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Flexural Behavior of Fire-Damaged Reinforced Concrete Slabs Repaired with Near-Surface Mounted (NSM) Carbon Fiber Reinforced Polymer (CFRP) Rods

Cao Nguyen Thi¹, Withit Pansuk²* and Lluis Torres³

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Abstract
After fire, the decrease in load capacity of reinforced concrete (RC) structure will take place and may lead to a damage. This research brings out a method to repair flexural reinforced concrete members after fire by using near-surface mounted (NSM) carbon fiber reinforced polymer (CFRP) rods. In this study, a series of slabs were conducted to evaluate the flexural behavior of fire-damaged RC structures strengthened with NSM CFRP rods and repairing material. The arrangement or the location of NSM CFRP rods were the main factor that had been considered to evaluate the effectiveness of this method. Besides, the direct bond test of C-shaped specimens was carried out to investigate the bond behavior between CFRP rods and concrete in three different embedding positions of CFRP rods. Based on the experimental databases, it is clear to conclude that the strengthened slabs not only improved endurance limits but also improved load-carrying capacities and stiffness values as compared to control slab and fire-damaged slab. Especially, the test results showed that slabs with CFRP rods embedded in repairing material overlay had more load-carrying capacity than slabs with CFRP rods embedded inside concrete or between concrete and repairing material overlay.

1. Introduction
In recent years, design and repair of fire-damaged concrete structure have emerged as a subject of remarkable interest to both specialist and public. When concrete structure are exposed to high temperatures, mechanical losses of its properties take place (Sangluiaia 2013). This may result in cracking and loss in the bearing capacity of components (Haddad et al. 2011). After fire, concrete suffers a reduction of its strength as well as damage that can be observed at the surface. Furthermore, another main problem is the reduction of bond strength between concrete and steel reinforcement, which cause reduction in the structural load carrying capacity. Consequently, after exposure to fire, reinforced concrete (RC) structures cannot be used as effectively and safely. Nowadays, a lot of different techniques may be used for repairing and retrofitting of fire-damaged RC structures. Among these techniques, the use of fiber reinforced polymer (FRP) composites has become increasingly popular due to their unique properties, including a high strength-to-weight ratio, ease of installation, excellent corrosion resistance and long fatigue life. For flexural strengthening, this technique may be used applying different kinds of FRP products as for example sheets, strips or rods to the tension side of the member to take advantage of their high strength in tension. Two popular strengthening methods are distinguished by the different ways in which FRP materials are applied, namely Externally Bonded (EB) Reinforcement and Near Surface Mounted (NSM) Reinforcement.

Most of researches on strengthening or repairing flexural fire-damaged structures focused on using EB FRP systems. Haddad et al. (2011) used carbon FRP (CFRP) and glass FRP (GFRP) sheets to repair heat damaged slabs (heated at 600°C for 2h). Based on the results, the authors concluded that CFRP and GFRP strengthened elements regained up to 158 and 125% of the control slab’s ultimate load capacity with a substantial increase in stiffness, first cracking load, with a corresponding decrease in mid-span deflections at ultimate load. Kai et al. (2011) carried out a series of tests with T-beams (5400x200x200 mm and flange 900x80 mm) exposed to fire at 1000°C. After fire exposure, the fire damaged beams were strengthened with CFRP sheets following three steps: 1) serious fire damaged concrete was removed before strengthening; 2) epoxy bonding adhesive was used to retrofit the damaged surface of the T-beams; and 3) CFRP sheets were used to strengthen the beams. External strengthening with CFRP sheets was found to increase the yield and ultimate loads of the fire-damaged T-beams.

The second strengthening method indicated above, i.e. NSM reinforcement, has become an emerging retrofitting technique for RC members and masonry which has attracted an increasing interest of research as well as practical applications (De Lorenzis et al. 2002; Sharaky et al. 2013). In the NSM FRP technique, grooves are first cut into the concrete cover of a RC member and the FRP...
reinforcement is bonded therein with groove filler. This technique offers various advantages over the EB FRP method such as: it is less prone to debonding from the concrete surface, provides greater protection from external damage, can be more easily pre-stressed and the aesthetic of the strengthened structure is virtually unchanged (Al-Mahmoud et al. 2010; De Lorenzis and Teng 2007). Based on those benefits, the NSM technique could also be an effectively choice for strengthening or repairing fire-damaged RC structures.

After a fire, the concrete cover is often the most deteriorated part of a RC member. Therefore with regard to a fire-damaged structure, NSM FRP cannot be applied in the usual way that has been conducted in prior works for not fire-damaged RC members. FRP reinforcement cannot be installed in the damaged cover and this concrete layer must be replaced by repairing material.

