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Flexural Behavior of Corroded RC Members with Patch Repair – Experiments & Simulation

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Abstract

Structural performance of RC repaired by patching method was experimentally investigated and compared with non-corroded as well as corroded RC. Two repair materials; namely, polymer-modified mortar and epoxy-based repair material was applied for the repair work. The mechanical properties as well as the bonding characteristics of these two repair materials are different. It was found that the polymer-modified mortar can partially restore the structural performance while the more ductile epoxy-based repair material strengthening corroded RC structure so that its ultimate load carrying capacity is beyond that of non-corroded RC.

The numerical analysis was conducted to reproduce the structural performance of repair RC observed by incorporating experimentally measured properties of repair materials. The bonding characteristic between repair materials and base concrete is represented by basic Coulomb friction model. The proposed concept of analysis shows satisfactorily accurate results which match well with experimental findings.

1. Introduction

Deterioration of concrete structure is a serious problem which may drastically degrade both load-carrying capacity as well as serviceability of the structure. Among different deteriorations, the corrosion of reinforcing bar is one of the most widely found deterioration. The corrosion reduces cross sectional area of reinforcing bar as well as induces corrosion cracking which destroys the bonding between concrete and reinforcing bar. It was reported that, when degree of corrosion increases, the ultimate strength of steel bars as well as the corresponding elongation of the bar before failures decreases (Almusallam 2001). The bonding between reinforcing bar and concrete is affected by the corrosion. With relatively low level of corrosion, reinforced concrete exhibited better bonding characteristics; however, the bonding capacity decreases when weight loss is greater than 4% and the loss of bonding capacity is substantial when the corrosion cracks forms (Almusallam et al 1996). The corrosion can also severely degrade ultimate load carrying capacity as well as ductility of RC (Uomoto and Misra 1984). Toongernthong and Maekawa (2005) studied structural performance of corroded RC under shear load and reported that the loss of bond caused by corrosion crack may change failure mode of RC and capacity of anchorage performance as well as the strength in shear span. The brittle failure by buckling of compression reinforcement may also be induced if the corrosion of compression reinforcement takes place (Uomoto and Misra 1984).

Remedial action is necessary not only to maintain safety and serviceability of the deteriorated structure but also to extend its service-life. The patching repair method is recommended as an appropriate method for the RC members which are in the corrosion acceleration period as well as the deterioration stage (JSCE 2001). There have been some researches on the performance of RC in both structural as well as durability aspects (Shash 2005; Nounu et al 1999). It is generally accepted that the response of the repair system to the changes exerted by the repair must be understood for design and construction of durable concrete repair (Emmons and Vaysburd 1995). Satisfactory compatibility of repairing materials to base concrete structure in both structural performance and durability aspect must be ensured.

There are several factors affecting the performance of RC structure repaired by patching method. Effect of rebar cleanliness, surface preparation of base concrete, and mechanical properties of repair materials and patching area have been experimentally investigated (Al-Dulaijan 2002; Nagataki et al 1987; Nagataki et al 1990). The structural performance of RC repaired by patching method is highly related to the mechanical properties of repair materials as well as their bonding characteristics with base concrete. In addition, it is expected that the structural behavior of RC repaired by patching method changes with the geometry of the repaired area. However, there is still no general methodology to analyze the structural behavior after repair precisely.

This study is therefore an attempt to investigate the
structural performance of RC repaired by patching method as well as the mechanical properties of repair materials and conduct some numerical analysis to reproduce the observed performance by finite element method incorporating bond element. Corrosion of reinforced concrete specimen is induced by galvanic acceleration process. The repair procedure is controlled to be as similar to the practical operation as possible. Two different repair materials; namely, polymer-modified mortar and epoxy-based repair material was selected for the study since these two materials are commercially available and possess different mechanical properties.

2. Methodology

2.1 Experimental plan

Flexural behavior of corroded RC members and repaired members are investigated and compared with the flexural behaviors of control RC specimens. The tested specimens consisted of two undamaged RC members, three corroded RC members and six repaired RC members. Specimens were initially given a load until transverse cracking takes place. The corrosion was induced by accelerated galvanic corrosion until longitudinal corrosion cracks became observable. The corrosion acceleration process was stop when the degree of corrosion was approximately 2.5%. The corroded specimens were then repaired by patching repair method. There are three patching lengths studied in this experiment, namely, short patching (300 mm), long patching (1000 mm), and full span (1700 mm). Subsequently non-corroded, corroded, and repaired specimens were experimentally loaded to investigate their structural performance. The details of each process are given in following sections.

Fig. 1 Drawings of specimen & arrangement of reinforcement.

Table 1 Details of reinforcement.

<table>
<thead>
<tr>
<th>Reinforcement Ratio (%)</th>
<th>Area of main steel (mm²)</th>
<th>Area of Shear Reinforcement (mm²/m)(A_s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual</td>
<td>Balance</td>
<td>Tens. (A_t)</td>
</tr>
<tr>
<td>1.67</td>
<td>3.95</td>
<td>402</td>
</tr>
</tbody>
</table>
specimens. Lengths of corrosion zone or repair area were varied as 300 mm, 1000 mm, and full length (1700 mm). Two types of repair materials; i.e., polymer modified mortar and epoxy based repair material, were applied in this study.

2.3 Materials
Ordinary Portland cement-only concrete with water to cement ratio of 0.49 was used. The mix proportion is given in Table 3. The longitudinal reinforcement and shear reinforcement are deformed bar (Grade SD40) and round bar (Grade SR24), respectively. The tensile mechanical properties of reinforcing bars were obtained experimentally. Two types of repair materials – polymer modified mortar and epoxy based repair material were used for patching repair. Polymer modified mortar (repair material type I) contains cement and silica powder as important ingredients. Its behavior is therefore similar to cement paste with high early strength. The mix proportion of polymer modified powder to water was 4:7 by weight. The epoxy based repair material (repair material type II) provides outstanding bonding capacity and lower modulus. It is therefore generally recommended as an adhesive to bond most surfaces in general structure. The epoxy based repair material has the ratio between epoxy and sand of 4:3 by weight.

