.3. Failure Modes of STSRC

The representative failure patterns for all tested STSRC specimens are presented in Figure 5. The failure mode presented in the comparison column [Figure 5(a)] is a typical shear failure pattern. For the columns 10, 20 and 30 not exposed to high temperatures and reinforced by CFRP, vertical cracks appeared in the middle of the column ends, which grew longer and wider when approaching the edge of columns, mid-height, from a direction of 60°. When the load approached the bearing capacity of column, the concrete was crushed, and the upper portion of the steel tube began to buckle. For the columns exposed to 500°C [Figure 5(b)], numerous small cracks appeared on the reddish surface of the columns as the result of damage induced by complex internal changes under high temperatures. Higher replacement content ratios resulted in the presence of more mortar powder on the surface of specimens. The

<table>
<thead>
<tr>
<th>Number</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>105</th>
<th>305</th>
<th>R105</th>
<th>R205</th>
<th>R305</th>
<th>R107</th>
<th>R207</th>
<th>R305</th>
</tr>
</thead>
<tbody>
<tr>
<td>Replacement Ratio (%)</td>
<td>0</td>
<td>50</td>
<td>100</td>
<td>0</td>
<td>100</td>
<td>0</td>
<td>50</td>
<td>100</td>
<td>0</td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td>High Temperature (°C)</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>500</td>
<td>500</td>
<td>500</td>
<td>500</td>
<td>500</td>
<td>700</td>
<td>700</td>
<td>700</td>
</tr>
<tr>
<td>CFRP Reinforcement</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
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</table>
failure mode of columns were similar to that of the comparison column, and the angle of cross cracks was equal to 45° when the upper portion of the steel tube began to buckle, with no concrete to attach to. No obvious warning signs appeared prior to column destruction. The failure mode of columns reinforced by CFRP after exposure to 500°C or 700°C was characterized by pasting one layer of CFRP sheets that were boned onto the column surfaces. However, the columns failed by snapping the CFRP sheets and the buckling of the steel tube at the upper portion of the specimens under the loading point. As shown in Figures 5(c) and 5(d). When the load approached one-third of the bearing capacity, the CFRP began to wrinkle, accompanied by the debonding of the CFRP sheets. Higher replacement ratios or temperatures resulted in worse failure modes. Eventually, the internal concrete was reduced to powder. The experiments demonstrated the typical failure modes for STSRC specimens with external FRP reinforcements.

### 3.4. Residual Bearing Capacity for the Specimens

Table 5 shows the bearing capacity of various tested columns. Based on these results, it can be concluded that ultimate loads depend primarily on temperature, the replacement ratio, and CFRP reinforcement. Comparison of columns of uniform conditions but different replacement ratios indicate that higher replacement ratios result in greater bearing capacity or stiffness. The specimen with a replacement ratio of 50% demonstrated better properties than the other specimens under axial compressive conditions. When the columns were exposed to a temperature 500°C, they lost nearly 29% of their bearing capacities. With the synergistic effect of CFRP, the growth ratio of bearing capacity almost approached 70%.

### 4. FEA MODELLING

#### 4.1. Materials

It’s well known that the interaction between a steel tube and its peripheral concrete is the key to STSRC behaviour when exposed to high temperatures. In order to analyze the interaction of STSRC reinforced by CFRP when exposed to high temperatures, ABAQUS software was employed to conduct calculations.

The concrete damage plasticity (CDP) mode which assumes two failure mechanisms (tensile and compressive failure) was used to determine the constitutive behaviour of concrete. The concrete yield was controlled by tensile and compression equivalent plastic strain ($\varepsilon_p^t$ and $\varepsilon_p^c$). For the steel tubes, an elastic-plastic stress-strain relation model consisting of five stages was employed. Detailed derivation of the stress-strain relationship has been previously described by Han, et al. [30,31]. In this paper: $E = 187$ GPa, and $f_y = 289$ MPa. CFRP, also an elastic-plastic material, was determined to be the orthogonal anisotropic material in this analysis. The detailed parameters of CFRP are identical to those mentioned above. The vertical elasticity modulus was 1/106 of the fibre development direction elasticity modulus as used in the calculation.

### 4.2. Element Type, Mesh and Boundary

Both the steel tube and the peripheral recycled con-

<table>
<thead>
<tr>
<th>Bearing Capacity</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>105</th>
<th>305</th>
<th>R105</th>
<th>R205</th>
<th>R305</th>
<th>R107</th>
<th>R207</th>
<th>R207</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experiment (kN)</td>
<td>166.9</td>
<td>187.3</td>
<td>177.2</td>
<td>118.0</td>
<td>143.6</td>
<td>285</td>
<td>311</td>
<td>270</td>
<td>197</td>
<td>204.8</td>
<td>163</td>
</tr>
<tr>
<td>FEM/kN</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>300</td>
<td>318</td>
<td>289</td>
<td>211</td>
<td>209</td>
<td>178</td>
</tr>
<tr>
<td>Error Rate (%)</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>5.26</td>
<td>2.25</td>
<td>7.04</td>
<td>7.11</td>
<td>2.45</td>
<td>9.2</td>
</tr>
</tbody>
</table>
crete of STSRC were modeled using 8-node brick elements (C3D8R), with three translational degrees of freedom at each node, as shown in Figure 6(a).