Due to the lack of research works on the use of NSM FRP in RC structures after fire, in the present study an experimental program that focus on repairing and strengthening fire-damaged flexural RC structure using the NSM FRP method has been carried out. NSM CFRP rods have been used for repairing and strengthening RC slabs previously subjected to fire conditions. The specimens in the experimental campaign were prepared following the next main steps: a) the RC slabs were exposed to fire; b) after fire, the fire-damaged concrete layer was removed; c) the NSM CFRP reinforcement was applied on the tension side of the remaining part of slab after the second step; the layer of damaged concrete was replaced by repairing material and to restore the full size of the slab.

Three different positions of the NSM CFRP rods with respect to the non-damaged concrete have been studied in this work: inside the concrete; between the concrete and repairing material; in repairing material. Pull-out tests with C-shaped specimens suitable for NSM FRP reinforcement have been carried out to study the bond behavior between FRP reinforcement and surrounding substrate corresponding to three situations as mentioned above. Subsequently the NSM FRP strengthened slabs have been tested in bending to obtain the load-deformation response. Results of the tests carried out are presented and discussed.

2. Experimental program

The tests were conducted in the Fire Laboratory and the Concrete Laboratory at Chulalongkorn University. The experimental program consisted of two parts: the bond test on C-shaped specimens and the flexural test on RC slabs.

2.1 Bond test

To investigate the bond behavior between NSM CFRP rods and surrounding substrate with different strengthening locations, a bond test of C-shaped concrete block with a square groove in the middle for embedment of NSM rod was accomplished. The dimensions of tests were designed based on some previous researches (De Lorenzis et al. 2002; De Lorenzis and Teng 2007; Novidis et al. 2007; Sharaky et al. 2013) as shown in Fig. 1. Three different positions of NSM CFRP rods are shown in Fig. 2 and summarized in Table 1: Group A – inside concrete; Group B – between concrete and repairing material overlay; Group C – inside repairing material overlay.

The concrete blocks were cast using ready mixed concrete with concrete grade of 35 MPa and the specimens were not fired. After 28 days of curing, the groove was first cut into the concrete block and NSM CFRP rods were applied therein. Epoxy paste SIKA®-30 was

Note: \( d_b \)-diameter of FRP bar (9 mm); \( b_g \)-groove width; \( h_g \)-groove high.

Fig. 1 Dimensions of C-shaped Specimens.

Fig. 2 Different locations of CFRP rods in C-shaped specimens.
used as a groove-filling material. The dimension of
grooves were 15x15 mm in case of specimens A and
15x7.5 mm with respect to specimens B. On the contrary,
grooves and epoxy were not used in case of specimens C.
After the FRP rod had been placed in the groove with
the surrounding epoxy for bonding to surrounding concrete,
a thin layer of resin SIKADUR® 32TH was brushed on
the concrete surface where the repairing material would
be applied. This resin was used for supporting the good
bond between concrete and repairing material. Then a
layer of repairing material SIKA® MONOTOP® 614T
was covered above with the depth of 5 cm. This was also
a depth of fire-damaged concrete layer that had been
observed in slabs after fire. The details of six specimens
with three different NSM FRP rod positions and variable
groove sizes are performed in Table 1.

Besides, four strain gauges were glued onto the surface
of every CFRP rods (Fig. 3 and Fig. 4) to measure the
axial strains. All of the specimens had been cured at the
temperature room before they were tested. The load was
applied to the load end of CFRP rod by a testing machine
with a capacity of 500 kN, under displacement control at
a rate of 1 mm/min. One LVDT was used to measure the
free end slip. The applied tensile load and load end slip
were measured from the test machine.

2.2 Flexural test
This test was performed to investigate the bending be-
havior of fire-damaged RC slabs repaired and strength-
ened with repairing material and NSM CFRP rods. The
experimental process is represented in Fig. 5 following
by six steps.

a) Step 1: casting of RC slabs
Eight RC slabs were designed under standard ACI
318-08 (ACI 2008) and cast in the laboratory. Every slab
had the same design as described in Fig. 6. The slabs had
the dimension of 1000x900x150 mm and were rein-
forced longitudinally in flexure with 10 mm diameter
steel bars. The concrete cover had a depth of 20 mm. The
slabs were divided into two series by concrete strength of
24 and 35 MPa. They had been cured for 28 days before
conducting the fire test. Two slabs (SC1 and SC2) were
used as control specimens without firing or strengthening.
The other six slabs (SF1-SF6) were subjected to fire.

b) Step 2: fire test
The installation and testing of fire exposure were oper-
nered.