2.4 Additional test for rate of corrosion
In order to accelerate the corrosion of RC specimen into the former acceleration state precisely, additional test for calculating the amount of electrical charges (coulombs) that will activate the corrosion as well as for determining relationship between electrical charges supplied with degree of corrosion. The specimen in this accelerated corrosion test is shown in Fig. 2. The specified portion of specimen is submerged in the 5.0% NaCl solution and supplied with 100 mA direct current where the positive side is connected to reinforcing bar and the negative voltage is connected to stainless steel plate beneath the NaCl solution. The submerge area is the 30-mm central part of the specimen. The setting of this test is shown in Fig. 3.

During the testing on the rate of corrosion, the state of deterioration including crack pattern, rust exudation. After the corrosion acceleration process, the specimen was examined for distribution of corrosion cracks on the surface of specimen. The corrosion cracks width were also measured using crack measuring microscope. The specimen was split subsequently in order to take the corroded rebar. The corroded rebar was cut at the specified length and was cleaned with 10% di-ammonium hydrogen citrate solution along with 150 ppm of Thioglycolic acid solution. Exact length and weight of...
cleaned corroded rebar was measured. Finally, the weight loss was calculated based on the weight of rebar with the same length. The weight lost of reinforcement can be related with the corrosion current density applied to the specimen based on the Faraday’s law as follows:

$$\delta = \frac{Ait}{ZF}$$

where:
- $\delta$ : lost of metal (cm)
- $A$ : atomic weight of iron (56 g)
- $i$ : corrosion current density (Amp/cm$^2$)
- $t$ : time elapsed (seconds)
- $Z$ : valency of the reacting electrode (iron), commonly taken as 2
- $F$ : Faraday’s constant (96,500 Amp-seconds)
- $\gamma$ : density of material (iron = 7.86 g/cm$^3$)

From the experimental investigation, it was found that the galvanic corrosion acceleration by direct current of 100 mA for 192 hrs induces the actual weight loss of 0.30 gram/cm$^2$/cm which is slightly lower than the predicted corrosion which is 0.4156 gram/cm$^2$/cm (2.63 %). It was also found that the total length of corroded portion is approximately 200 mm. In the other words, under this configuration, the corrosion can be induced as far as 85 mm from the submerged zone. This information was then used in the corrosion acceleration of flexural member.

2.5 Corrosion acceleration of flexural member

From the data obtained from the test, the time that each flexural RC specimen must be subjected to galvanic corrosion acceleration process. In order to induce the corrosion of the RC member, cracking was initially introduced into specimen by slow loading. The location of cracks was ensured to be in the predetermined location. Three points loading was applied to induce crack in the specimen to have shortest repair region (FCB1R-O, FCB1R-O, and FCB1R-O) and full length (FCB1R-I, FCB1R-I, and FCB2R-I) and four-point loadings with constant moment span of 350 mm and 500 mm was applied to the specimen to have a repair region of 1000 mm (FCB1R-O, FCB1R-O, and FCB1R-O) and full length (FCB1R-I, FCB1R-I, and FCB2R-I), respectively. All induced cracks had a specified depth of 50 to 60 mm from bottom surface. Examples of flexural cracks induced by pre-cracking process are shown in Fig. 4. The cracked specimens were connected to the direct current supply and the corrosion was accelerated. The setting of equipments is shown in Fig. 5. The direct current supplied for specimens and accelerated periods were set from the calculation for each case so that acceleration period was shorter than 7 days.

In order to confirm the level of corrosion, the actual corrosion level of corroded RC specimen was measured after the loading. The corroded tensile reinforcements were taken out by cutting out of corroded reinforcement with specific lengths (200 mm for shortest corrosion length and 300 mm for medium and full corrosion length). Actual weight losses of the taken corroded reinforcement samples were then calculated based on original weight of reinforcement with same length. The summarized detail of setting as well as actual measured degree of corrosion is shown in Table 4. The actual weight losses of reinforcement of all specimens were from 0.3 to 0.4 g/cm$^2$/cm. The patterns of induced corrosion cracks are also shown in Fig. 6. Corrosion cracks were observable on the sides as well as at the bottom of specimens.

It is noted that the accelerated corrosion induced to

Fig. 3 Experimental setting for test of rate of corrosion (Unit: meters).

Fig. 4 Examples of pre-cracked specimens.
the specimens is not exactly the same with the corrosion in actual circumstances. The difference is mainly the uniformity of the corrosion. In real situation, some portion of reinforcing bar acts as anode and the other acts as cathode. There is therefore high possibility of pitting corrosion especially in the case of chloride attack. On the other hands, the accelerated corrosion in this study is more uniform since all position on reinforcing bar is forced to be anode and cathode is located at the stainless steel plate. Previous study (Oyado et al. 2006) showed that, at the same corrosion ratio, RC in real corrosive circumstances usually gives lower ultimate strength when compared with RC subjected to accelerated corrosion.

