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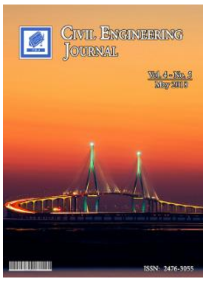
Civil Engineering Journal

Vol. 4, No. 5, May, 2018



Simulation of the Behavior of Corrosion Damaged Reinforced Concrete Beams with/without CFRP Retrofit

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Received 23 January 2018; Accepted 27 April 2018

Abstract

Harsh environmental conditions along with aggressive chemical agents are known as one of the main reasons behind damages observed in reinforced concrete members. Corrosion of reinforcement worldwide is one of the leading causes of damages occurred in reinforced concrete over the lifespan. There are many critical energy and transportation infrastructures located on coastal regions exposed to high humidity and chloride content where they are highly prone to reinforcement corrosion. This calls for retrofit methods, which safeguard not only the strength but also the durability of corrosion deteriorated reinforced concrete structures. Carbon fiber polymers considering their mechanical and chemical properties are recognized as one of the main retrofit techniques. In this study, the influence of different levels of corrosion on the structural behavior of reinforced concrete beams is studied. ABAQUS software package is employed to simulate the nonlinear behavior of reinforced concrete beams with tensile reinforcements and stir-ups corrosion degrees of 20% and 40%. The structural behavior of original damaged specimen as well as the same specimen strengthen with carbon fiber reinforced polymer (CFRP) is studied. The purpose of the retrofit is compensate for the loss of shear and flexural capacity of the member due to corrosion. Different variants for the arrangement of CFRP strips are studied and compared. The result of the current research further uncaps the efficiency of fiber polymers to secure strength and durability of corrosion damaged reinforced concrete members.

Keywords: Retrofit; Reinforcement Corrosion; Reinforced Concrete Beam; Carbon Fiber Reinforced; Polymer (CFRP).

1. Introduction

Strain compatibility' between reinforcement and concrete is one of the most fundamental assumptions widely used to simulate the structural behavior of reinforced concrete members. This means that no slip can take place between steel and concrete [1]. Hence, the stress-strain relationship of concrete and steel can be defined separately, and assigned accordingly to steel and concrete provided within the cross-section [2]. However, this assumption mostly does not lead to the reliable prediction of the structural behavior of reinforced concrete members with corroded reinforcements primarily due to the critical decline of the bond between steel and concrete in such specimens [3]. Previous studies suggests that the below corrosion impairments play a prominent role in the process of strength deterioration in reinforced concrete members [4]:

1. Steel reinforcement concrete cover cracking due to the expansion of corroded rebars.
2. Steel cross section reduction.

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http://dx.doi.org/10.28991/cej-0309148

3. Deterioration of the bond between steel and concrete.

In this research, the first corrosion impairment has been put away from consideration. Ordinary concrete exhibits relatively negligible tensile strength. Therefore, cracking of tensile steel reinforcements induced by corrosion of tensile reinforcements does not considerably influence the structural behavior of reinforced concrete beams [5]. Thus, the second and third items of above list are taken into account in steel reinforcement corrosion simulations conducted in the current study. Several experimental investigations have been demonstrated the effectiveness of CFRP retrofit to restore the loss of flexural strength of corrosion damaged reinforced concrete beams [6]. CFRP retrofit is widely used and the effectiveness increased by several modifications [7]. Moreover, an extensive research has been carried out at University of Waterloo under different loading regimes and conditions [8], which further strengthened the significance of CFRP retrofit for corroded reinforced concrete members. More recently, Lingga conducted an experimental study on six reinforced concrete beams, which shows the efficiency of CFRP to retrofit corrosion damaged reinforced concrete beams [9]. Moghadam et al. reported one of the first numerical studies on the behavior of corrosion damaged reinforced concrete beams retrofitted with CFRP sheet. However, the influence of corrosion on the bond between the steel and concrete is not taken into account in their study [10]. Recent studies concentrated on improving the accuracy of this approach to increasing the efficiency of CFRP to retrofit corrosion damaged reinforced concrete in tidal zone [11]. The effect of materials on the reliability of reinforced concrete beams in normal and intense corrosion, is widely investigated on supplementary research [12]. The efficiency of Finite element ABAQUS software in numerical modeling was illustrated in several studies [13]. Development of a numerical model for the behavior of concrete is a challenging task. Concrete is a brittle material in tension and has different behavior in compression and tension. ABAQUS demonstrated the reliable numerical results and the result of the concrete modeling has been verified [14].

The present research investigated the structural behavior of original damaged specimen as well as the same specimen strengthen with carbon fiber reinforced polymer (CFRP). The purpose of the retrofit is compensate for the loss of shear and flexural capacity of the member due to corrosion. Different variants for the arrangement of CFRP strips are studied and compared. This research takes a step further by seeking for a numerical technique to simulate the structural behavior of corroded reinforced concrete beams retrofitted by CFRP.

2. Modelling Assumptions and Limitations

In this study, ABAQUS is employed to handle nonlinear simulations in the current study [15]. Concrete is modeled using 'solid element,' while steel reinforcements are modeled using 'wire element.' Nonlinear behavior of concrete is captured by using 'damaged plasticity concrete,' and nonlinear behavior of steel reinforcement is modeled in trilinear fashion using 'plastic/isotropic' model. Corrosion deteriorative effects are concentrated into a reduction of reinforcement cross-sectional area and decline in the bond between steel reinforcement and concrete. It is practical to use a specific constraint available by ABAQUS known as 'embedded region to simulate slips between steel reinforcement and concrete. It is assumed that there is strain compatibility or perfect bond between steel and concrete. Although this approach works well for intact steel reinforcements, however, it is unable to mimic critical decline of the bond between steel reinforcement and concrete. Therefore, a different method should be devised to simulate the interface between steel reinforcement and concrete. The bond between corroded steel reinforcement and concrete is modeled using springs shown on Figure 1. Bond spring is a spring in which all degrees of freedom are constrained except the axial translational degree of freedom. One end of bond spring is connected to reinforcement node, and the other end is connected to the concrete node. Slip between concrete and steel can be simulated by defining slip-bond behavior in the axial direction of the spring [16]. It is required to maintain the necessary accuracy of the simulation by choosing small enough intervals between bond springs. The accuracy of this approach has been extensively verified by several researchers [17]. The behavior of bond spring resembling bond to intact steel reinforcement can be defined as shown in Figure 2 [18]. Parameters of bonding strengths condition between concrete and steel of the beam is expressed in the Table1.