The process of flexural test.
ated under ASTM E119 standard (ASTM 2000). The slabs had been fired for 1 hour and 30 minutes to reach the peak point of temperature curve about 1000 °C. The specimens were exposed to fire on one side reinforced longitudinally by three steel bars. The fire test was carried out by using a furnace as shown in Fig. 7.

c) Step 3: removing the fire-damaged concrete layer

After fire, a fire-damaged concrete layer in every slab was observed on the side directly exposed to fire (Fig. 8). The depth of the weakened concrete layer was defined based on the results from a previous research carried out by Daungwilailuk (2012). The condition and installation of fire test in that research were similar to the test in this study. A semi destructive test or core test was carried out by drilling the fire-damaged slabs to collect the cores having a diameter of 50 mm. The depth of weakened concrete layer was assessed by color change as well as pull-off test on different depths. Based on it, a degraded concrete layer of 50 mm was removed on the fire-damaged slabs in this experimental study to arrive at the sound material.

A hammer machine was used to remove this weakened layer. To make the work easier, a concrete saw was used to cut lines on the slab surface and then the degraded concrete could be chiseled out by a hammer machine (Fig. 9). In this study, a pull-off test also was operated to determine the tensile strength of concrete on the surface that exposed to fire based on standard ASTM C1583-04 (ASTM 2005) (Fig. 10). This test can be used as an indicator of the adequacy of surface preparation before applying a repairing material. The correlation between the pull-off load and concrete tensile strength makes it possible to have sufficient information on concrete strength for the application of mortar and protective coating (fib 2008). The values of pull-off test are shown in Table 2. These values were collected at the depth of 0 mm and 50 mm from the surface.

d) Step 4: installing the NSM CFRP rods

After fire, except for the slab SF1, which was not re-
paired, the remaining specimens (SF2-SF6) were strengthened by NSM CFRP rods and repairing material. After the weakened layer had been removed, a concrete saw was used to cut the grooves in the new surface along the slab (Fig. 11). The concrete surface was smoothed by a grinder machine and cleaned by a blower machine. Subsequently, epoxy was installed into the grooves for bonding the rods to the surrounding concrete by using a small trowel (Fig. 12a). Then CFRP rod was installed into the groove filled with epoxy (Fig. 12b). Finally, a repairing material overlay was applied above. Details of the different slab specimens are given in Table 3 and Fig. 13 and are described below:

- Slab SF2 (concrete grade of 24 MPa): 6 grooves of 15x15 mm were first cut inside the concrete part. Then epoxy and CFRP rods were installed into the grooves. The distribution of NSM rods is indicated in Fig. 13. The distance between the longitudinal steel bar and the NSM rod was 20 mm.
- Slab SF3 (concrete grade of 24 MPa): Similarly to slab SF2, 6 grooves of 15x15 mm were first cut inside the concrete part, which were filled with epoxy and NSM rods, but their distribution was different from slab SF2. The distance between the longitudinal steel reinforcement and the NSM rod was 100 mm, except for the extreme rods in which was 50 mm. In this case, the dimension of groove also was 15x15 mm and CFRP rod in full depth of groove.
- Slab SF4 (concrete grade of 35 MPa): the design, distributions and dimensions were similar to SF3.
- Slab SF5 (concrete grade of 35 MPa): in this case the reinforcement was placed in the repairing material.

Table 2 Values from pull-off test.

<table>
<thead>
<tr>
<th>Slab</th>
<th>Control slab</th>
<th>Fire-damaged slab</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SC1</td>
<td>SC2</td>
</tr>
<tr>
<td>Concrete grade (MPa)</td>
<td>24</td>
<td>35</td>
</tr>
<tr>
<td>Depth (mm)</td>
<td>00</td>
<td>00</td>
</tr>
<tr>
<td>Load (kN) – First Time</td>
<td>4.35</td>
<td>7.10</td>
</tr>
<tr>
<td>Load (kN) – Second Time</td>
<td>5.80</td>
<td>7.30</td>
</tr>
<tr>
<td>Load (kN) – Third Time</td>
<td>4.90</td>
<td>6.50</td>
</tr>
<tr>
<td>Average Load (kN)</td>
<td>5.02</td>
<td>6.97</td>
</tr>
<tr>
<td>Average Stress (MPa)</td>
<td>2.55</td>
<td>3.55</td>
</tr>
</tbody>
</table>
The 6 CFRP rods were installed onto the surface of the concrete and located at the same position on the horizontal axis as for slabs SF3 and SF4.