2.6 Patching repair procedure
Six corroded specimens (FCB1R-I, FCB1R-O, FCB1R-F, FCB2R-I, FCB2R-O, and FCB2R-F) were repaired by patching repair method. The patching regions were 300 mm, 1000 mm and 1700 mm (full span) for specimens with 120 mm, 820 mm, and full span corrosion acceleration, respectively. Initially, the damage portions of concrete were removed by a portable demolisher powered by high pressure pump. The reinforcements were then polished carefully by the electrical polishing tools equipped with steel brush. Chisel steel rods were then used to trim the area to be repaired into rectangular shape with uniformly rough surface. The patching areas were then cleaned by high pressured air activated by air pump. The specimens were flipped so that the patching area is on the top during patching in order to fill the repairing materials in to the patching area efficiently as well as to ensure a quality of compaction. After patching, the patched portions were wrapped by the plastic sheets in order to prevent the early lost of water from the repair materials. Special attention was given to the epoxy-based repair material which has a very short setting time. After the repair material set, the surface of the repaired region was then trimmed to have an exact dimension as original size of RC specimens.

2.7 Flexural loading test
The non-repaired RC flexural members (NCB-1 and NCB-2) were loaded at the age of 28 days. The corroded
RC members (FCBNR-I, FCBNR-O, FCBNR-F) were loaded at 7 days after galvanic corrosion acceleration was completed. And, the repaired RC members were loaded at 14 days after repair work. All specimens were loaded under 4-points loading with a total clear span (distance between supports) of 1500 mm. The distance between two loading points and shear span were 500 mm. Displacement transducers were installed at the middle of the specimens, at the location of loading points, and at the supports in order to measure the deflection of the specimen during the loading. Monotonic loading was slowly applied to specimen. The load cell was installed in order to measure the exact load applied on the specimen and cracking observations were performed every 5 kN of load. The loading continued until the specimens failed.

2.8 Testing for mechanical properties of materials

Measurements of mechanical properties of concrete and repairing materials were conducted in accordance with the standard guidelines. Table 5, 6, and 7 show lists of tests conducted in order to measure mechanical properties of concrete, repairing materials, and reinforcement, respectively. The tests on concrete were conducted at 28 days while the tests on repairing materials were conducted at 7 days and 14 days. Strains were measured on both sides of specimen and the average value is used to determine modulus of elasticity and Poisson ratio. Special note is given to the measurement of tensile strength of concrete. The pull-out test was conducted instead of other standard testing methods since the pull-out test can simulate the condition of concrete in tension more closely.

Table 5 Tests on mechanical properties of concrete.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Description</th>
<th>Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength</td>
<td>Compression Test on Cylinder Specimen with strain gauges</td>
<td>150 mm diameter &amp; 300 mm height</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>(ASTM C-39, C-192, C-469)</td>
<td></td>
</tr>
<tr>
<td>Poisson Ratio</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>Tension pull-out test</td>
<td>100x100x100 mm³ cube</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50 mm diameter core drills cut</td>
</tr>
<tr>
<td></td>
<td></td>
<td>through concrete surface, penetrating</td>
</tr>
<tr>
<td></td>
<td></td>
<td>at least 55 mm</td>
</tr>
</tbody>
</table>

Table 6 Tests on mechanical properties of repairing materials.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Description</th>
<th>Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength</td>
<td>Compression Test on Cylinder Specimen with strain gauges</td>
<td>50 mm diameter &amp; 100 mm height</td>
</tr>
<tr>
<td>Elastic Modulus</td>
<td>(ASTM C-39, C-192, C-469)</td>
<td></td>
</tr>
<tr>
<td>Poisson Ratio</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile-Strength and Stress-Strain</td>
<td>Flexural test under loading at the center of specimen. Strain</td>
<td>75 mm depth, 50 mm width, and 400 mm length</td>
</tr>
<tr>
<td>Relationship</td>
<td>gauges were attached at top, bottom, and mid-depth of the</td>
<td></td>
</tr>
<tr>
<td></td>
<td>specimens (ASTM C293)</td>
<td></td>
</tr>
</tbody>
</table>
materials, the testing on bonding between concrete and each type of repairing material (polymer-modified mortar and epoxy-based repairing material) was conducted. The objective of the bond testing is to determine the bonding capacity under tension and under combination of shear and axial load. The tension bond test was conducted to find the maximum stress that the contact interface can withstand before the separation takes place. In the case of shear bonding capacity, both direct shear test and combined compression-shear test were conducted in order to investigate the dependency of shear capacity of bonding surface on the perpendicular stress.

Table 8 describes details of direct tension test, direct shear test, and combined shear compression test on the contact interface between concrete and repairing material. Five different contact angles were applied in the combined shear compression test (Table 9). Figure 7 illustrates the testing condition of each bond test.

3. Experimental results

3.1 Mechanical properties of materials

Table 10 shows the mechanical properties of concrete, polymer modified mortar, and epoxy-based repair materials. The mechanical properties of polymer-modified mortar are similar to those of concrete. The tensile strength is approximately double of concrete and the modulus of elasticity is lower by 40% when compared with concrete. The epoxy-based repair material shows an outstanding record in tensile strength (approximately 8 times of concrete’s tensile strength). However, the modulus of elasticity is very low (0.1 of modulus of elasticity of concrete).

Yielding strengths of RB6, DB12, and DB16 were 280, 540 and 600 N/mm², respectively. The modulus of elasticity of all reinforcement is 206,000 N/mm².

Table 9 Dimensions (angle of contact interface and height) of specimens in combined shear-compression test.

<table>
<thead>
<tr>
<th>Angle (degree)</th>
<th>Height of contact interface (mm)</th>
<th>Total Height of Specimen (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>130</td>
<td>150</td>
</tr>
<tr>
<td>45</td>
<td>75</td>
<td>95</td>
</tr>
<tr>
<td>50</td>
<td>63</td>
<td>83</td>
</tr>
<tr>
<td>55</td>
<td>53</td>
<td>73</td>
</tr>
<tr>
<td>60</td>
<td>45</td>
<td>65</td>
</tr>
</tbody>
</table>

Table 8 Tests on bonding properties between concrete and repairing materials.