Table 1. Parameters of bonding strengths condition between concrete and steel for the study of the beam

Parameter	Good bond condition	Other bond condition
τ max	2.5 fck	1.25 fck
s1	1.0 mm	1.8 mm
s2	2.0 mm	3.6 mm
s3	cclear	cclear
α	0.4	0.4
τ f	0.4 τ max	0.4 τ max

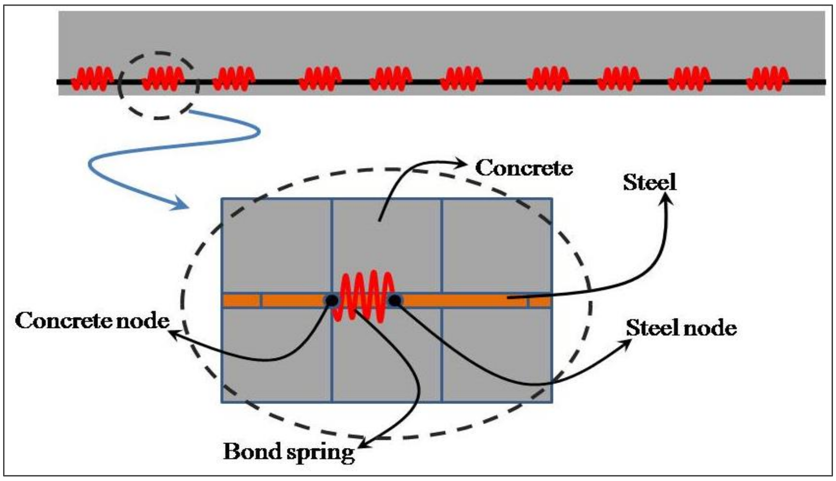

Figure 1. Simulation of the bond of steel reinforcement to concrete using bond spring

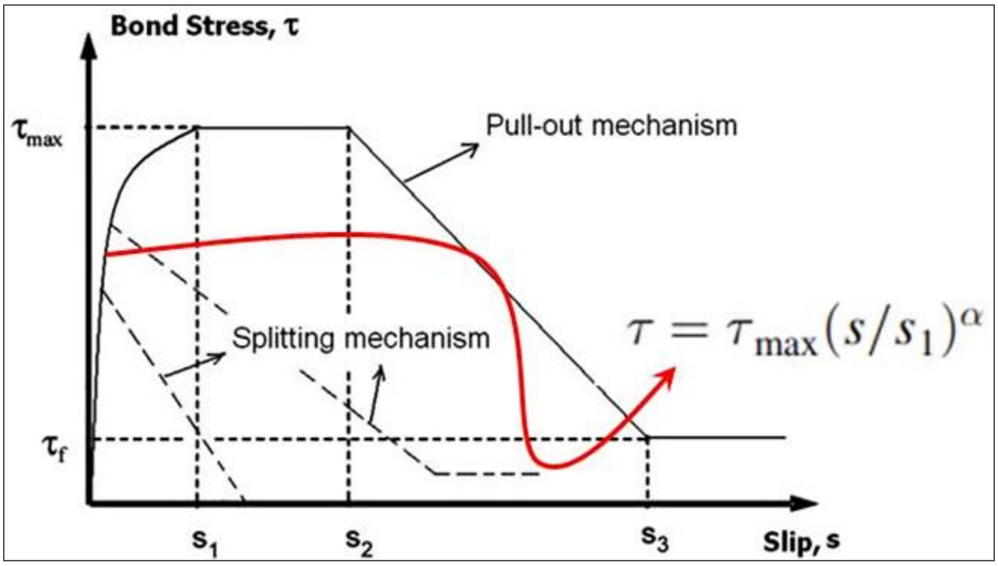

Figure 2. Spring behaviour of intact steel reinforcement and concrete

Corrosion influence can be captured as follows:

- The ultimate bond strength (T^c_{max}) needs to be modified by the corrosion level as illustrated on Figure 3.
- The residual bond is set to be 15% of the ultimate bond strength.
- The failure mode is assumed to be splitting mechanism.

The ratio of the ultimate bond strength (T^c_{max}) to developed bond strength (T^c) can be obtained for each corrosion degree based on the above assumptions and using Figure 3 [19]. In this study, the bond behavior is idealized as plotted in Figure 4.

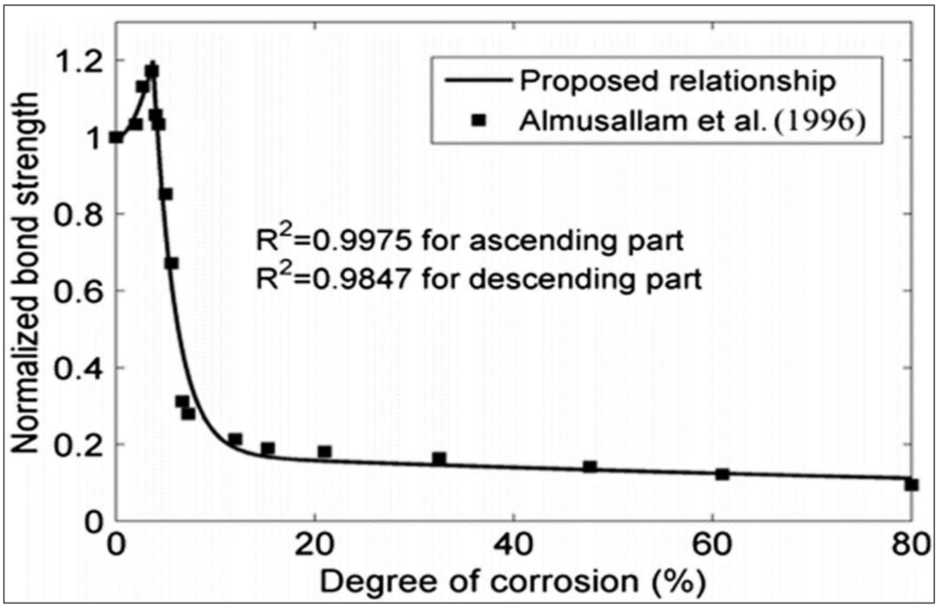

Figure 3. Corrosion influence on the ultimate bond between steel reinforcement and concrete

