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Finite element investigation of the flexural behavior of corroded RC beams before and after repairing

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ABSTRACT: It is necessary to determine the flexural behavior of existing deteriorated beams before and after repairing for reducing the cost of repair operations in concrete structures. This paper presents the numerical investigation of the cracking load and load-carrying capacity for sound, deteriorated and repaired beams as well as the debonding tendency for repaired members. Results showed that the load of cracking and ultimate load capacity of deteriorated beams depends on the loss of concrete cover and degree of reinforcement corrosion, respectively. In repaired beams, the load of cracking depends on the mechanical properties of repair materials and substrate and the debonding behavior of repaired area depends on the mechanical properties together with the bond strength between repair concrete and substrate.

1. INTRODUCTION

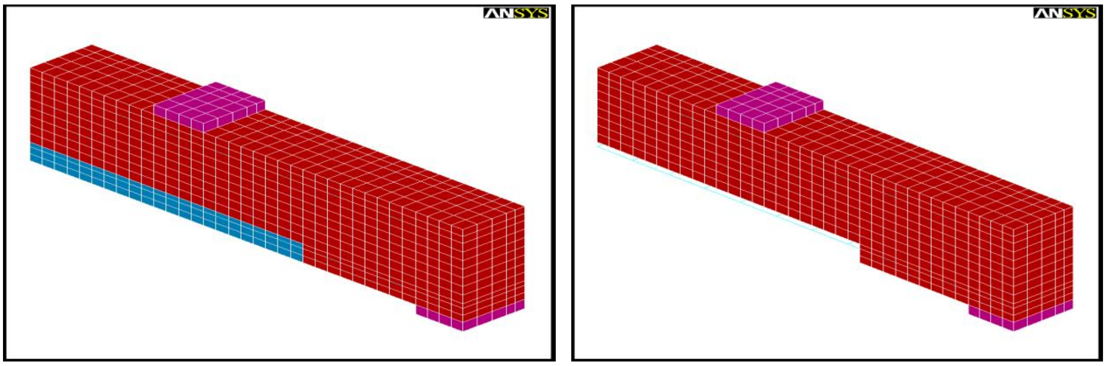
Extensive degradation of reinforced concrete (RC) structures around the world especially in harsh environmental regions have recently become of great concern. Reports show that approximately £500 million is spent annually on the repair of reinforced concrete structures within the UK (Swiss Bank Corporation (1989)). Also every year in North America, billions of dollars are spent to repair bridge deck delaminations (Lachemi et al. (2007)). So, it is necessary to carry out effective structural repairs for restoring or increasing the service life of deteriorated structures and eliminating the need for reconstruction or ‘repair the repairs’ (Wood (2009), Vaysburd et al. (2004), Vaysburd et al. (2000) and Soleimani et al. (2010)).
One of the principal causes of deterioration in reinforced concrete structures is reinforcement corrosion that can lead to reduction of the service life of structure. This problem is so critical in concrete members with exposed reinforcement. In these members, the location of the neutral axis and also the distribution of strain at the cross section will be changed. Cairns and Zhao (1993) showed that in simply supported beams with tension exposed reinforcement, the maximum compressive strain of concrete will increase and neutral axis depth from top surface of concrete member will decrease due to the equilibrium of forces and compatibility of deformations. They concluded that the flexural behavior of concrete beams with exposure of tensile reinforcement changes to the combination of flexural and tied arch action. Minkarah and