- Slab SF6 (concrete grade of 35 MPa): 6 grooves of 15x7.5 mm were first cut into the concrete part and epoxy was used to fill the grooves. Only the half of every CFRP rod was installed inside the groove. The other half was covered by repairing material instead of epoxy (Fig. 13).

e) Step 5: installing the repairing material
In the last step of the procedure, repairing material was applied to replace the weakened concrete layer that had been removed in step 3. A steel frame was fixed around the slab as a mold to cast the repairing material layer (Fig. 14). Before the installation of repairing material, a resin was applied directly to the concrete substrate to improve bond between concrete and repairing material. Finally, the repairing material was poured on to get the thick of 50 mm and subsequently the surface was smoothed (Fig. 15 and Fig. 16).
f) Step 6: Flexural test
All of these specimens had been cured at the room temperature for 28 days before the bending test. The simply supported slabs with the clear span of 800 mm were tested under three-point bending test to determine the load-deflection curves. The applied load was controlled by a hand-operated hydraulic jack with full capacity of 500 kN under a loading rate 7 kN/min and was monitored by the load cell. An LVDT was placed at the middle point on the top of the slab and two other LVDTs were placed at 1/4 point of the span to obtain the deflection profiles. Strain gauges were bonded to the CFRP rods to measure the strain profiles. Fig. 17 shows details of the setup of strain gauges on CFRP rods in strengthened slabs. In every slab, nine strain gauges were used to bond on three CFRP rods. They were installed along half of the bond length of CFRP rods in a half of the slab.

2.3 Material properties
The slabs were cast by using ready mixed concrete with two concrete grade of 24 and 35 MPa. The compressive strengths obtained from the tests performed on the standard cylinder (150x300mm) were 24.5 and 35.4 MPa. The epoxy paste SIKADUR®-30 was used for the embedding of NSM rods, which had a compressive strength and tensile strength of 85 MPa and 26 MPa at curing temperature of 35 °C in seven days. It had compressive and tensile E-modulus of 9600 and 11200 MPa. SIKADUR®-32TH was used as a resin for bonding of repairing material to concrete substrate. Its compressive and tensile strength were 60 and 18 MPa, the bond strength to concrete reached 2.5 MPa. The type of repairing material was SIKA®MONOTOP®-614T which had a compressive strength of 60 MPa at curing time of sixty days and bond strength of 2.0 MPa to concrete. These parameters were provided by producer. The spirally-wound CFRP rod was used and its properties were determined on direct pull out test, including tensile strength of 2220 MPa and E-modulus of 133 GPa. The length of CFRP rod is 1000 mm and diameter is 9 mm.

3. Test results and discussion
3.1 Bond test on NSM CFRP rod in concrete
3.1.1 Failure Pattern
The types of failure are reported in Table 4. The pull-out failure at the interface between CFRP rod and

<table>
<thead>
<tr>
<th>Slab code</th>
<th>Concrete Strength (MPa)</th>
<th>Dimension of groove (mm)</th>
<th>Note</th>
<th>Depth of repairing material (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SC1</td>
<td>24</td>
<td>-</td>
<td>-</td>
<td>_Unfired</td>
</tr>
<tr>
<td>SC2</td>
<td>35</td>
<td>-</td>
<td>-</td>
<td>Unfired</td>
</tr>
<tr>
<td>SF1</td>
<td>24</td>
<td>-</td>
<td>-</td>
<td>Fired-Unrepaired</td>
</tr>
<tr>
<td>SF2</td>
<td>24</td>
<td>15x15</td>
<td>NSM rod in full depth groove</td>
<td>Fired-Repaired</td>
</tr>
<tr>
<td>SF3</td>
<td>24</td>
<td>15x15</td>
<td>NSM rod in full depth groove</td>
<td>Fired-Repaired</td>
</tr>
<tr>
<td>SF4</td>
<td>35</td>
<td>15x15</td>
<td>NSM rod in full depth groove</td>
<td>Fired-Repaired</td>
</tr>
<tr>
<td>SF5</td>
<td>35</td>
<td>-</td>
<td>-</td>
<td>No groove and filling material.</td>
</tr>
<tr>
<td>SF6</td>
<td>35</td>
<td>15x7.5</td>
<td>NSM rod in a half depth groove</td>
<td>Fired-Repaired</td>
</tr>
</tbody>
</table>

Fig. 14 Installing steel frame around the slab to cast repairing material.
Fig. 15 Installing repairing material.
Fig. 16 The slab after repairing.