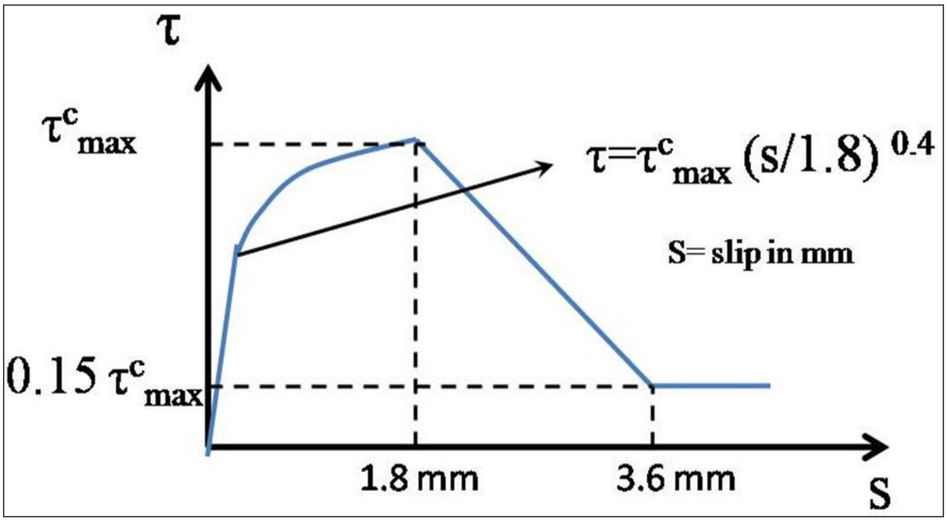

Figure 4. Bond spring behaviour used for corroded specimens

3. Validation of Numerical Modelling

3.1. Example 1

A beam is investigated to verify the adopted approach in handling simulation of the bond between steel and concrete (Figure 5). The capacity of the beam is predicted for different cases as explained in Table 2.

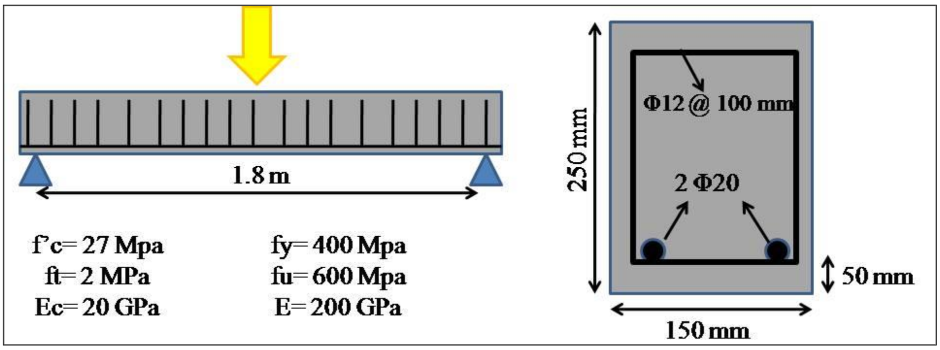

Figure 5. Beam specimen undertaken to study bond effects on the beam structural behaviour

Table 2. Different bond strengths between concrete and steel adopted for the study of the beam

	Ultimate bend strength (MPa)	Modelling technique
Case 1	33	Embedded region
Case 2	6	Bond spring
Case 3	1	Bond spring

Reinforcement cross-sectional area is assumed to be constant for all cases. Moreover, the perfect bond is assumed between steel stirrups and concrete. Force mid-span curve of the specimen is predicted using the embedded region as well as bond spring modelling approach as presented on Figure 6. This reveals the significant influence of bond between steel and concrete on the strength and stiffness of the specimen.

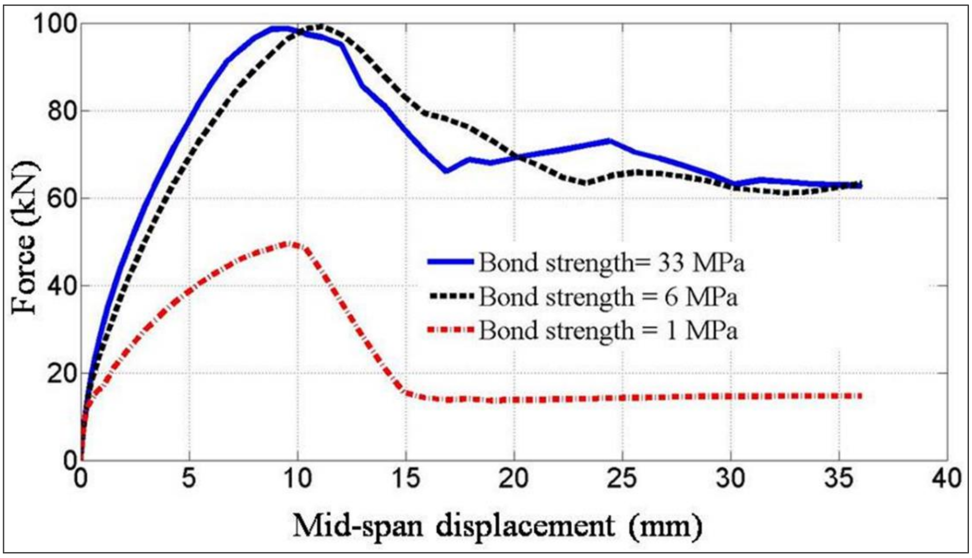

Figure 6. Beam specimen undertaken to study bond effects on the beam structural behaviour

3.2. Example 2

The experimental result [4] and numerical findings on structural behavior of the beam are compared in Figure 7 to validate the results of numerical simulations. Beam cross section includes a depth of 250 mm and width of 200 mm with six rebars with an intact diameter of 13 mm were used. Stirrups with the size of 6 mm spaced at 50 mm in two ends of the beam and 100 mm in the middle of the beam were utilized. Tensile reinforcement was exposed to accelerated corrosion using electrical currency to achieve different levels of corrosion. After that, specimens are tested under two concentrated loads. Test specimens consist of an intact specimen and two corroded specimens with 7.9% and 25.3% degrees of corrosion. The degree of corrosion is obtained based on weight loss of rebars in the corrosion process. Mechanical properties of the specimens are demonstrated in Table 3.