<table>
<thead>
<tr>
<th>Testing Method</th>
<th>Description</th>
<th>Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension Bond Test</td>
<td>Tension pull-out test</td>
<td>100x100x100 mm³ with 50-mm core drill cut through repair material penetrating at least 5 mm into concrete stratum</td>
</tr>
<tr>
<td>Combined shear compression test</td>
<td>Compressive test carried out on prism with one-half of concrete replaced with repair material</td>
<td>75x75 mm³ with different heights according to the angle of contact. Concrete prism cut at an angle of 30, 45, 50, and 60 degree</td>
</tr>
<tr>
<td>Direct shear test</td>
<td>Test for shear capacity of the contact interface when the compressive forces is applied on the top</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 7 Tests on bonding properties of interface between concrete and repair material.
3.2 Structural behaviors & crack patterns

3.2.1 Non-corroded flexural RC specimens (NCB-1 and NCB-2)
Cracking loads were between 15-20 kN. Flexural cracks form initially in the constant moment span followed by the formation of shear cracks. Both NCB-1 and NCB-2 collapsed by flexural-compression failure. The loading capacity of NCB-1 and NCB-2 were 145.97 kN and 144.99 kN, respectively. The final crack pattern of NCB-2 is shown in Fig. 8.

3.2.2 Corroded flexural RC specimens
Three corroded RC specimens with different corrosion lengths were tested under flexure. Slight reductions of ultimate loading capacity were observed in all corroded RC specimens. The ultimate load of FCBNR-I, FCBNR-O, and FCBNR-F were 137.34 kN, 138.51 kN, and 139.4 kN, respectively. The load-deflection relationship of NCB-1, FCBNR-I, FCBNR-O, and FCBNR-F is given in Fig. 9. The reduction in ultimate loading capacity is approximately 4.5% to 6% when compared with non-corroded RC specimens. The loss of cross sectional area of tensile reinforcement is a main reason of the lower ultimate loading capacity. It could be observed that the stiffness of all corroded specimen is significantly degraded mainly because bonding between tensile reinforcement and concrete was destroyed. However, final crack patterns of FCBNR-I (shortest corrosion length) was very similar to the final crack pattern of non-corroded specimen (see Fig. 10). In the case of FCBNR-F. Propagation of shear cracks was disturbed by the corrosion crack. As the result, no traditional shear cracks was observable in this case. The shear crack which started from support was trapped by corrosion crack. (Fig. 10c); however, this mechanism did not have any significant effect on load carrying capacity of the beam. The absence of shear crack may be the reason why the deflection of FCBNR-F is smaller than that of FCBNR-O at the same load.

3.2.3 Specimens repaired with polymer-modified mortar
All corroded specimens repaired by polymer-modified mortar (FCB1R-I, FCB1R-O, and FCB1R-F) were cracked before the loads reached 10 kN which is lower than cracking load of non-corroded flexural RC specimens. This lower cracking load may be a result of shrinkage of the polymer-modified mortar itself. The shrinkage seems to affect the structural behaviors of RC specimens.

Table 10 Mechanical properties of materials (28 days for concrete and 14 days for repair materials).

<table>
<thead>
<tr>
<th></th>
<th>Compressive Strength (N/mm²)</th>
<th>Tensile Strength (N/mm²)</th>
<th>Modulus of Elasticity (N/mm²)</th>
<th>Poisson Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete (28 days)</td>
<td>43.00</td>
<td>3.70</td>
<td>35,000</td>
<td>0.22</td>
</tr>
<tr>
<td>Polymer-Modified Mortar (7 days)</td>
<td>35.26</td>
<td>6.13</td>
<td>16,500</td>
<td>0.16</td>
</tr>
<tr>
<td>Polymer-Modified Mortar (14 days)</td>
<td>42.43</td>
<td>8.14</td>
<td>20,250</td>
<td>0.22</td>
</tr>
<tr>
<td>Epoxy-Based Material (7 days)</td>
<td>34.08</td>
<td>29.25</td>
<td>3,421</td>
<td>0.35</td>
</tr>
<tr>
<td>Epoxy-Based Material (14 days)</td>
<td>36.32</td>
<td>30.13</td>
<td>1,313</td>
<td>0.34</td>
</tr>
</tbody>
</table>
specimens repaired with polymer-modified mortar.

Figure 12 shows the comparison between the load-deflection relationships of flexural RC specimens repaired with polymer-modified mortar and that of non-corroded RC specimen (NCB-1). The specimen with shortest patching area (FCB1R-I) shows very similar structural behavior to the control RC specimen regardless of lower cracking load. Figure 11a shows the final crack pattern of FCB1R-I. It was found that the first flexural crack was formed at the interface between polymer-modified mortar and base concrete. One additional crack was formed at the middle of repair material. These cracks propagated vertically toward neural axis of the section without any horizontal cracks observable at the interface between repair material and base concrete. The shear crack formation of FCB1R-I is similar to non-corroded RC specimen.

In the case of FCB1R-O of which the patching area is 1000 mm, the first flexural cracking took place at the vertical interface between polymer-modified mortar and base concrete although the flexural bending moment is smaller than the middle of the span (Fig. 11b). The sub-
sequent crackings within the patching area were observed. When the loading continued, the cracks in the patching show larger crack growth. Discontinuity of the vertical bending cracks at the horizontal interface was observable. The shear crack was subsequently developed in the shear span and there was a clear interface failure caused by the excessive shear force in the interface (Fig. 11b). The interface cracks was connected with flexural cracks in repair material and propagated along the vertical interface at the end of patching area subsequently. Later on, additional shear crack (outermost) formed when the load increased and followed by the crushing of concrete in compression zone.