As a result of symmetry, 1/8 of the STSRC reinforced by CFRP was taken into account for the subsequent analysis. Fixed boundary conditions were applied to the bottom surfaces of columns, and the top surfaces of columns represent free boundaries. The binding constraints between the steel tube and concrete are set (concrete and CFRP). The uniformly distributed load is applied to the top surface of columns. As shown in Figure 6(b), the mesh size of CFRP, concrete and steel tube is equal to 2 when considering computational efficient and accuracy. The calculation concluded when the shear stress transferred by the surfaces exceeded the limit value.

4.3. Comparisons of Results

The bearing capacities predicted using the FEA model described as above were compared to the six experimental results provided in Table 5. The errors between the predicted and measured results are all less than 10%. The vertical stress distributions of cross sections or ventral surfaces are shown in Figure 7. The predicted compression moment load (F) and deflection (D) or strain (με) of R205 (R207) is calculated and shown in Figure 8. Results generally indicate that the F-D curves demonstrate good agreement between the predicted and measured results at the elastic stage, as well as small negligible deviations in the plastic stage.

As shown in Figures 8(c) and 8(d), the measured vertical stress is less than the predicted vertical stress of the debonding of CFRP sheets from the concrete; the CFRP takes shape in the timpanists. The load-compression strain curve demonstrates an obvious strengthening phase. The yield load is 230 kN, which exceeds the bearing capacity of non-reinforced specimens. Results demonstrate that CFRP plays a part in the restraint when the internal concrete is crushed, which is in agreement with the failure mode as shown in Figures 5(c) and 5(d).

5. FAILURE MODE AND DISCUSSIONS

5.1. The Vertical Strain Distribution on CFRP

The direction of vertical strain distribution development in CFRP at the moment of failure is shown in Figure 9. The left portion represents the loading end, while the right portion depicts the middle portion of the column. The main conclusions drawn from Figure 10 are described as follows.

1. The loading fixed constraint in the analysis is set at the loading end. The strength constraint can limit the development of tensile strain at the loading end. The CFRP in the middle portion of the columns snaps when the tensile stress from the lateral deformation of the concrete exceeds the limiting value.

2. The direction of the uniform distribution of vertical strain fiber development results from the two layers of CFRP applied during reinforcement, which contribute to improvement of the elastic modulus of composite columns.

3. The predicted failure area was smaller than that observed in the experiment, as shown in Figure 10, in which the ultimate strain approaches 14.44628 × 103με. The column is destroyed when tensile stress approaches the limit of CFRP in the finite element analysis. However, in the experiment, the CFRP
snapped before the column lost its bearing capacity, resulting in stress redistribution on the CFRP and an increasingly large failure area.

5.2. The Lateral Strain Distribution on CFRP

The stress concentration appeared at the corners of the columns reinforced by two layers of CFRP. Future analysis will investigate problems related to the concentrated stress or the improvement of the bearing capacity with a magnifying arc radius.

5.3. The Stress Distribution for the Central Cross Section of the Columns

As a result of symmetry, Figure 11 shows the stress distribution of a 1/4 cross-section at the center of the column. The primary regularity mechanics of STSRC reinforced by CFRP is described as follows.

The column corner was the main place of compression stress by CFRP, in which the maximum compressive stress is equal to 39.6 MPa. With the constraints of CFRP and the steel tube, the stress of the area between the CFRP and the steel tube initially decreases, and then increases. CFRP provides lateral restraint at the column corner, a bigger arc radius, and results in better strain effects due to the CFRP restraint. The circular cross-section columns gives better reinforcement effect than square cross-sections.

5.4. The Maximum Principal Stress on Concrete Profile

As shown in Figure 12, the maximum principal
stress is negative due to the fixed constraint at the top end, and decreases from the side to the center of the concrete profile. The maximum principal stress exceeded the tensile strength limit, resulting in concrete destruction consistent with the failure mode in the experiment. More layers of CFRP results in more even stress distribution, inducing better restraint effects. The disabled position of CFRP snaps from the center transfer to the corner.

5.5. The Predicted Results for the Steel Tube

As compared to Figure 13b and the buckling position of the steel tube, the steel tube buckles when the stress exceeds the yield limit, which results in decreasing stress at the top end from corner to center. The minimum stress is equal to 117MPa in Figure 13a.

Figure 13b shows the negative displacement at the low stress area where the steel tube is pressed into the side, which causes the other to become convex as a result of deformation compatibility. For not considering large deformation, and the constitutive relation is two sections, the calculated displacement at yield moment is smaller the actual measured.

6. CONCLUSION

The present research provides an attempt to study the mechanical properties of CFRP reinforced STSRC when exposed to high temperatures. The following observations and conclusion are drawn based on the limited research reported in this paper.

1. The recycled aggregate concrete demonstrated better physical properties than normal concrete when sustaining damage from identical high temperatures. Higher replacement content resulted in shorter, narrower, and fewer cracks. The concrete specimen with a replacement ratio of 50% demonstrated the best temperature resistance. This paper also summarized the primary influences and mechanisms that resulted in temperature damage to the concrete specimens.

2. The high temperature greatly influenced the failure mode and ultimate load carrying capacities. For the specimens with CFRP reinforcement, great increases were observed in load carrying capacity and displacement. Moreover, the failure mode transitioned from ductile to brittle failure.

3. Finite element analysis was employed to investigate CFRP reinforced STSRCs after exposure to high temperatures while subjected to axial compression strength. A comparison of results calculated using this model shows good agreement with experiment results.

7. ACKNOWLEDGEMENT

The authors gratefully acknowledge the financial
support provided by the National Science Foundation of China (No: 51408382). Thanks are also given to Mrs. Lang Li, Pu Jia and Miss Wei Liang for their assistances in the experimental work.

8. REFERENCES