Table 3. Mechanical properties of beams tested [4]

Concrete		Flexural bar		Transverse bar	
Compressive strength	39.2 MPa	Yielding strength	343 MPa	Yielding strength	226 MPa
Moduli of Elasticity	29.419 GPa	Ultimate strength	477 MPa	Ultimate strength	415 MPa
Poisson ratio	0.19	Moduli of Elasticity	182 GPa	Moduli of Elasticity	194 GPa

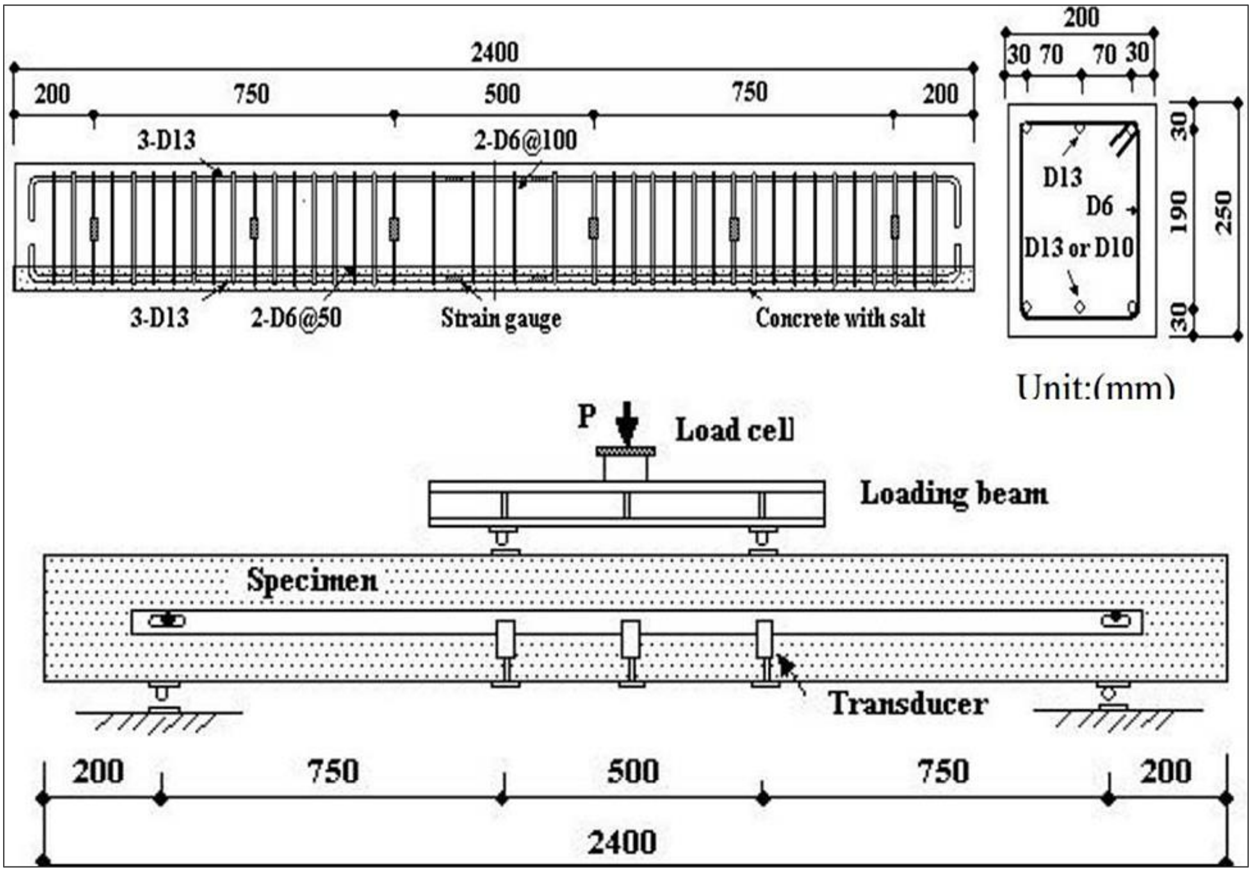

Figure 7. Beam specimen adopted for validation of the numerical procedure [4]

Figure 8 presents details of the modelling the specimens by using ABAQUS software package. Concrete is modelled by ‘solid element’ using ‘damage plasticity model.’ Concrete tensile strength is assumed to be 3 MPa. Rebars are modelled by ‘wire element’ using ‘trilinear model.’ Therefore, a rebar continues to gain strength beyond yielding point up to the strain of 5% at which it reaches the ultimate strength of reinforcement, and thereafter the plateau starts. This finding matches well with experimental result of rebar uniaxial tension test. Bond spring is modelled by ‘connector element’ distributed at a spacing of 100 mm over the length of the reinforcement. This element connects the reinforcement to the surrounding concrete at a constant distance specified as 100 mm. However, the bond of intact tensile reinforcements, compressive reinforcements and stirrups to the concrete was modelled using ‘embedded region’ technique. Figsure 9 to 11 compare the experimental and analytical results separately for each specimen. Results demonstrate a good agreement between results for different degrees of corrosion. It can be concluded that corrosion not only reduces the moment capacity of specimens but also it leads to relatively more concentrated crack patterns.

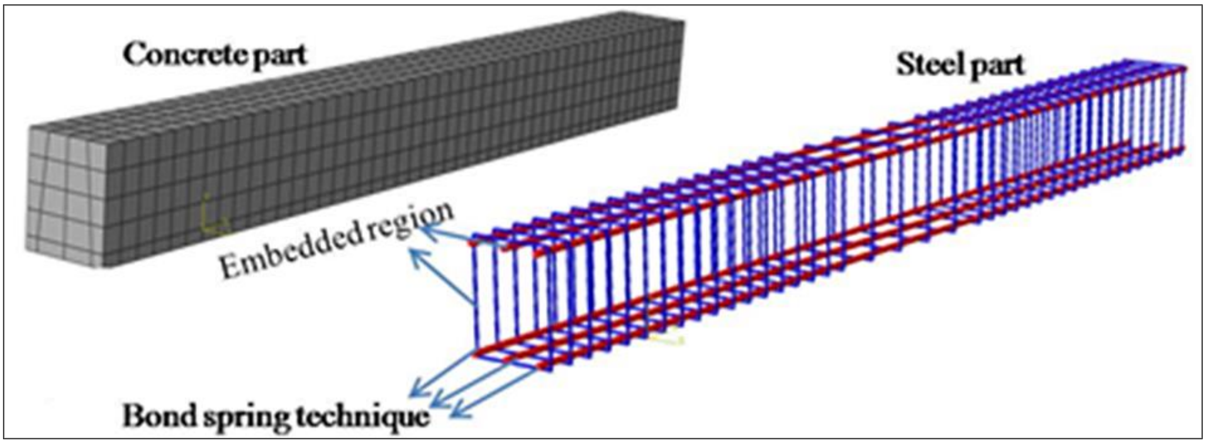

Figure 8. Bond-slip behaviour of bond springs in different levels of corrosion

In other words, corrosion to some extent precludes propagation of flexural cracks along the length of the beam, which can be attributed to the decline of the bond between steel reinforcement and concrete as well as reinforcement cross-sectional area.