![Fig. 17 Position of strain gauges (SG) on CFRP rods in slab.](image)

Table 3 Details of slabs.
epoxy or repairing material was the critical mechanism in all cases. A very thin layer of epoxy was observed along the bonded length of the rod. This indicates that the interface failure occurred due to cohesive shear failure in the epoxy. It may be similar to the observations reported by Sharaky et al. (2013), Galati and De Lorenzis (2009). As for specimens of group B and C, a very thin layer of repairing material was also observed on the rod, which could be caused by cohesive shear failure in the repairing material. Since the CFRP rod and groove were covered by repairing material, it was not possible to observe the failure in the substrate adjacent to the groove during the test. No visible cracks were on the surface of repairing material and the concrete block. At the load end of specimens C3, C4, C5 and C6, a layer of repairing material in the form of cone-shaped crack has been still attached on the surface of the rods (Fig. 18). This was not observed in cases of specimens C1 and C2.

3.1.2 The maximum load
Figure 19 shows that the ultimate load of specimens having grooves was different negligible (13 – 16 kN). On the contrary, the ultimate load of specimens without grooves (C5, C6) was around three times higher than the others. Based on these results and the observed failure patterns, it seems that the location of CFRP rod significantly affected the load capacity and that the repairing material supplied higher bond strength than the epoxy. This effect is also discussed in the section of bond behavior below.

3.1.3 The tensile strain distribution along CFRP rod
The tensile strain distributions along the CFRP rod are reported as shown in Fig. 20 for all specimens. The values on horizontal axis starts from the free-end to load-end along the bonded length of the rod and each curve correspond to a specific load level. At the lower load levels, the strains were small toward the free end and the strain distribution along the bonded length was nonlinear. As the load increases, the strain distribution can be approximated to follow a linear distribution along the bonded length with the highest value obtained at the position near load end. This is similar to previous reported on C-shaped pull-out test (De Lorenzis and Nanni 2002; Hassan and Rizkalla 2004; Soliman et al. 2010). This result indicates that the bond stresses between CFRP rod and epoxy redistributed along the bonded length as the load increased. The values of strain from specimens in group C were higher than other groups due to their high load capacity.

3.1.4 Bond stress-slip relationship
Based on the well-known Bertero-Popov-Eligehausen (BPE) relationship (Eligehausen et al. (1982) for bond of steel reinforcement in concrete, the bond-slip curve for pull-out failures of CFRP rod has been adopted from the equations of ascending branch and descending branch (De Lorenzis et al. 2002):

$$\tau(s) = \tau_m \left( \frac{s}{s_m} \right)^\alpha, 0 \leq s \leq s_m$$

(1)

$$\tau(s) = \tau_m \left( \frac{s}{s_m} \right)^\alpha, s \geq s_m$$

(2)

where $\tau$ is the local bond stress, $s$ is the local slip, $\tau_m$ is maximum bond stress and $s_m$ is correspondingly maxi-

Table 4 The results of bond tests.

<table>
<thead>
<tr>
<th>Group</th>
<th>Specimen Code</th>
<th>Maximum Load $P_{\text{max}}$ (kN)</th>
<th>Average bond stress (MPa)</th>
<th>Average slip at $P_{\text{max}}$ (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>C-1</td>
<td>16.1</td>
<td>2.843</td>
<td>4.31</td>
<td>Pull-out (rod-epoxy)</td>
</tr>
<tr>
<td></td>
<td>C-2</td>
<td>15.2</td>
<td>2.691</td>
<td>4.46</td>
<td>Pull-out (rod-epoxy)</td>
</tr>
<tr>
<td>B</td>
<td>C-3</td>
<td>13.4</td>
<td>2.363</td>
<td>4.29</td>
<td>Pull-out (rod-epoxy and repairing material)</td>
</tr>
<tr>
<td></td>
<td>C-4</td>
<td>16.7</td>
<td>2.955</td>
<td>5.78</td>
<td>Pull-out (rod-epoxy and repairing material)</td>
</tr>
<tr>
<td>C</td>
<td>C-5</td>
<td>39.1</td>
<td>6.917</td>
<td>4.92</td>
<td>Pull-out (rod-repairing material)</td>
</tr>
<tr>
<td></td>
<td>C-6</td>
<td>43.4</td>
<td>7.669</td>
<td>4.99</td>
<td>Pull-out (rod-repairing material)</td>
</tr>
</tbody>
</table>

Fig. 18 CFRP rods after testing.

Fig. 19 Maximum load values of C-shaped specimens.
mum slip. Parameter $\alpha$ and $\alpha'$ were calibrated by best fitting of the experimental results. The value of $\alpha$ can be obtained by equating the area underneath the ascending branch of the experimental curve to the value (De Lorenzis et al. 2002):

$$\int_0^{s_m} \tau_m \left( \frac{s}{s_m} \right)^\alpha \, ds = \frac{\tau_m s_m}{1 + \alpha}$$  \hspace{1cm} (3)

As visible in Fig. 21, the bond-slip behavior of all specimens has an initial steep ascending branch until the maximum value followed by a rather smooth softening branch toward an asymptotic value of bond stress due to the residual friction.