Ringo (1981) carried out tests on singly reinforced simply supported beams with nominal top steel and links. They concluded that the ultimate load for beams with exposed tension reinforcement will be significantly large if nominal top steel present in them.
Selecting the suitable repair materials can provide the integrity of repair system and improve its performance. Modulus of elasticity, compressive strength, tensile strength, bond strength between repair concrete and substrate and the location, geometry and dimensions of repaired area and reinforcement are effective parameters in repair of deteriorated concrete structures. Emberson and Mays (1990) performed some tests on simply supported beams which were loaded in two-point bending. They recommended that generally, the compressive and tensile strength of repair concrete should be greater than that of substrate to enhance the flexural performance of repair beams. Atashi et al. (2007) showed that the use of high-strength concrete (HSC) for repairs of concrete slabs in compressive zones is not effective in mechanical performance of repair system. However, using HSCs for thick repairs in tensile zones can increase the debonding load. Also, increasing the tensile strength of concrete by adding fibers to the mix design can significantly increase the debonding load for thick repairs in tensile zones. They also concluded that adding one layers of rebar for thick or thin repairs in tensile zones increases both the ultimate load capacity and the debonding load of repair system. Granju (2001) showed that the reinforcement content of repair material reduces the debonding load due to the reduction of stress amplitude. Also, the risk of debonding will be minimized when the bond strength between repair concrete and substrate is higher than 1.5 MPa (Granju (1996)).
Modulus of elasticity is one of the important parameters that have a direct influence on the behavior of repair system which can control the distribution of stresses. The stress distribution between the repair concrete and substrate with different modulus values won't be uniform and significant difference in deformability will cause problems under specific loading conditions (Czarnecki (2000)). The critical case is when the repair material is stiffer than substrate. In this case, the repair will carry more portion of the load and thus the stress concentration will occur in the contact surface that can cause the sudden failure of the interface and so it will be necessary to using the repairs having high bond strength with substrate. Emberson and Mays (1990) recommended that the difference between the elastic modulus of the repair material and substrate should not exceed 10 MPa. Sajedi et al. (2010) showed that in the repaired reinforced concrete beams with constant dimensions and geometry, the only parameter which can change the stress distribution in the repair system before formation of crack is the difference of elastic modulus between repair concrete and substrate. They concluded that with increasing the elastic modulus of repair concrete with respect to the substrate ($E_{repair} / E_{substrate}$), stress concentration in the repair concrete and at the interface will be increased. So the probability of cracking and debonding will increase and using the repair materials with high values of tensile strength and bond strength will be necessary
In this paper, the influence of reinforcement corrosion and loss of concrete cover on the load of cracking and ultimate load capacity of reinforced concrete beams is investigated by numerical modeling. Then, two different mix designs were chosen to investigate the flexural behavior and debonding risk of repaired beam.

2. NUMERICAL SIMULATION

2.1 Finite element model

Finite element program (ANSYS) was used to simulate the 3-dimensional modeling of deteriorated and repaired reinforced concrete beams. It is assumed that all beams have symmetrical geometry. So, only the quarter part of each beam is modeled. Figure 1 shows the model of quarter part of repaired reinforced concrete beam (a) and deteriorated beam with exposed reinforcement (b) in ANSYS.



(a) (b)

Fig. 1 Modeling of quarter part of (a) repaired and (b) deteriorated reinforced concrete beams

Six groups of beams are investigated in this study. Beam A is the sound beam without any deterioration of concrete or reinforcement. In concrete beam B it is assumed that 25% of tensile reinforcement in 220cm length is corroded without loss of concrete cover. In beam C, it is assumed that there is no corroded reinforcement. However, it is assumed that the tensile reinforcement with 220 cm length is exposed. In other words, it is assumed that in this beam, the concrete region in tensile zone with 220 cm length, 10 cm height and 50 cm width is removed from the mid span of the beam. Concrete D is assumed as a combination of beams B and C. In other words it is assumed that in concrete beam D, tensile reinforcement with 220 cm length and 25% corrosion is exposed. Finally it is assumed that corroded beam D is repaired with two different mix designs. Beam E is repaired with concrete mix design C-C, while the beam F is assumed to repair with C-SFS mixture. The repair thickness was assumed to 10 cm. Figure 2 shows the details of repaired beams.