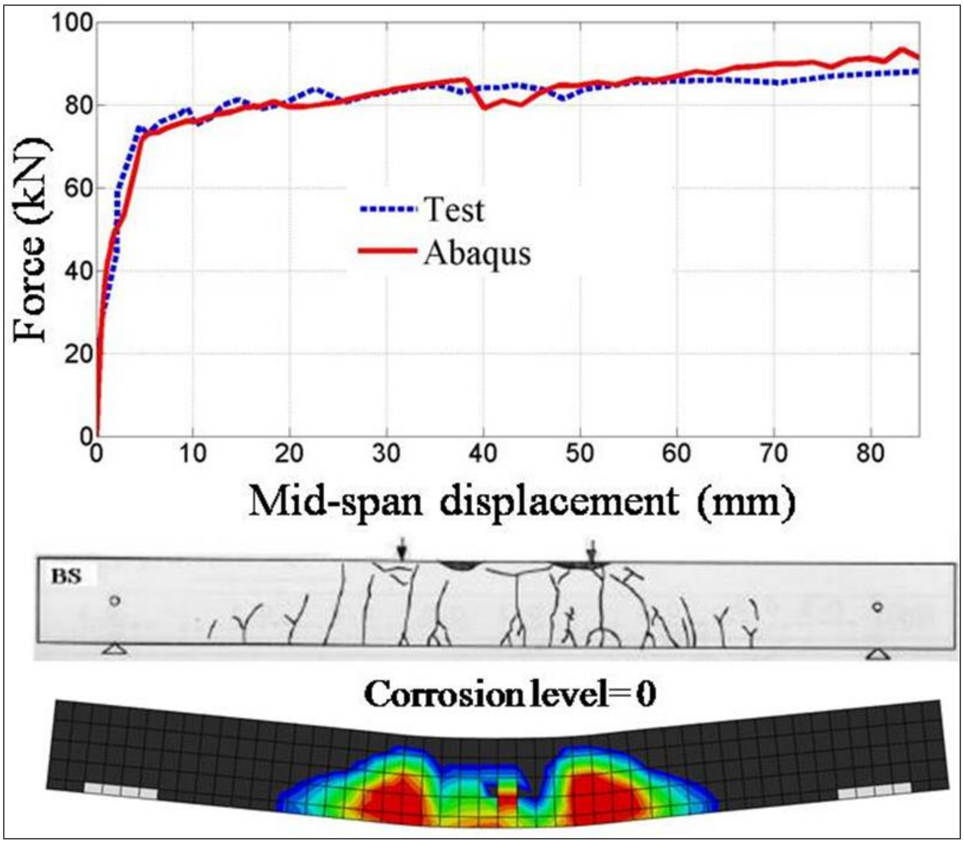

Figure 9. Comparison between analytical and experimental load-deformation response and crack pattern of intact beam specimen

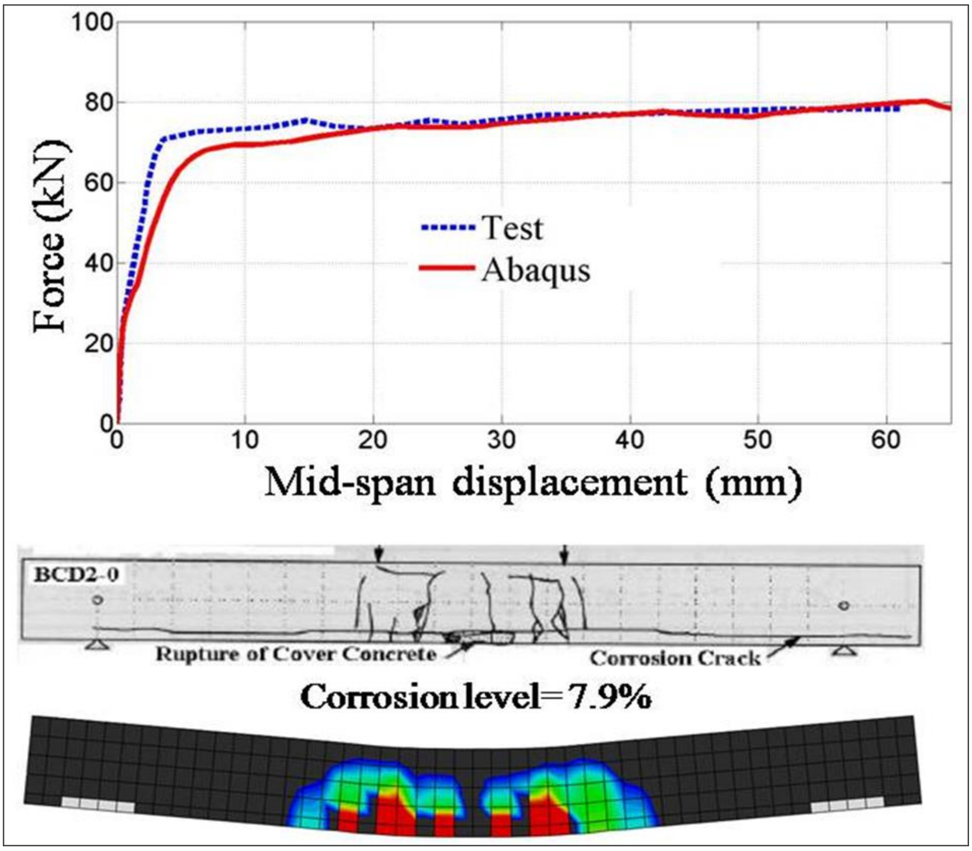

Figure 10. Comparison between analytical and experimental load-deformation response and crack pattern of beam specimen with 7.9% degree of corrosion

