Sustainable Solutions in Structural Engineering and Construction
Edited by Chantawarangul, K., Suanpaga, W., Yazdani, S., Vimonsatit, V., and Singh, A.

ISBN: 978-0-9960437-0-0

.

RETROFIT STEEL CORRODING RC BEAMS USING CFRP COMPOSITES: NLFE ANALYSIS

RAMI H. HADDAD

Civil Engineering Dept, Jordan University of Science and Technology, Irbid, Jordan

Corrosion of steel reinforcement is one of the main durability problems facing reinforced concrete infrastructures worldwide. Steel degradation and/or cracking of concrete both cause severe reduction in bearing capacity, leading ultimately to failure. The potential of repairing corroded concrete beams with two configurations of carbon-fiber reinforced polymeric (CFRP) composites was investigated with a nonlinear finite Element (NLFE) model. CFRP composites were assumed to be either bonded directly to existing concrete or to a new concrete cover; replacing the old one. The data generated showed that the load-bearing capacity and stiffness for concrete beams, as long as corrosion levels were below 10%, can be fully restored. However, the ductility in terms of deflection at failure would be reduced, especially for repair techniques that involved anchoring with CFRP sheets. For corrosion levels greater than 10%, attaching CFRP composites to a new concrete cover contributed to additional improvements in load capacity and stiffness ranging from 10 to 15% of that achieved from similar repairing on existing concrete. The failure modes indicated that debonding failure prevailed, and that the extent of debonding prior to failure depended upon the corrosion level and on whether the concrete cover was replaced or not.

Keywords: Corrosion, Repair, Nonlinear finite element, Debonding, Cover removal.

1 INTRODUCTION AND REVIEW

For structural elements characterized by corrosion, the external application of Fiber Reinforced Polymers (FRP) represents an effective and economic upgrade technique. It provides an alternative solution to traditional methodologies (e.g., externally-bonded steel plates), allowing for upgrades of the structure without significant interruption of its use, and providing high performance in terms of durability. This results in important reductions in application and maintenance costs. The strengthening of existing RC flexural members by FRP composites is one of the most widely-adopted solutions for supplying additional external tensile reinforcement to joists, slabs, or bridge girders where the steel reinforcement area had been reduced by corrosion. Many researchers have attempted to characterize the performance of corrosion-damaged RC structures, but little information is available in the literature on the structural behavior of such beams strengthened or repaired with FRP composites (Sherwood and Soudki et al. 2000, Soudki and Sherwood 2000). It can be hypothesized that an FRP-wrapped member undergoing active corrosion may exhibit improved structural performance by a combination of three mechanisms: 1) confinement of the concrete section, thereby lessening corrosion cracking and bond splitting cracks; 2) prevention of further chloride ingress into concrete, thereby the reducing rate of corrosion; and 3) increased flexural and shear resistance to overcome the loss in the steel cross-section. However, studies and code provisions regarding this new technique are still in the preliminary stages; hence further analyses due to steel reinforcement corrosion appear to be strongly needed for investigating the main aspects of the flexural behavior.

This paper develops and validates a nonlinear finite element (NLFE) model to predict flexural performance of corroding steel-concrete beams being repaired with effective configurations of CFRP plate reinforcement and CFRP sheets anchorage. The first, designated as S-CFRP, consisted of attaching CFRP plates to concrete beams at their tension side. The second, designated as S-CFRP-A, consisted of attaching CFRP plates to the concrete beams at their tension side, anchored with one laminate of CFRP sheets. Those were bonded laterally to concrete that was soft at a depth of 50 mm, and extended along the full bond length of the underlying plates at 800 mm using a special epoxy. Factors studied were steel corrosion level and concrete cover removal prior to repair.

2 NLFE MODELING

2.1 Introduction

The proposed NLFE model considered the nonlinear constitutive material properties of concrete, cracking of concrete due to steel corrosion, and yielding of steel reinforcement. The model predictions shall include mechanical behavior and failure mode of modeled beams. Typical beams of the structures in Figure 1 were considered in the modeling for unstrengthened and repaired reinforced-concrete beams. The dimensions and steel reinforcement were computed such that flexural failure occurs prior to shear failure. For modeling the effect of reinforcing steel corrosion, four major factors were considered: (1) the reduction in steel bars' cross-sectional area; (2) the reduction in strain at failure for corroding bars, accounted for by using a formula from Bertoa et al. (2008); (3) radial stresses generated due to the formation of rust, which was estimated using the equation by El Maaddawy and Soudki (2007); (4) a perfect bond between concrete and steel reinforcement, assumed because of the relatively large embedment length of the steel bars in concrete.