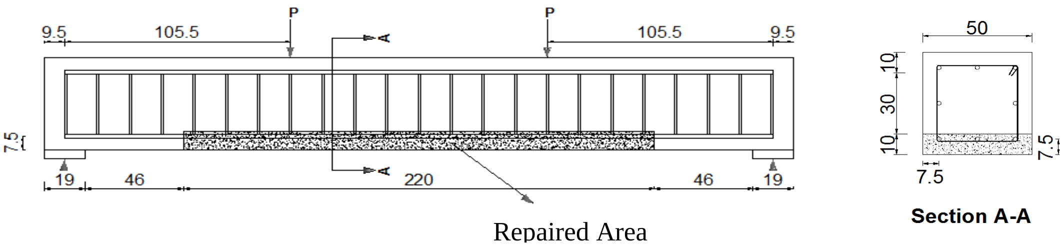


Fig. 2 Details of specimen, reinforcement and the location of repair

As seen in figure 2, the width, height and length of all beams were assumed 50, 50 and 350 cm, respectively. All specimens were loaded under 4-points loading. The distance between two loading points was assumed 120 cm. Three longitudinal reinforcing bars with 20 mm diameter were used as the compressive and tensile reinforcements and the stirrups with 10 mm diameter were used every 15 cm in the beam. The cover thickness is 7.5 cm from the surface of concrete to the centre of longitudinal bars for all beams.

2.2 Properties of repair materials and substrate

Two groups of Repair materials were designed: C-C and C-SFS. Non pozzolanic material was used in mixture C-C and in mixture C-SFS, 32.5% of cement was substituted by GGBS (25% of cement weight) and SF (7.5% of cement weight). Also, non pozzolanic material was used in substrate concrete. The mechanical properties of repair concretes and substrate are listed in table 1.

Table 1. Mechanical properties of materials

	C-C	C-SFS	Substrate
Compressive strength (MPa)	51	70	61
Tensile strength (MPa)	3.8	5.16	4.47
Bond strength (MPa)	0.97	1.5	0.77
Modulus of elasticity (GPa)	37	39	36

It should be noted that the bond strength tests were done on the 15×15 cubic specimens with Bi-surface shear method (Momayez et al. (2004)). In all of the shear specimens, the roughness of the substrate surface is visually kept almost constant in a middle range of 4-5 mm.

2.3 Concrete modeling

An eight-node solid element (solid65) was used to model the concrete materials. The element is capable of plastic deformation, cracking in three orthogonal directions and crushing. In this study, the crushing capability of the concrete element is turned off to avoid the convergence problems. So, the cracking of concrete controls the failure of the models. It is assumed that the concrete is a homogeneous and initially isotropic. The compressive uniaxial stress-strain relationship for the concrete model was obtained using the following equations to compute the multilinear isotropic stress-strain curve (MacGregor (1992)). The multilinear isotropic material uses the von- Mises failure criterion along with the Willam and Warnke (1975) model to define the failure of the concrete. Figure 3 shows the compressive uniaxial stress-strain relationship that was used in this study.

$$f = \frac{E_c \varepsilon}{1+\left(\frac{\varepsilon}{\varepsilon_0}\right)^2} \tag{1}$$

$$\varepsilon_0 = \frac{2 f_c^{'}}{E_c} \tag{2}$$

It should be noted that as shown in figure 3, the first point is assumed as $0.3 f_c^{'}$ for calculating the linear part. Poisson ratio of concrete is assumed to be 0.2 for both repair concretes and substrate. Shear transfer coefficient (β_t) represents the conditions of the crack face. The value of β_t ranges from 0.0 to 1.0, with 0.0 representing a smooth crack (complete loss of shear transfer) and 1.0 representing a rough crack (no loss of shear transfer). But based on the recommendations (Bangash (1989) and Hemmaty (1998)) the values between 0.05 and 0.25

should be considered as a value of β_t. In this study, the coefficient for open cracks (β_t) was set to 0.25 and 0.15 and the coefficient for closed cracks is assumed to be 0.7 and 0.55 for substrate concrete and repair material, respectively. The density for the concrete was not added in the material model.