The similar interface failure was also observed in FCB1R-F (full span repair with polymer based material). It was found that the number of shear cracks in shear span increased in this case and there was no formation of larger shear crack at all. This is because the repair interface was extended to the end of the specimen. Final crack pattern of FCB1R-F is shown in Fig. 11c.

The ultimate loading capacity of FCB1R-I, FCB1R-O, and FCB1R-F were 143.42 kN, 143.23 kN, and 139.40 kN, respectively (Fig. 12). The stiffness of FCB1R-I is almost same with non-corroded RC specimen (NCB-1). This indicates that the loss of cross sectional area may not significantly affect the drop of stiffness if the bonding is restored by the repairing material. However, the stiffness of FCB1R-O and FCB1R-F was decreasing when load increased, especially beyond 100 kN. This loss of stiffness is caused by the shear cracks as well as the interface failure in shear span.

3.2.4 Specimens repaired with epoxy-based repair material

Figure 13 shows the final crack patterns of the specimens repaired with epoxy-based repair material (FCB2R-I, FCB2R-O, and FCB2R-F) and their load-deflection relationships are given in Fig. 14. It should be note that, before the first cracking, the deflections of NCB and FCB2R-I were smaller than those of FCB2R-O and FCB2R-F under the same load because the smaller modulus of elasticity of epoxy-based repairing material. The structural behavior of FCB2R-I is very similar with the control RC specimen because the patching area is shorter of the loading span (300 mm). The failure of FCB2R-I was governed by the compression failure of the concrete inside constant moment span but outside patching area. Since epoxy-based repairing material applied in this study is very ductile and strong in both tension and bonding. In FCB2R-O and FCB2R-F, the flexural cracks were formed in the base concrete without flexural cracks in the epoxy-based repair material (see Fig. 13b and 13c). A good cracking resistance of epoxy-based repair materials contributed to the stiffness of the member. The deflections of NCB and FCB2R-I therefore became larger than the deflections of FCB2R-O and FCB2R-F after flexural cracking. When
the load increased, distributed damage at the horizontal interface between epoxy-based repair material and base concrete could be observed in both FCB2R-O and FCB2R-F; however, it was observable that there is still some stress transfer across the interface. The bonding was not completely vanished by these small distributed cracks. At approximately 90 kN, large shear cracks formed in FCB2R-O (Fig. 13b) and hence the deflection of FCB2R-O was consequently increased. On the other hand, the formation of shear crack in FCB2R-F was prevented by the interface between concrete and repair material. As the results, several shear cracks were formed instead of a single large shear crack (Fig. 13c). FCB2R-O was failed by crushing of concrete after shear crack fully propagated while the failure of FCB2R-F was caused by a sudden rupture of epoxy-based material. The maximum loading capacity of FCB2R-I, FCB2R-O, and FCB-2R-F were 154.12 kN, 171.77 kN, and 184.53 kN, respectively.

Figure 15 shows the comparison among different RC specimens in the study. All corroded beams (FCBNR-I, FCBNR-O, and FCBNR-F) show a lower load carrying capacity when compared with non-corroded RC (NCB-1 and NCB-2) of which the load carrying capacity was approximately 145 kN. Patching repair with polymer-modified mortar was able to slight improve load carrying capacity of corroded RC; however, the load carrying capacity after repair is still less than the original capacity of RC. The loss of load carrying capacity is caused mainly by the loss of cross sectional area of reinforcement. It should be also noted that the load carrying capacity of the RC repaired with polymer-modified mortar reduced when the patching length increased in this study. This may be due to a larger effect of debonding between repairing material and base concrete in specimen with longer patching area.

The most interesting experimental result in this study is that the patching repair with epoxy-based modified mortar can restore or even strengthen the flexural capacity of RC. The flexural capacity was increased up to 27% in the case of full-span patching although there is no replacement of reinforcement. In addition, the ultimate capacity increased when the patching length was longer. By comparing FCB2R-O and FCB2R-F which have same cross section in the loading span, it can be concluded that the ultimate load carrying capacity of RC repaired with epoxy-based repair material is not controlled by the cross section at the maximum moment. The structural behavior of RC is influenced by the stress transfer in the shear span especially in the case that repair material is very ductile.

3.3 Bonding between based concrete and repair material
3.3.1 Direct tension bond test
Table 11 shows results of the direct tension bond test. Only the data from the specimens which failed by the interface failure was recorded. The bonding strength under direction of epoxy-based repair material is approximately 2 times to the polymer-modified mortar. In addition, there is not much difference between bonding strength at 7 days and 14 days of each material.
3.3.2 Direct shear and combined shear compression test

The experimental results of direct shear test and combined shear compression test are shown in Table 12 and Table 13, respectively. In the direct shear test, only data which was obtained from the interface failure was recorded while all data obtained from the combined shear compression test is used in this study although some specimens failed by the material failure. Both interface failure and material failure are taken as possible modes of failure in the actual structure and hence are considered in this study.

A so-called Coulomb friction model is employed to simulate the bonding behavior of the repair interface. Three important parameters, namely, friction coefficient ($\mu$), cohesion resistance ($\tau_0$), and maximum shear stress ($\tau_{\text{max}}$) can be obtained from the experimental results of the direct shear test together with the combined shear compression test. Cohesion resistance between concrete and repair material can be obtained from direct shear test with no axial stress. The friction coefficient can be determined from both direct shear test with an axial stress and combined shear compression test with relatively small angle. The maximum shear stress can be derived from the maximum shear stress when the normal stress is relatively high (combined shear compression test with larger angle). By applying Coulomb friction model, it is assumed that sliding occurs independent on normal pressure when shear stress reach maximum shear strength (Wrigger 2002).