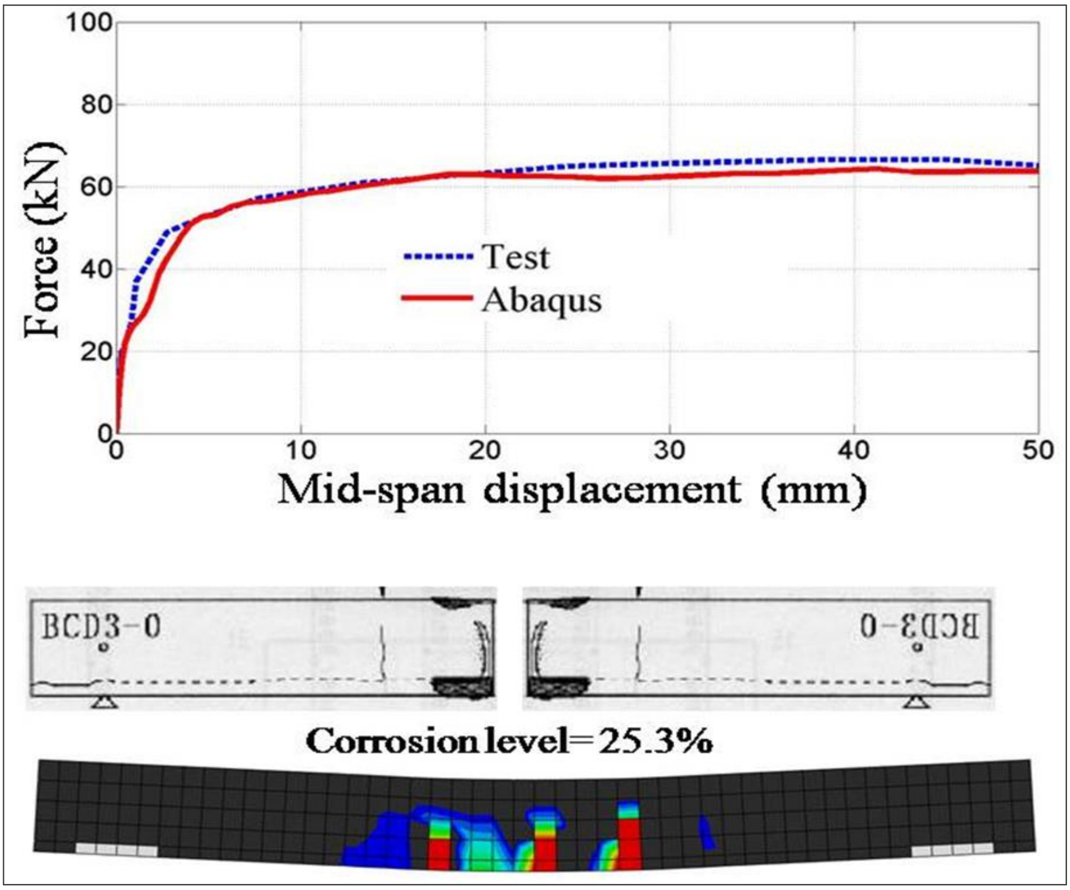

Figure 11. Comparison between analytical and experimental load-deformation response and crack pattern of beam specimen with 25.3% degree of corrosion

4. Behaviour of the Corrosion Damaged Specimens

Two corrosion levels of 20% and 40% are chosen for beam specimens. The modeling approach is similar to validation part of the study. However, the cross-sectional area reduction is taken into account for stirrups and bond decline is neglected. One reason is the observation made in the validation phase of the study, which proves for stirrups cross-sectional area reduction is greatly more influential than bond decline. Secondly, stirrups transfer loads to the surrounding concrete by 'bearing mechanism' rather than 'bond mechanism' due to their short lengths and 90º bending. Figure 12 compares the structural behavior of intact specimen and corroded specimens with different degrees of corrosion. Results demonstrated that specimens with 7.9% and 25.3% degrees of corrosion are the same of the beam as discussed in the previous section except stirrups are intact. Specimens with 20% and 40% degrees of corrosion adopted for the current section including both tensile longitudinal and transverse reinforcements corrosion. Compressive reinforcements are intact in all cases. This assumption matches well with the real construction practice: typically there is a slab resting on top of the beam protecting it against corrosion. Figure 12 reveals an increase of the degree of corrosion leads to beam moment capacity reduction. This reduction can be more attributed to the reduction of the area of reinforcements than the decline in the bond due to the fine sizing of bars. Moment-curvature of the beams are constructed using SAP 2000 to prove this argument further, [20]. Strain compatibility is assumed which neglects any slip between steel and concrete. Table 4 summarizes beam moment capacity for different degrees of corrosion.

Table 4. Comparison between SAP2000 and ABAQUS results

		ABAQUS predictions			SAP2000 moment-curvature analysis
Corrosion degree (%)	**Effective tensile rebar size (mm)**	**Ultimate load (kN)**	**Moment capacity (kN.m)**	**Percentage of moment capacity reduction (%)**	**Moment capacity (kN.m)**
0	13.00	90	33.8	0	32.9
7.9	12.47	80	30	11	29.7
20	11.6	70	26.3	22	26.2
25.3	11.23	67	25.1	26	25.0
40	10.0	58	21.8	36	20.8

One can observe that results obtained from SAP 2000 (exclusive of reinforcement slip) and ABAQUS (inclusive of reinforcement slip) are very close. The main reason behind slight differences is ignorance of tensile strength in SAP 2000 modeling. Moreover, reinforcement stiffness beyond yielding is slightly different in these two models. The sectional moment-curvature curve is obtained using a feature of SAP 2000 software package called 'section designer.'

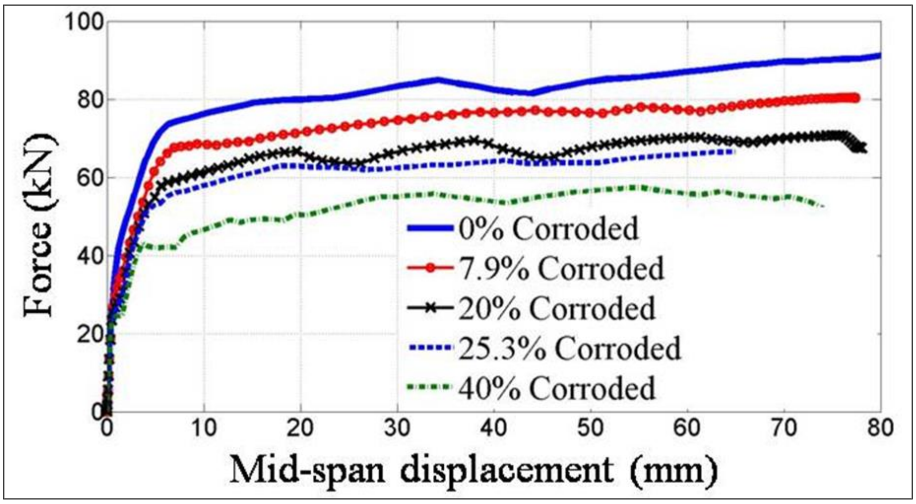

Figure 12. Comparison between the load-deformation responses of beams at different levels of corrosion

5. Retrofit Variants

Retrofit variant A, consists of inclusion of a CFRP strip with thickness and width given in Table 5 and constant length of 1700 mm to the soffit of the beam. Retrofit variant B in addition to variant A includes three CFRP plates on both sides with a thickness of 1 mm and width of 100 mm for shear strengthening. Shear CFRP plates center to center spacing is 150 mm. Retrofit variant C is similar to variant B except vertical shear CFRP plates is substituted by inclined shear CFRP plates to an angle of 45^0. Thickness, width and spacing of shear plates of variant C are same as variant B. Figure 13 presents detailing of each variant of retrofit. Width and number of strip layers are designed using SAP 2000 software package.