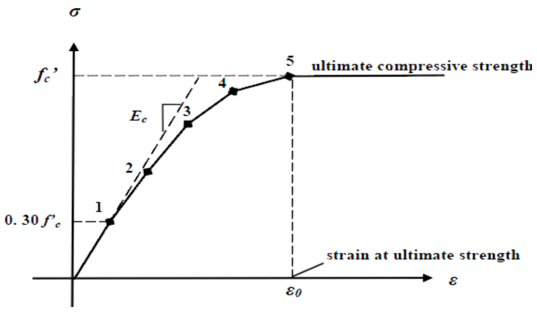


Fig. 3 Simplified compressive uniaxial stress-strain curve for concrete

2.4 *Reinforcing steel model*

Fanning (2001) modeled the response of the reinforcement using the discrete and smeared model for reinforced concrete beams and concluded that the best strategy for reinforcement modeling is the discrete model. In discrete model, the concrete and the reinforcement mesh share the same nodes and concrete occupies the same regions occupied by the reinforcement. A 3D spar element (link8) is used to model the steel reinforcement in discrete model. This element is capable of plastic deformation. Perfect bonding between the concrete and steel reinforcement is considered in this study. It should be noted that only half of the reinforcement was modeled at the center of the beam due to the symmetry. The stress–strain relationship for steel is modeled with a bilinear representation, identical in tension and compression, as shown in Fig. 4. The tangent modulus for the plastic part is taken as $E_{sp} = 0.02E_s$, where E_s is the initial modulus.

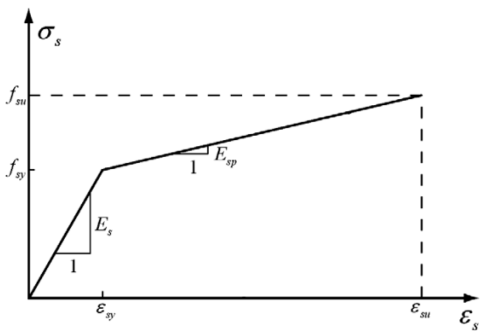


Fig. 4 Idealization of steel stress–strain behavior

Yielding stress of longitudinal and shear reinforcements is taken as 4000kg/cm2 and 3000kg/cm2, respectively. Also the modulus of elasticity and Poisson's ratio is taken as 2.06e6 kg/cm2 and 0.3 for all reinforcing bars.

2.5 *Steel plate model*

An eight-node solid element, Solid45, was used for modeling the steel plates at the support of the beam and loading location in order to avoid the stress concentration problems. This element is modeled as a linear isotropic element. The modulus of elasticity equal to 2.06e6 kg/cm2 and Poisson's ratio of 0.3 were used for the plates.

2.6 Contact behavior

Two types of local failures exist at the interface between the repair concrete and substrate: shear and tensile failures. When the shear stress at the interface exceeds the maximum allowable shear stress, shear failure happens. The result of shear failure is the sliding of two layers relative to each other. Tensile failure occurs when the tensile stress at the interface between the two layers exceeds the maximum allowable tensile strength and leads to debonding. For proper assessment of contact behavior between repair material and concrete substrate, the contact parameters should be determined from bond strength tests such as pull off test, direct tension test, direct shear test and slant shear test. However in this study for modeling of sliding between two layers, Coulomb friction model was used by contact element. Based on the literature, the coefficient of friction can be assumed 0.8 for smooth surfaces (Robins and Austin (1995)) and 1.0 for roughened surfaces (ACI Comm. 303 (1991)). So in this study, the friction coefficient of 0.9 was used for moderate roughness. For modeling of debonding, non-linear spring element (combin39) was employed to simulate crack opening due to tensile stress. It is assumed that the tensile strength between repair concrete and substrate is about 0.56 of bond stress obtained from the results of bi-surface shear test (Momayez et al. (2005)). The bilinear crack opening diagrams can be obtained by CEB-FIP model code 1990 by following equations (CEB (1990)):