Figure 16 and Fig. 17 show the comparison between model and experimental result. Main parameters of Coulomb friction model for both repairing material are given in Table 14. Both repairing materials have approximately similar cohesion resistance ($\tau_0$) but the fric-

<table>
<thead>
<tr>
<th>Repairing Material</th>
<th>$\sigma$ (N/mm$^2$)</th>
<th>$\tau$ (N/mm$^2$)</th>
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<td>Polymer-modified</td>
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<td></td>
<td>3.00</td>
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<tr>
<th>Repairing Material</th>
<th>Contact Angle (degree)</th>
<th>$\sigma$ (N/mm$^2$)</th>
<th>$\tau$ (N/mm$^2$)</th>
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<td>6.89</td>
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<tr>
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<table>
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<tr>
<th>Repairing Material</th>
<th>$\mu$</th>
<th>$\tau_0$ (N/mm$^2$)</th>
<th>$\tau_{\text{max}}$ (N/mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polymer-modified mortar</td>
<td>0.78</td>
<td>2.68</td>
<td>18.00</td>
</tr>
<tr>
<td>Epoxy-based repair material</td>
<td>1.35</td>
<td>2.98</td>
<td>19.00</td>
</tr>
</tbody>
</table>
tion coefficient ($\mu$) of epoxy-based repair material is substantially larger.

4. Numerical simulation – FEM analysis

4.1 Finite element model

A 3-dimensional finite element model is created based on a so-called discrete model concept. The reinforcement is modeled using line elements connected to nodes of concrete or repair materials. The analysis was performed primarily using available models and element formulations available in the finite element software ‘ANSYS’. Figure 18 shows finite element model with the information of mesh and boundary condition applied for non-corroded RC beam and repaired reinforced concrete beam. Taking an advantage from symmetry of the member, one quarter of specimen with appropriate displacement boundary condition was analyzed in this study (Fig. 19). At the plane of symmetry, the displacement in the direction perpendicular to the plane was held to zero.

Concrete and repair material have been modeled as a three-dimensional eight node solid isoparametric element in order to model the non-linear response of brittle materials based on a constitutive model for the triaxial behavior of concrete (Williams and Warnke 1975). The element includes a smeared crack analogy for cracking in tension zones and a plasticity algorithm to account for the possibility of concrete crushing in compression zone. The longitudinal and transverse reinforcement have been modeled as discrete reinforcements using three dimensional spar elements embedded within the solid mesh. The stress-strain response of reinforcement based on experimental results was applied.

In this study, each FEM model contains two top longitudinal reinforcements and two bottom longitudinal reinforcements. The reinforcements with diameter of 12 mm and 16 mm were located at 37 mm and 161 mm from the top of the model, respectively. Transverse reinforcement with 6-mm diameter was also provided in the model. Note that only half of the transverse reinforcement was modeled at the center of the beam due to the symmetry. Figure 20 illustrates the reinforcement model. Since the tensile reinforcement in corroded RC specimens is subjected to the loss of cross-section. The smaller area of the reinforcements was taken into account by reducing the cross sectional area of tensile reinforcement according to the experimentally measured weight loss. The perfect bonding is assumed for the interface between reinforcement and concrete or repair materials. Perfect bond can be reasonably assumed in this case because the bond model between rebar and concrete has only a minor effect on the tension stiffening which is already included with in fracture of concrete (in the case of discrete FEM) especially in the case of fine mesh element is applied (Jendele and Cervenka 2006).

The loading is applied by specifying a vertical dis-
placement to the node of the loading plate located on top of the model (see Fig. 20). Similarly, the model was supported by support plates. Loading plate and support plates were modeled by solid element with elastic material model. This strategy was used instead of constraining vertical displacement of the beam nodes in order to avoid stress concentration problems. The summation of the reaction forces on these plates constitutes a load applied to the beam.

4.2 Material models
4.2.1 Concrete material model
Concrete is modeled by the solid element ‘Solid 65’ available in ANSYS. The element behaves in a linear elastic manner until either of the specified tensile or compressive strength is exceeded. After cracking, the element becomes orthotropic and has stiffness based on a bilinear softening stress-strain response. After crushing, the concrete is assumed to lose its stiffness in all directions. The failure surface of the element is computed based on the model proposed by William and Warnke (1975).

The tensile stress-strain relationship shown in Fig. 21 is applied to plain concrete. The post-cracking tensile stiffness of concrete is accounted for when the strain is less than six times of the strain corresponding to the peak stress. $R_t$ is the secant modulus which work with adaptive descent and diminishes as the solution con-
verges. And $T_c$ is the multiplier for amount of tensile stress relaxation and was taken as 0.6 in this study. The tensile strength and strain corresponding to maximum stress were set as 3.5 N/mm$^2$ and 194 $\mu$, respectively.

The compressive stress-strain relationship for concrete in this study follows the formula recommended by CEB-FIB model code (1990). Compressive strength, tensile strength, and modulus of elasticity were taken as 43 N/mm$^2$, 3.7 N/mm$^2$, and 35,000 N/mm$^2$ accordingly to the experimental results. In the FEM analysis, the CEB-FIB model was represented by multilinear stress-strain relationship as shown in Fig. 22. A shear transfer coefficient is also introduced in order to represent a shear strength reduction factor for those subsequent loads which induce sliding (shear) across the cracked plane. The coefficient for open cracks ($\beta_t$) should be in the rage of 0.05-0.5, rather than 0.0, in order to prevent numerical difficulties (Hemmaty Y. 1998). In this study the coefficient for open cracks ($\beta_t$) was set at 0.25 (Job T., and Anath R. 2006) while the coefficient for closed cracks was assumed to be 0.7.