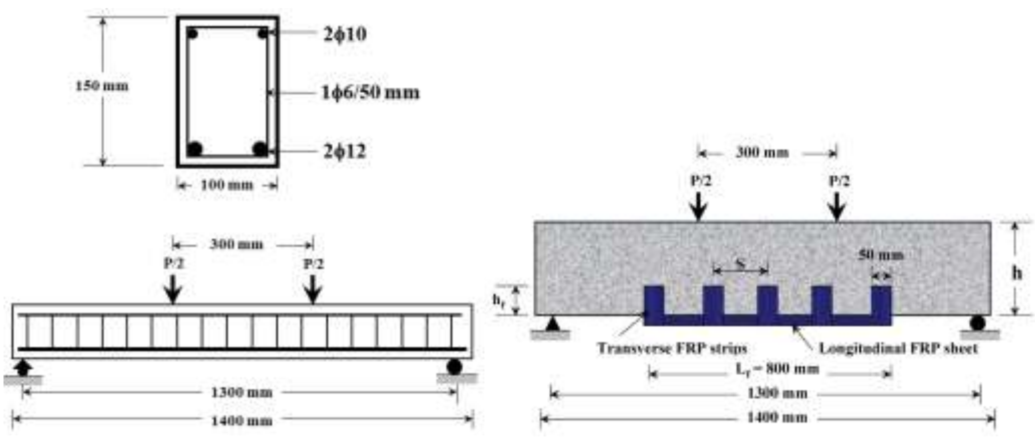

Figure 1. Reinforcement details of the test specimens and adopted repair techniques.

2.2 Materials Modeling and Characteristics

Various types of elements were used in the modeling of reinforced-concrete repaired beams. For concrete, SOLID 65 element was used with compressive strength of 30 MPa, elastic modulus of 40.5 GPa, tensile strength of 3.61 MPa, and Poisson's ratio of 0.2 adopted for analyzing all beams. LINK8 was used for steel reinforcement, whereas SOLID 45 was used for steel support with a yield stress of 416 MPa, Poisson's ratio of 0.3, and an elastic modulus of 200 GPa adopted as mechanical properties. Shell 99 element was used for CFRP composites, with the input data including number of layers, thickness of layer, and orientation of fiber in each layer. The tensile strengths were 2505 and 3900 MPa, whereas the longitudinal and transverse elastic modulus were 150 and 230 GPa for CFRP plates 12 mm thick, and 6 and 9.2 GPa for sheets 0.17 mm thick. Convergence of results by the NLFE model was performed for control concrete beams (Bathe 1996). The meshing with element sizes of 12.5 mm and 25 mm gave almost similar results; hence a mesh size of 12.5 mm was adopted in the analysis. The radial pressure due to steel corrosion was simulated using polar forces acting on the nodal points surrounding the two steel bars.

The total load applied was divided into a series of load increments, or load steps. Newton-Raphson equilibrium iterations provided convergence at the end of each load increment within tolerance limits equal to 0.001 with load increment of 0.35 kN. Load-step sizes were automated by ANSYS program for the maximum and minimum load-step sizes. Failure for each model was identified when the solution for 0.0035 kN load increment was not converging. The implementation of an element with a mesh size of 12 mm yielded good convergence hence used in the present NLFE analysis.

3 RESULTS AND DISCUSSION

3.1 Repair without concrete cover removal

Figure 2 shows the load vs. deflection diagrams for control (unrepaired); those underwent four levels of steel corrosion (0, 5, 10, and 15%). The characteristics of these curves, stiffness, ultimate strength, and deflection at failure, were also determined and their residuals computed with respect to those of beams without steel corrosion for the purpose of comparison. The curves demonstrated clearly that steel corrosion has caused significant reduction in load capacity, stiffness, and ultimate deflection. All curves showed negligible deflection up to about 10% of their ultimate load capacity, before experiencing linear followed by nonlinear trend behavior up to failure. The percentage reduction in ultimate strength, stiffness, and deflection at failure were the highest at 15% at 45, 11, and 40%, respectively.