$$\sigma_{ct} = f_{ctm}(1 - 0.85\frac{w}{w_1}) \quad \mathit{for}\ 0.15 f_{ctm} \le \sigma_{ct} \le f_{ctm} \tag{3}$$

$$\sigma_{ct} = \frac{0.15 f_{ctm}}{w_c - w_1}(w_c - w) \quad \mathit{for}\ 0 \le \sigma_{ct} \le 0.15 f_{ctm} \tag{4}$$

$$w_1 = 2\frac{G_F}{f_{ctm}} - 0.15 w_c \tag{5}$$

$$w_c = \alpha_F \frac{G_F}{f_{ctm}} \tag{6}$$

$$G_F = G_{FO}(f_{cm} / f_{cmo})^{0.7} \tag{7}$$

$$f_{cm} = f_{cmo}(\frac{f_{ctm}}{f_{ctmo}})^{1.5} \tag{8}$$

$$f_{cmo} = 10\,MPa \tag{9}$$

$$f_{ctmo} = 1.4\,MPa \tag{10}$$

$$f_{ctm} = 0.56\sigma_{bond} \tag{11}$$

where:
σ_{ct} = tensile strength between repair material and substrate (*MPa*)
w = crack opening (*mm*)

w_1 = crack opening (*mm*) for $\sigma_{ct} = 0.15 f_{ctm}$

w_c = crack opening (*mm*) for $\sigma_{ct} = 0$

G_F = fracture energy ($N.mm/mm^2$)

f_{cm} = compressive strength of concrete (*MPa*)

σ_{bond} = bond strength from bi-surface shear test (*MPa*)

In above equations, α_F and G_{FO} are coefficients depend on the maximum aggregate size of repair concrete. The maximum aggregate size of two investigated repair concretes was 16 mm. So, these coefficients will be equal to 7 and 0.03 respectively. Table 2 shows the parameters used to determine the force-displacement behavior of nonlinear spring element (combin 39):

Table 2. Parameters for determination of force-displacement in nonlinear spring elements

Repair concrete	f_{ctm} (*MPa*)	f_{cm} (*MPa*)	G_F ($N.mm/mm^2$)	w_1 (*mm*)	w_c (*mm*)
C-C	0.54	2.34	0.011	0.0194	0.143
C-SFS	0.84	4.65	0.017	0.0199	0.146

Figure 5 shows the tension softening model for two repair concretes used in numerical modeling:

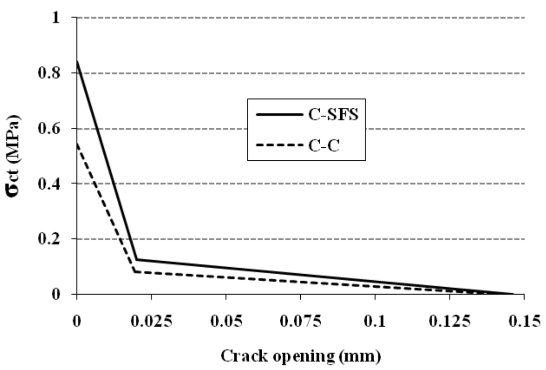


Fig. 5 Tension softening model for simulation of debonding behavior of repair system

2.7 Model verification

The "FCB1R-O" reinforced repair concrete beam from the experimental study of Sahamitmongkol et al. (2008) was simulated in ANSYS and used to calibrate the designed model. Figure 6 shows the comparison between the experimental values and finite element (FE) results in terms of mid-span load versus deflection curve.

As seen in figure 6, there is a good agreement between the results of experimental study and numerical investigation. The maximum load-carrying capacity of the repaired beam obtained from the FE study is about 137.6 kN, while the experimental value was equal to about 143.23 kN.