4.2.2 Material model for repair material

Special attention was paid on the mechanical properties of repair materials. The stress-strain relationships were obtained experimentally and were slightly modified into multi-linear relationship (Fig. 23 and Fig. 24). Note that the stress-strain relationship at 14 days was selected as an input for FEM analysis. The polymer modified mortar behaves very similarly with concrete and the tensile response was almost linear although the cracking strain capacity or crushing strain was much higher to those of normal concrete. The response of epoxy-based repair material was nonlinear under both tension and compression and shows remarkable ductility. Very large tensile cracking capacity (12,000 $\mu$) is the reason why the RC specimen repaired with epoxy-based material has no flexural cracks formed in the patching area. The shear transfer coefficients for both repair materials were set at 0.55 for closed crack condition ($\beta_c$) and 0.15 for open crack condition ($\beta_t$). These coefficients are lower than coefficients of concrete since the maximum size of aggregate is lower (no coarse aggregate). Poisson ratio and modulus of elasticity applied in the numerical analysis were taken from experiments and is available in Table 10.

---

Fig. 21 Stress-strain response of concrete in tension-idealization used in ANSYS.

Fig. 22 Compressive stress-strain relationship of concrete employed in FEM analysis.

Fig. 23 Stress-strain relationship of polymer-modified mortar.

Fig. 24 Stress-strain relationship of epoxy-based repair material.
4.2.3 Stress-strain relationship of the reinforcement
Stress-strain relationship of reinforcement was modeled to be rate independent with multi-linear isotropic hardening with von-Mises yield criterion. By fitting the model with experimental data, the tensile stress-strain responses of all reinforcement were derived as shown in Fig. 25. Modulus of elasticity and Poisson’s ratio were taken as 206,000 N/mm² and 0.3 for all reinforcing bars.

4.3 Contact interface between concrete and repair material
Two modes of local failure at the interface between concrete and repair material, namely, crack opening due to tensile stress (Mode I) and fracture due to shear stress (Mode II) were considered in the numerical analysis. To simulate crack opening due to tensile stress (Mode I), non-linear spring element was employed while the contact element was used to simulate interaction between shear and compressive stress at the interface (Mode II). The contact element contains a constitutive law which is expressed in terms of the contact tractions and relative displacements of two surfaces.

4.3.1 Modeling of mode I tension softening
Bilinear stress-crack opening diagram for concrete according to CEB-FIP model code 1990 was adopted in order to capture mechanical behavior of contact interface under tension. The model considers the softening of interface response after the formation of surface crack. The bilinear stress-crack opening diagrams can be obtained as shown below;

\[ \sigma_c = f_c \left( 1 - 0.85 \frac{w}{w_i} \right) \quad \text{for} \quad 0.15 f_c \leq \sigma_c \leq f_c \]  

(2)

\[ \sigma_c = \frac{0.15 f_c}{w_i - w_i} (w_i - w) \quad \text{for} \quad 0 \leq \sigma_c \leq 0.15 f_c \]  

(3)

With \( w_i = \frac{2G_f}{f_c} - 0.15w \) \( \frac{G_f}{f_c} \) is the fracture energy (N-mm/mm²)

\( f_c \) is the tensile strength (N/mm²)

\( \alpha_c \) is the coefficient which depends on the maximum aggregate size (taken as 8 for polymer-modified mortar and 7 for epoxy-based repair material)

The fracture energy \( (G_f) \) is the energy required for propagation of tension crack of a unit area and can be estimated as following;

\[ G_f = G_{fo} \left( \frac{f_c}{f_{0c}} \right)^{0.7} \]  

(6)

where,

\( f_c \) is the compressive strength of concrete (N/mm²)

\( f_{0c} \) is 10 N/mm²

\( G_{fo} \) is the base value of fracture energy which can be determined based on maximum aggregate size (taken as 0.025 for polymer-modified mortar and 0.030 for epoxy-based repair material) (N-mm/mm²)

Table 15 shows parameters used to defined the force-deformation response of non-linear spring element. The values at 14 days were employed in FEM analysis. Tension softening model corresponding to these parameters are shown in Fig. 26.
4.3.2 Modeling of mode II relative displacement due to shear stress
Constitutive law of the repair interface under shear stress is expressed in terms of the contact tractions and relative displacement of two surfaces. The basic Coulomb friction model was adopted to simulate the response of interface in this mode. In the model, it is assumed that contact interface can carry shear stress up to a certain magnitude before sliding starts. The Coulomb friction model defined a shear stress \( \tau \) at which sliding on the interface begins as a linear proportional function of the contact pressure \( \sigma_n \) with maximum limit \( \tau_{\text{max}} \) which is provided so that calculated shear stress does not exceed the yield stress in the interface subjected to high contact pressure. Once shear stress limit is exceeded, slip takes place between the two surfaces. The model can be expressed in numerical formula as follow:

\[
\tau = \tau_s + \mu \sigma_n \leq \tau_{\text{max}},
\]

\[
\tau_{s,i} = \begin{cases} 
    k_i u & \text{if } \tau < \sqrt{\tau_s^2 + \tau_t^2} - \mu \sigma_n \text{ (sticking)} \\
    \mu \sigma_n & \text{if } \tau = \sqrt{\tau_s^2 + \tau_t^2} - \mu \sigma_n \text{ (sliding)}
\end{cases}
\]

where,
- \( \tau_s \): Cohesive sliding resistance (N/mm²)
- \( \mu \): Frictional coefficient
- \( k_i \): Tangential contact stiffness
- \( u \): Contact slip distance in \( y \) or \( z \) direction

The values for main parameters in this basic Coulomb friction model were obtained experimentally and available in Table 14. The integration of the friction mode is a nonassociated theory of plasticity. In each step that sliding friction takes places, and elastic predictor is computed in contact traction space. The predictor is modified with a radial return mapping function, providing small deformation along sliding response.