The curves of specimens in groups A (C1, C2 - NSM in full depth groove) and B (C3, C4 - NSM in a half depth...
groove) are not much different, and their highest bond strengths were below 3 N/mm². However, the ascending branch of specimens C3 and C4 have a higher slope at the initial load levels in comparison with those of specimens C1 and C2. Group C (C5, C6 – CFRP rod in repairing material) reached maximum bond strengths of more than 6 N/mm², about three times higher than the other specimens, with high slope in the initial branch. Based on the observation of failure patterns and the bond stress – slip curves, it could be concluded that the location of CFRP rod was a main parameter affecting the bond performance of the NSM systems that were used in this study. Appearance of repairing material in this case affected the bond stress on NSM rod and lead to higher bond performance than the epoxy resin.

In case of group B, with half of rod embedded in repairing material, a higher slope in the bond-slip response could be created in comparison with group A (the rod in full depth groove with surrounding epoxy). Therefore, at lower slips, the higher bond force could help the rod of specimens in group B to prevent the pull-out load, or delay the slip for similar loads. The initial slope of group B was similar to that of group C, however it showed a change for a value around 1.5 N/mm² to follow a trend similar to specimens of group A. As a result, group B could not get a high bond stress as group C. In this study, the dimension of square groove was selected to have a width-to-diameter ratio of approximately 1.7, based on previous researches. De Lorenzis and Teng (2007) reported a minimum value of 1.5 for lightly sand-blasted bars, which was enough to prevent splitting failure of the epoxy cover (De Lorenzis and Nanni 2002). Based on it the groove dimension were not likely to affect the observed results.

### 3.1.5 Development Length
The fracture energy of the bonded joint can be obtained from the curves in Fig. 21 by equating the area underneath the bond-slip curve. In case of FRP reinforcing rods, this value is usually infinite due to an unlimited post-peak friction branch as can be observed in the figure, hence a development length always exists (De Lorenzis and Teng 2007). Herein, the value of development length will be compared in each case. First, the equilibrium equation of tensile force on rod is used:

\[
\frac{d\sigma}{dx} = \frac{E}{d} \frac{ds}{dx}
\]

where \(\sigma\) is tensile stress in rod and \(E\) is the elastic modulus of the rod. From Eqs. (4) and (5), the limit tensile stress in the rod \(\sigma_l\) can be achieved corresponding to a slip equal to \(s_m\) and average bond stress is \(\tau_m\) by the equation (Cosenza et al. 2002):

\[
\sigma_l = \sigma(s_m) = \sqrt{\frac{8E\tau_ms_m}{d}(1+\alpha)}
\]

Furthermore, the development length \(l_m\) can be calculated from Eqs. (4), (5) and (6):

\[
l_m = \frac{\pi \tau_ms_m}{4d} \Rightarrow l_m = \sqrt{\frac{Eds_m}{2\tau_m(1+\alpha)}}
\]

The values of development length are reported in Table 5 for each specimen. According to these values, a minimum embedment length needed for NSM CFRP rod to prevent the failure at epoxy-rod interface in case of C5 and C6 (CFRP rod was embedded inside repairing material) was approximately 18.50 cm. Meanwhile, this value with respect to C1, C2 (CFRP rod was embedded inside concrete) and C3, C4 (CFRP rod was embedded between concrete and repairing material) should be at least 3 times higher than the case of C5, C6. Besides, the relationship of the tensile stress on rod \(\sigma\) versus the dimensionless development length \(l/db\) was also predicted as show in Fig. 22 by equation (Cosenza et al. 2002):

<table>
<thead>
<tr>
<th>Specimen Code</th>
<th>(\tau_m) (N/mm²)</th>
<th>(s_m) (mm)</th>
<th>(A_s) (N/mm)</th>
<th>(a)</th>
<th>(\alpha)</th>
<th>(l_m) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-1</td>
<td>2.837</td>
<td>1.763</td>
<td>3.641</td>
<td>0.374</td>
<td>-0.40</td>
<td>52.03</td>
</tr>
<tr>
<td>C-2</td>
<td>2.638</td>
<td>1.520</td>
<td>2.637</td>
<td>0.520</td>
<td>-0.38</td>
<td>47.62</td>
</tr>
<tr>
<td>C-3</td>
<td>2.317</td>
<td>1.474</td>
<td>2.755</td>
<td>0.240</td>
<td>-0.35</td>
<td>50.82</td>
</tr>
<tr>
<td>C-4</td>
<td>2.897</td>
<td>1.648</td>
<td>3.792</td>
<td>0.259</td>
<td>-0.35</td>
<td>45.45</td>
</tr>
<tr>
<td>C-5</td>
<td>6.784</td>
<td>0.513</td>
<td>2.526</td>
<td>0.378</td>
<td>-0.10</td>
<td>18.12</td>
</tr>
<tr>
<td>C-6</td>
<td>7.521</td>
<td>0.544</td>
<td>3.209</td>
<td>0.275</td>
<td>-0.18</td>
<td>18.43</td>
</tr>
</tbody>
</table>
\[ l = l_n \left( \frac{\sigma}{\sigma_t} \right)^{\frac{1}{k}} \] (8)