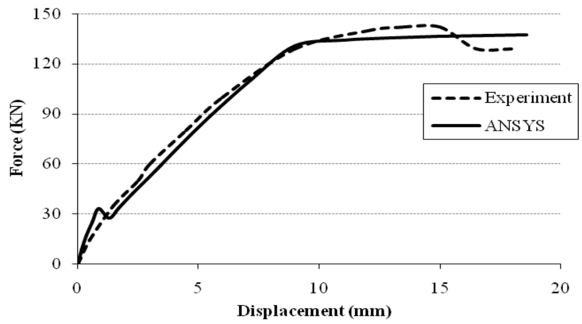


Fig. 6 Comparison between experimental results and numerical simulation for “FCB1R-O”

3. RESULTS AND DISCUSSIONS

Figure 7 shows the load-deflection curves for the mid span of all investigated beams. Also, cracking load and ultimate load capacity for all beams are listed in table 3. From the results it can be concluded that the reinforcement corrosion doesn’t change the load of cracking in deteriorated beams. However in this case, cross section loss of reinforcing bars due to corrosion and its effect on rebar loading capacity reinforcements have caused the reduction of ultimate load capacity. The results of this study show that by 25% reduction of tensile reinforcement area with 220 cm length due to the corrosion, the ultimate load capacity of beam will decrease by approximately 22%. So, corrosion of reinforcement can significantly reduce the ultimate load capacity of beam. On the other hand, loss of concrete cover can reduce the load of cracking in deteriorated beams and it doesn’t change the load-carrying capacity of beams.

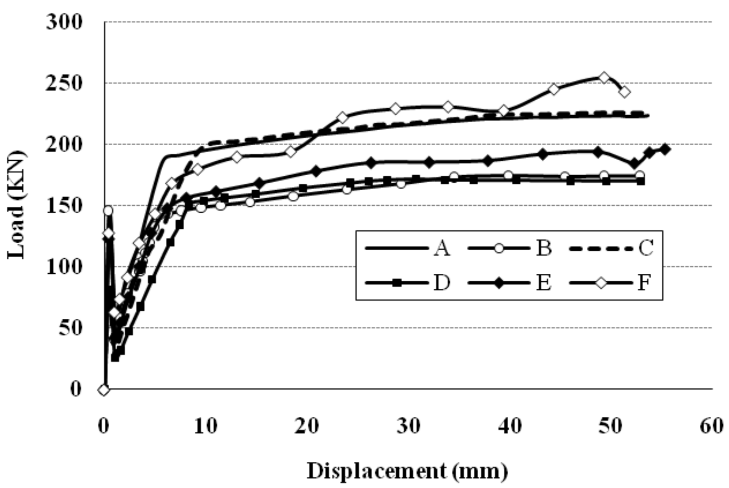


Fig. 7 Load-deflection curves for investigated beams

Table 3. The load of cracking and load-carrying capacity of investigated beams

Beam	A	B	C	D	E	F
Cracking load (KN)	146.6	146	82.1	80.97	123.15	127.52
Ultimate load (KN)	223.4	174.4	225.3	169.55	196.33	253.87

Figure 8 shows the crack distribution and deformation of repaired beams (E and F) under their ultimate load capacity. From the results and considering the form of load-deflection curve for these beams, it is can be concluded that the risk of debonding in beam repaired with C-SFS (beam F) after yielding of reinforcements is high with respect to the beam E which is repaired with C-C mix design. This is probably due to the high difference between the elastic modulus of repair concrete and substrate in this concrete with respect to concrete C-C under high point load that can lead to stress concentration in the interface of repaired area which can lead to debonding of repair concrete (although the tensile and bond strength of this concrete is high). It should be noted that the load of debonding for C-SFS mixture is higher than the load carrying capacity of C-C concrete. So between these two mixtures, it seems that the mechanical performance of C-SFS is better than C-C.
Based on the results of this study, it seems that the behavior of repaired beam depends on the combination of mechanical properties of repair concrete and substrate as well as the bond strength. It is preferred that cracking is initiated before debonding of repair concrete. Because, after delamination of repair layer, the structural role of it will be completely lost; however after cracking of repaired beam, it can be carrying external loading. So, using the repair materials with high tensile strength, similar elastic modulus with respect to substrate and high bond strength can delay the load of cracking and debonding in repaired concretes.