Repair concrete beams with the two proposed techniques, S-CFRP and S-CFRP-A, resulted in significant recovery of mechanical characteristics lost due to steel corrosion, as can be deduced from the graphs in Figure 2. As can be expected, the repair scheme involved using anchoring by CFRP sheets has imparted significant improvement over that employed CFRP plates only. The residual percentages for load capacity using S-CFRP attained 134%, 104%, 81%, and 60%, while S-CFRP-A attained 160%, 121%, 106%, and 79% for beams that underwent corrosion levels at 0%, 5%, 10%, and 15% respectively.

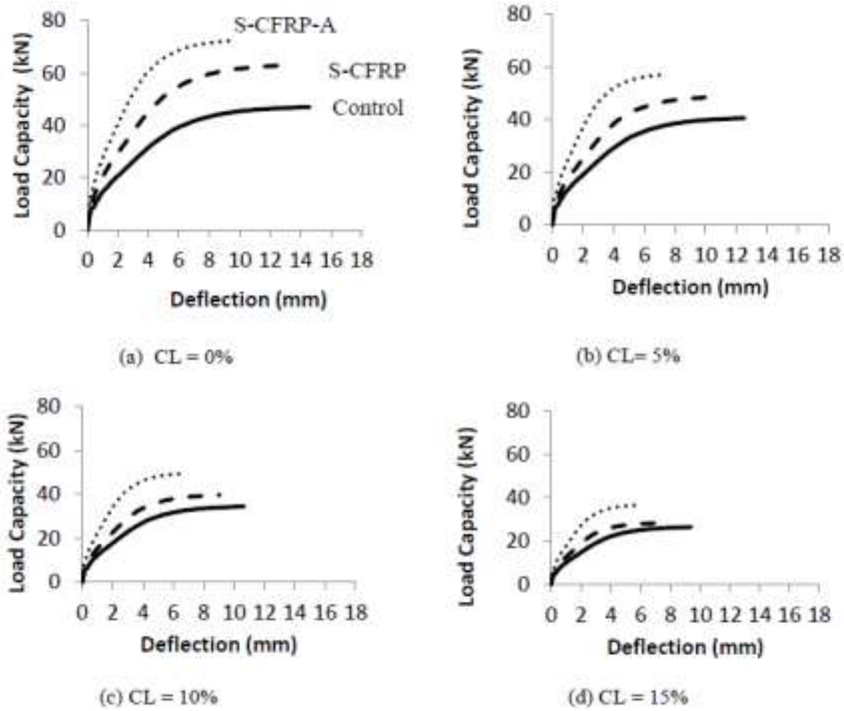

Figure 2. Load capacity versus deflection for repaired reinforced concrete beams without concrete cover removal.

These results suggested that beams with corroding steel at 10% regained its original load capacity when the proposed anchorage was incorporated in the repair process, and that the full recovery of original load capacity would not be possible upon a corrosion level of 15%. The stiffness, on the other hand, was improved by repairing, especially when an anchorage system was applied to CFRP plates, even for beams that underwent steel corrosion at 15%. The present results indicated that the ductility of the system, as indicated by the deflection at failure, was reduced upon repair and with more steel corrosion: the residual values at a steel corrosion of 15% were 47% and 40%. The results showed that using the proposed anchorage system, although contributing to a higher recovery of load capacity, reduced the overall ductility of the system.

3.2 Repair with Concrete Cover Removal

In practice, concrete cover under corroding reinforcement is usually removed, and rusting steel cleaned before a new concrete cover is cast and cured. To simulate this, the radial stresses resulting from steel corrosion were modeled such that the concrete cover was stress free; hence concrete was treated as intact with no cracking. Accordingly, the mechanical properties for the concrete cover were computed based upon those of the properties of present concrete. The results showed similar trend behavior as those observed for concrete without cover replacement, but with a higher load capacity for beams repaired with the CFRP composites. Of course, such behavior is expected because concrete-cover replacement had resulted in augmented bonds between the concrete and the attached CFRP composites. The results indicated that the concrete cover replacement had no effect on the mechanical properties of repaired beams when corrosion levels were at 0% and 5%. However, it was effective on promoting the recovery of mechanical properties for corrosion levels of 10% and 15%. The additional increase in load capacity ranged from 10% to 15% over beams repaired without concrete-cover replacement. The residual stiffness and deflection at failure for repaired beams with concrete cover replacement followed an almost similar trend as that of load capacity.