The curves in Fig. 22 have been derived by equating the development length for each value of the stress. In case of installing CFRP rod inside repairing material without groove and epoxy, the embedment length is always shorter than other cases. For example, if the tensile stress on the rod can reach 2000 N/mm² then the embedment length in this case only need approximately 24 times the diameter of the rod. However, the embedment length ranging from 87 to 101 times the rod diameter with respect to two remaining cases.

### 3.2 Flexural test of slabs
#### 3.2.1 Failure patterns
The crack patterns of each specimen are depicted in Fig. 23. The bending test on two control slabs (SC1 and SC2) had been carried out and their failure patterns were typical bending cracks that located in the maximum moment zone. This type of failure can be observed with respect to any normal flexural reinforced concreted members. In case of SF1, slab after fire without strengthening by CFRP rods, the bending cracks were also observed similar to the failure pattern of SC1 and SC2. However, the cracks appeared early due to the loss of load capacity of the slab after fire. The initial cracks were observed at the load of 60 kN on slab SF1 while the cracks started at 80 kN on slabs SC1 and SC2.

All slabs repaired with CFRP rods and repairing material cracked at an early stage of 60 kN and initial flexural cracks started on the side of repairing material at the highest moment zone. As the load increased, major bending cracks propagated towards the compression side and the crack width expanded until failure occurred. At higher load levels, inclined shear cracks occurred towards the load point. The cracks propagated in the slabs without any change in their angle at the intermediate layer. This indicated that a good bond between concrete and repairing material was formed in all repaired slabs. Only the slab SF6 had a small interface debonding area between the concrete and the repairing material while this was not detected in other slabs. It started when the slab SF6 reached the ultimate load level and propagated from the position where the shear crack cut the interface.
3.2.2 Load Deflection relationship

Based on the results of monotonic static loading test, it is clear to observe that slabs strengthened with CFRP rods had a significant effect to increase the stiffness of the specimens in comparison with control slabs and obviously fire-damaged slabs. The ultimate load of strengthened slabs was two times higher than the control slabs. In general the curves of strengthened slabs can be divided approximately three stages involve before cracking, steel yielding and post-yielding stage. This behavior was mentioned in the research of Al-Mahmoud et al. (2010). The first stage is short and it follows a linear elastic behaviors pattern, which like controls slabs. In the second stage, cracking starts and cracks occurred from the maximum moment zone. As load increased and reached the value that led to the yielding of steel bars, this stage will end. The last stage occurred between the steel yielding and the failure of slab. At this stage, the steel has yielded whereas the CFRP rods behaviour is elastic and control the crack widths until failure (Sharaky et al. 2014). Moreover, the increase of deflection has a higher rate than the previous stage. In the comparison between the strengthened slabs, the location of NSM CFRP rods had affected to their performance as mentioned below.

Figure 24 shows the test results of slabs SF2 and SF3 with concrete strength of 24 MPa. It is observed that strengthened slabs got higher ultimate load and stiffness than control specimen SC1 and fire-damaged slab without strengthening SF1. Slab SF1 only reached the ultimate load about 80% over control slab. Meanwhile, slabs SF2 and SF3 reached an ultimate load of higher 200 kN, about more 2 times higher than the control specimen. In the comparison between SF2 and SF3, the curve of slab SF2 shows a higher ultimate load and higher stiffness than the specimen SF3. Both of the slabs were strengthened with NSM CFRP rods inside concrete, but they had the difference in distance from NSM CFRP rod to longitudinal steel reinforcements. These values of the distance were 20 mm for SF2 and 100 mm for SF3. So this difference may have led to the change in redistribution of stress, which as a result of the difference of bending behavior over slabs.