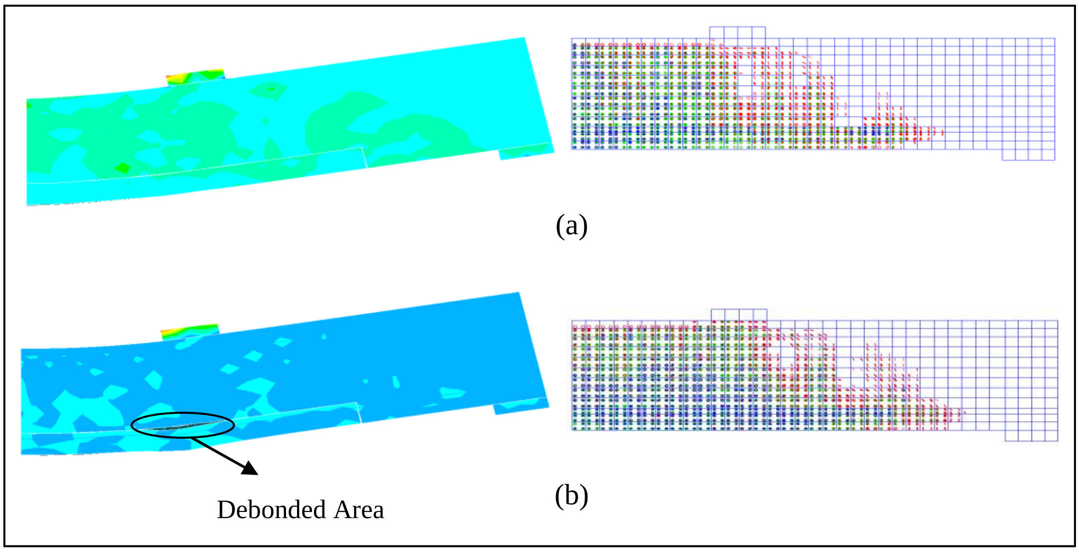


Fig. 8 Principal stress and crack distributions in repaired beams with repair mix desings C-C (a) and C-SFS (b)

It should be noted that for accurate assessment of debonding risk in repaired beams and using more realistic figures, the contact parameters such as friction coefficient as well as the bond strength between corroded reinforcement and concrete should be determined from experiments. However, the results of this study show that the numerical modelling can be used as a suitable way for investigation of deteriorated and repaired concrete beams and determination of the necessity and strategy of repair can to carry out the effective structural repairs and reduce the cost of repair operations in deteriorated concrete structures.

4. CONCLUSIONS

Finite element investigation on the flexural behavior of sound, deteriorated and repaired reinforced concrete beams was investigated in this study. Following results obtained:

- In the corroded reinforced concrete beams, the corrosion of reinforcement doesn't change the load of cracking, considerably. However, it reduces the load carrying capacity of beam, significantly.
- In deteriorated RC beams, the loss of concrete cover due to the corrosion of rebars can reduce the load of cracking. However, no changes in ultimate load values were observed.
- Cracking tendency of repaired concrete depends on the tensile strength of materials and elastic modulus of repair concrete with respect to the substrate ($E_{repair} / E_{substrate}$). Repair materials with high tensile strength and similar stiffness (with respect to substrate) will have a suitable flexural behavior.
- Debonding of repaired concrete depends on the mechanical properties of repair material and substrate together with the bond strength and repair materials with high bond strength has not the best performance, necessarily. Although the debonding of them will be delayed. deteiorated structures
- Numerical modeling of is the suitable tool for assessment of the existing deteriorated structures and decirding on the strategy of repair and selection of compatible repair materials for reducing the cost of repair operations and carry out the effective structural repairs.

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