4.4 Results of FEM numerical simulation
The FEM numerical simulation incorporating loss of reinforcement as well as response of repair interface was conducted for all specimens. The numerical analysis of corroded RC repaired by patching method gives a peak load as shown in Fig. 27. The result shows that the FEM analysis can be used to predict the load carrying capacity of the repaired structure with acceptable accuracy when the model of interface response and precise model of repair material is incorporated.

5. Discussion
As shown by the experimental results, the structural behaviors of repaired reinforced concrete beams are dependent on mechanical properties as well as the size and location of repaired portion. In general, if the repaired portion is inside the constant moment span, the flexural capacity of the repaired member is similar to that of the control specimens because it is controlled by the crushing failure of concrete inside constant moment span. However, when very ductile repair material is applied, the flexural capacity of the repaired member is increased because of cracking in tensile area can be effectively prevented until the failure of member. When the maximum flexural capacity is increased, the possibility of shear failure is questionable. In fact, the structural performance of repaired reinforced concrete beams
changed substantially when the section subjected to shear force is also repaired. Although the crack patterns give some clues how the mechanism of repaired beam changed, the information of stress field inside specimen could not be precisely measured in the experiment. Fortunately, the FEM analysis conducted in this study show the accuracy of applicable level for design work and some mechanisms of repaired beams can be captured by FEM analysis.

The explanation of how the stress field in shear span is changed by the length of repaired portion can be explained most clearly by the comparison between FCB2R-O and FCB2R-F. Figure 28 shows the responses of FCB2R-O observed during and experiment and obtained by FEM analysis. Although FEM analysis could not precisely simulate non-linearity of FCB2R-O and gave a higher deflection at a specific load, it showed a good agreement with experimental finding on cracking pattern that no crack was formed in the patching area.

Attention should be paid to a sudden change of stiffness is observable at 150 kN (in FEM response) which is very close to the formation of diagonal shear crack observed in the experiment. Both the principle stress distribution (Fig. 29) and crack distribution (Fig. 30) show that considerable damage is located in shear span at 150 kN. It is therefore concluded that the loss of stiffness is caused by shear crack formation.

Figure 31 shows the load-deflection relationship of FCB2R-F obtained from experiment and FEM analysis. Unlike the case of FCB2R-O, the sudden change of stiffness was not observable because the existence of repairing interface prevents the formation of diagonal shear crack. Instead of large diagonal shear crack, damage is distributed throughout the shear span of FCB2R-F. Figure 32 shows the crack distribution of FCB2R-F which agree very well with multiple shear cracks found in the experiment (see Fig. 13). The clear difference of damage localization can be observed (Fig. 30 and Fig. 32).

In the case that repair material is not ductile and has comparatively low bonding strength, special attention should be paid to the interface failure. Additional consideration about volume change of repair material should also be taken into account. Figure 33 shows the experimental data of FCB1R-F and the computational result. Very large discrepancy could be observed when the load was below 40 kN. This discrepancy is due to the fact that the shrinkage of the polymer-modified mortar was not considered in the numerical analysis and thus the cracking load could not be correctly calculated. However, both results show a good agreement when the load is beyond 40 kN. This indicates that FEM can predict flexural deflection of flexural RC repaired by polymer-modified mortar accurately. Nevertheless, unlike the case of ductile repair material, since the interface failure is likely in the case of polymer-modified mortar, special attention should therefore be paid to the controlled of deflection as well as other serviceability requirements.

Based on the experimental observation and numerical analysis, it is recommended that the influence of repair
material should be checked carefully in design stage. Since the mechanism of repaired beams varies based on properties of repaired material and location and size repaired portion, different type of failures is expectable for each repaired beam. This may be the reason why standard design guideline for patching repair cannot be easily proposed. However, this study shows that, with understanding on mechanical properties and application of FEM, the most likely failure mechanisms can be predicted with acceptable accuracy.

Although, as shown by the comparison between experimental data and numerical simulation in Fig. 27, the peak loads obtained from the numerical simulation were considerably close to the real loading capacity of the members, the applied numerical method was still unable to simulate the behavior after peak load. The authors expect that this limitation of the method may be caused by divergence problem when applying the proposed model with ANSYS. This limitation of the method should be solved in the future study.

6. Conclusion

Based on experimental observations and numerical simulation of repaired RC member, major findings can be concluded as follows:

1. The structural behaviors of corroded RC specimens repaired by patching method are different based on both mechanical properties of repairing materials and patching area. However, in all cases, the repairing improves the structural behaviors when compared with corroded RC without repair.

2. When polymer modified mortar is used as a repairing material, the failure of interface between repair materials is very likely to take place in the shear span. The interface failure disturbs the stress distribution in shear span as well as reduces overall stiffness of the repaired RC.

3. Since the epoxy-based repair material is very ductile under tension, the flexural cracking was not observable in the patching area experimentally. The interface failure could also be successfully prevented by a better bonding characteristic which change brittle interface failure into distributed tiny cracks around the interface.

4. The load carrying capacity of RC repaired by epoxy-based repair materials is higher than that of non-corroded RC. And the improvement of load carrying capacity is greater for the longer patching area.

5. By incorporating the bonding between repairing material and based concrete using tension softening model and basic Coulomb friction model, the finite element analysis can give the sufficiently accurate prediction of ultimate load carrying capacity. However, the consideration of shrinkage of polymer modified mortar should improve the calculation.

6. FEM analysis can successfully give information of stress distribution in repaired RC specimen and it is shown that the different patching area causes a specific stress distribution. For instance, in the case of epoxy-based repair materials, the patching length of 1000 mm caused a localization of stress in shear span and subsequently generated the large diagonal shear crack while the full-span patching helps in distributing stress over the shear span as well as preventing formation of diagonal shear crack.

References
concrete repairs.” *Construction and Building Materials*, 10(1), 69-75.