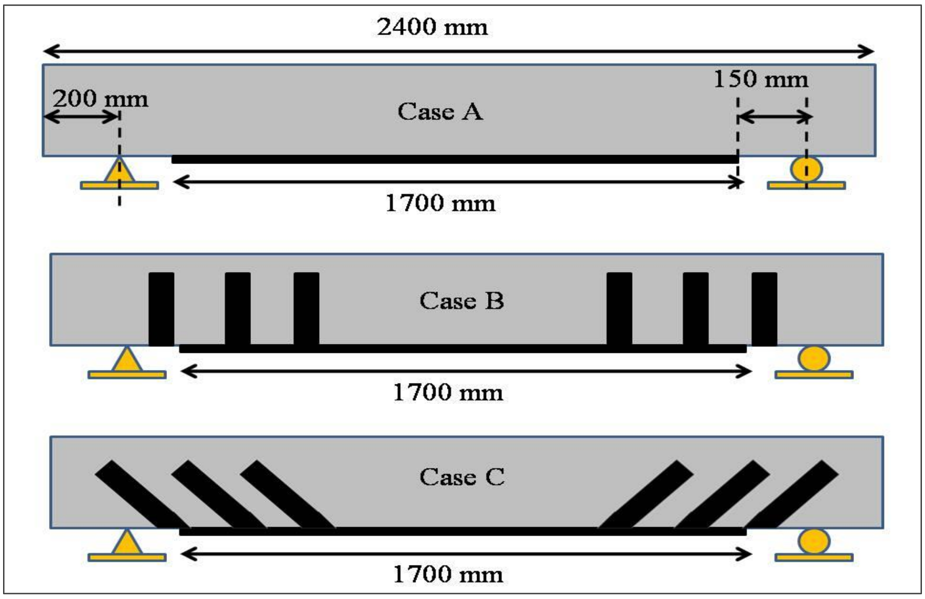

Figure 13. Different retrofit variants

Table 5. Flexural retrofit design for different corrosion degrees

Corrosion degree (%)	Flexural capacity before retrofit (kN.m)	CFRP strip width (mm)	Number of strip layers	Flexural capacity after retrofit (kN.m)
20	26.2	150	1	33.9
40	20.8	100	2	32.8

5.1. Behaviour with a Retrofitted Beam with 20% Degree of Corrosion

Figure 14 provides a comparison between the load-deflection curve of the beam specimen with 20% degree of corrosion before and after the retrofit. This plot reveals an anticipatable insignificant increase of initial stiffness due to small contribution of CFRP strips to the total moment of inertia of the section. All retrofit variants lead to more or less the same ultimate capacity. Moreover, shear retrofit causes more brittle behavior. However, this conclusion should not be generalized for all cases. Result demonstrated that beam did not require shear retrofit. CFRP strip de-bonds from concrete in higher deformation levels, which leads to decrease of the member strength to its preliminary before retrofitted strength. Similar to moment-curvature analyses, failure mechanism of retrofitted beams were recorded as failure of tensile reinforcements, followed by de-bonding of CFRP strips from the concrete (Table 6). Figure 15 shows the failure mechanism captured by numerical simulation for retrofit variant A.

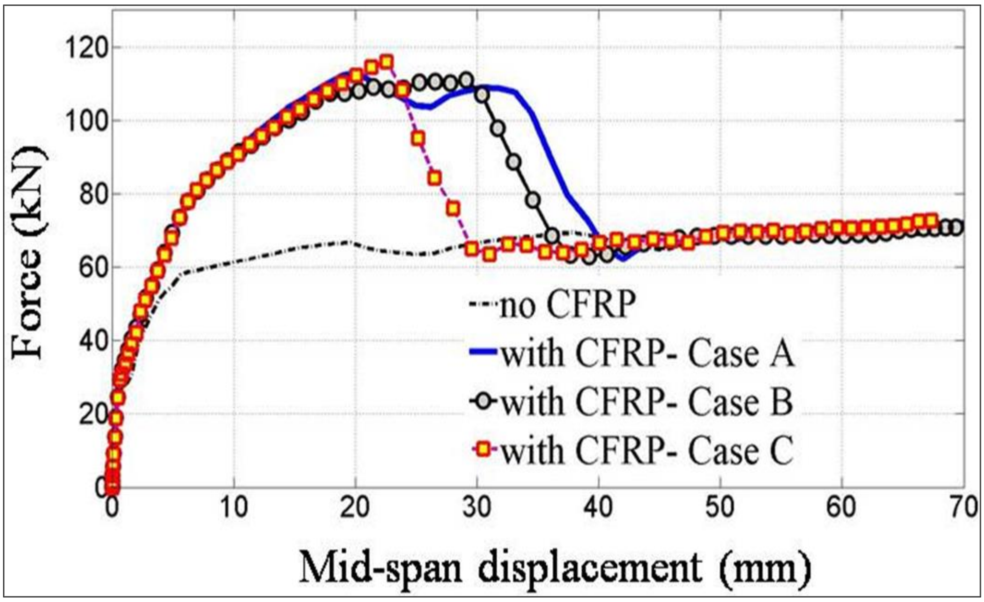

Figure 14. Behavior of the beam with 20% degree of corrosion using different corrosion variants