The load-deflection relationship of specimens with concrete strength of 35 MPa are shown in Fig. 25. The purpose is to evaluate the effect of location of CFRP rods with 3 cases: 1) inside concrete part; 2) between concrete and repairing material; 3) inside repairing material. The distance from the NSM CFRP rod to longitudinal steel reinforcement was fixed with 100 mm. It is clear to detect the highest value of ultimate load about 220 kN can achieve with respect to slab SF5 which CFRP rods in repairing material. Meanwhile, slabs SF6 and SF4 obtained approximately 200 kN. This indicates that both these strengthening ways did not have any significant difference on the strengthening effectiveness. It may have been due to the bond behaviour between NSM CFRP rol and substrate, which mentioned above in the bonding test of C-shaped specimens. This is coincidence with the bond test where the highest bond stresses were obtained for CFRP rods embedded in the repairing material while the other two cases showed lower results.

Besides, another comparison is shown in Fig. 26 between slabs SF3 and SF4, which have the same strengthening design, but different concrete strength. The deflection of SF3 was smaller than SF4 as the load increased and even the maximum load of SF3 was also higher, although the concrete strength of slab SF3 was lower than SF4. On the other hand the test results of
control slabs SC1 and SC2 were quite similar due to steel yielding failure. These results indicate negligible influence of concrete strength in the case of rods totally embedded in the concrete groove, which showed a relatively low bond strength compared to specimens with rods in the repairing material.

In order to evaluate the strengthening effectiveness, the results of maximum load capacity and corresponding deflection are carried out comparing strengthened slabs with control slabs as shown in Fig. 27. All of strengthened slabs had higher maximum loads than the control slabs, while their deflections were smaller. On the contrary, the lowest load capacity value was evidently obtained for the case of fired-slab SF1 (101.4 kN) along with the highest deflection value (6.68 mm). As for the results of slabs with concrete grade of 24 MPa, slab SF2 achieved a highest load value of 214.4 kN equal to 2 times the value of control slab SC1. Moreover, slab SF2 also reached a higher maximum load value than SF3, which indicated that the position or distance from the NSM CFRP rod to steel bar was a factor that affected to the strengthening effectiveness.

Among the specimens with concrete strength of 35 MPa, the highest load value was reached for slab SF5
Figure 28 shows the typical results of strain 3.2.3 Strain distribution

tiveness of this technique on fire-damaged slabs. CFRP mental program was carried out to evaluate the effectiv-
FRP reinforcement method is introduced. An experi-
damaged flexural RC structures by using the NSM

4. Conclusions

In this paper, a new method to repair and strengthen fire-damaged flexural RC structures by using the NSM FRP reinforcement method is introduced. An experimental program was carried out to evaluate the effectiveness of this technique on fire-damaged slabs. CFRP rods, epoxy resin and repairing material were used according to three different ways to rehabilitate service ability of fire-damaged slabs whereby a comparison of effectiveness between these three ways was also estimated. In addition, the bonding behavior of NSM CFRP rods in case of using with concrete and repairing material were also evaluated through the bond test on C-shaped specimens. Based on the test results the following conclusions can be drawn:

1. The results from bond test show that the bond stress – slip law of NSM system in this study was similar to the bond law for the typical NSM. The bond stress – slip curves had an initial ascending branch and followed by a rather smooth softening branch after the peak point. However, the location of CFRP rod significantly influenced the bond behavior of the method.

2. In both cases NSM FRP rod in full depth groove and in a half depth groove, the bond stresses obtained were not much different and the highest values were below 3 N/mm². The latter could delay the slip at the initial low load level. Meanwhile, the bond stress with respect to the case which NSM rod was embedded in repairing material reached 3 times higher than the others.

3. The bond failure at the rod-epoxy/repairing material interface with cohesive shear failure in the epoxy/repairing material was observed as the critical failure pattern in all cases.

4. Based on the bond test results, the minimum embedded length for NSM rod to prevent pull-out failure in case of CFRP rod in repairing material is approximately 3 times shorter than the other cases.

5. In flexural tests, the bending cracks was observed in all slabs and shear cracks occurred at high load levels in case of strengthened slabs. The cracks propagated from the repairing material layer toward the concrete layer without any change in their angel, which indicated the good bond between two layers (concrete – repairing material).

6. The experimental results show that all strengthened slabs obtained good results over control specimens and better than fire – damaged slab. Their maximum loads were 2 times higher than control slabs while the deflections were smaller. The highest load capacity (226.9 kN) was reached in case of slab SF5 with CFRP rods embedded inside the repairing material. This location of CFRP rodd also brought out the highest results in bond test. Therefore, in comparing the three strengthening methods proposed in this study, the method in which the CFRP rods were installed inside the repairing material could be considered as the most effective. The considered method also offers advantages such as ease of installation and economy as epoxy resin was not used and it does not need to make grooves.

In a next stage of this research, the results from this experimental study will be used to verify a modeling...
work to predict the behavior of fire-damaged slab strengthened with NSM CFRP rods.

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