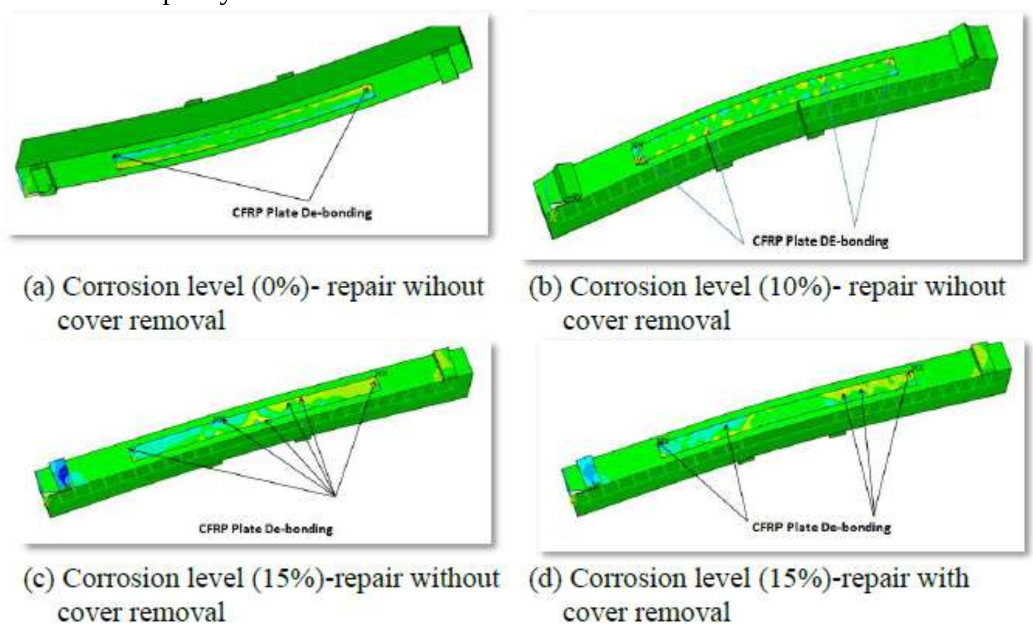

Figure 3. Modes of failure for repaired concrete beams.

3.3 Mode of Failure

The mode of failure of reinforced concrete beams with certain levels of steel corrosion that were repaired with CFRP composites using two typical techniques can be understood by referring to the schematics in Figure 3. As can be noticed, the extent of CFRP plates debonding was dependent upon the corrosion level, and on whether or not the concrete cover was replaced prior to repair. It is evident that with more corrosion, a higher contact area between CFRP and concrete were detaching prior to failure, as seen in Figure 3 (a through c). As may be expected, concrete-cover replacement had a positive impact on reducing the delaminated zones due to absence of corrosion-induced cracks, as can be concluded from Figure 3 (d).

4 CONCLUSIONS

Steel corrosion of under-reinforced concrete beams caused significant reduction in their load-bearing capacity, stiffness, and deflection at failure, reaching as high as 45%, 11%, and 40%, respectively. Repairing with CFRP composites without concrete replacement enabled full load capacity recovery for beams with steel corrosion at less than 10%.

Repairing using CFRP composites with concrete replacement helped promoting percentage recovery in load capacity and stiffness for beams with corroding steel levels of 10 and 15%. The repair technique caused reductions in flexural ductility as deduced from deflection at failure. CFRP composites debonded prior to repaired beams' failure, with a lower debonding extent for beams that a) had lower corrosion levels and b) were repaired with concrete-cover replacements.

References

Bathe, K. J., *Finite Element Procedures*, Prentice-Hall, Inc., New Jersey, 1996.

Bertoa, L., Simionib, P., and Saettab, A., Numerical modeling of bond behavior in RC structures affected by reinforcement corrosion, *Eng Struct*, 30, 1375–85, 2008.

El Maaddawy, T., and Soudki, K., A model for prediction of time from corrosion initiation to corrosion cracking, *Chem. Concr. Comp.* 29, 168–75, 2007.

Sherwood, E. G., and Soudki, K. A., Rehabilitation of corrosion damaged concrete beams with CFRP laminates-pilot study, Comp B: *Eng*, 31(1), 453- 459, 2000.

Soudki, K. A., and Sherwood, T., Behavior of reinforced concrete beams strengthened with CFRP laminates subjected to corrosion damage, *Can J Civil Eng*, 27(5), 1005-10, 2000.