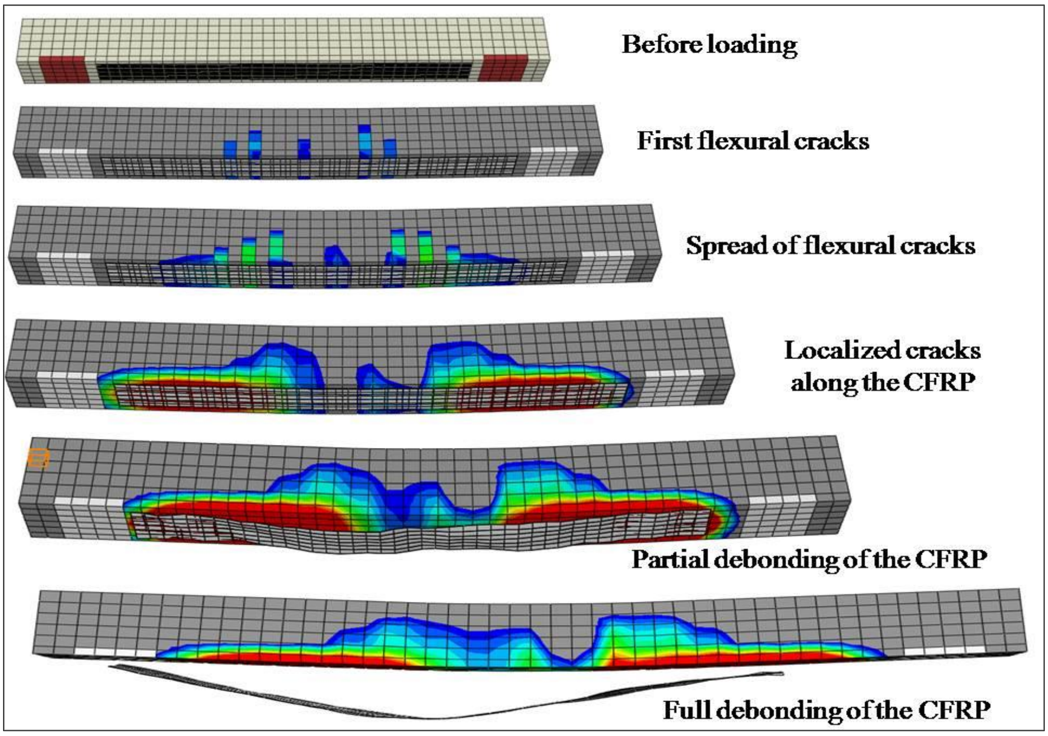

Figure 15. Crack propagation pattern and de-bonding of CFRP strip in the beam with 20% degree of corrosion using retrofit variant A

Table 6. Different variants of beam retrofit for the beam with 20% degree of corrosion

	No retrofit	Variant A	Variant B	Variant C
Ultimate load (kN)	70	113	111	116
Flexural strength (kN.m)	26.3	42.3	41.6	43.5

5.2. Behaviour with a Retrofitted Beam with 40% Degree of Corrosion

Figure 16 provides a comparison between the load-deflection curve of the beam specimen with 40% degree of corrosion before and after the retrofit. In contrast to the beam with 20% corrosion degree, the ductility of the beam remains unchanged after the retrofit. It can be attributed to the lesser ratio of CFRP strip width to the beam width resulting in higher bond strength and fracture energy between CFRP strip and concrete. In other words, the specimen with 40% corrosion ratio compared with the specimen with 20% corrosion ratio requires more excessive slips between concrete and CFRP strip to have CFRP strip de-bonded from concrete. As a result, CFRP strip is de-bonded in higher deformations. Moreover, the presence of shear CFRP strips does not contribute significantly to the beam behavior. This conclusion should be limited to this specific case and not generalized to other cases. Table 7 shows slight variations in the retrofitted beam moment and load capacity for different retrofit variants. Results demonstrated that retrofit design carried out by SAP 2000 targeted at a moment capacity of 32.8 kN.m, which is maintained in any of retrofit variants.

Table 7. Different variants of beam retrofit for the beam with 40% degree of corrosion

	No retrofit	Variant A	Variant B	Variant C
Ultimate load (kN)	58	92	91	90
Flexural strength (kN.m)	21.8	34.5	34.1	33.8

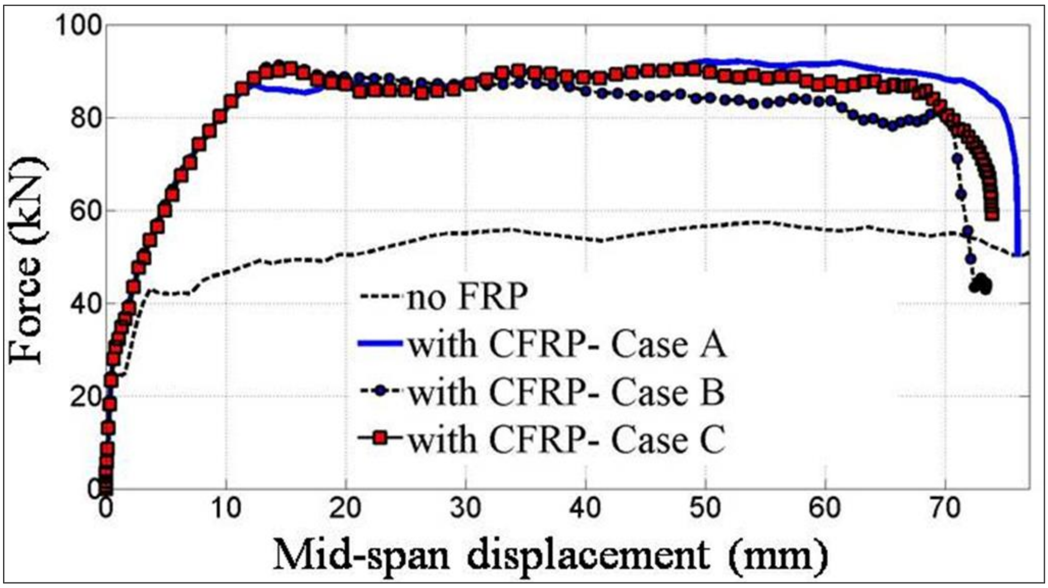

Figure 16. The behaviour of the beam with 40% degree of corrosion using different corrosion variants

6. Conclusion and Summary

Nonlinear modeling of various corroded reinforced concrete beam specimens, as well as CFRP, retrofitted corroded beams through three different alternatives is conducted. The following conclusion can be expressed:

- CFRP can be considered as an appropriate technique to retrofit corrosion damaged reinforced concrete beams. There is adequate evidence to support the role of CFRP to increase the capacity of structures. However, CFRP should not be overly used to have more than 40% capacity gain as recommended by ACI 440 [21].
- Bond slip modeling of specimens with corrosion degrees of 20% and 40% shows a reduction of 41 MPa and 42 MPa of the ultimate bond strength compared with the intact specimen.

- Bond slip modeling of beams studied herein does not considerably influence the beam capacity. It can be attributed to the concrete strength, and even specimen section property. It can be further understood by comparison between ABAQUS and SAP200 modeling results which is 26.3 and 26.2 kN.m. SAP 2000 analysis does not incorporate with slip between steel and concrete. As a result, one can quickly arrive at the negligible role of bond slip modeling in overall beam structural behavior.
- Corrosion deteriorative effects include cross-sectional area reduction of reinforcements, longitudinal cracks developed in the concrete due to the more voluminous product of steel rusting and decline in the bond between steel and concrete. Amongst others, cross-sectional area reduction of reinforcements plays the most prominent role. Ignoring the other two corrosion effects in numerical simulation has no tangible influence on the predictions.
- Moment capacity reduction of corroded specimens with 20% and 40% degrees of corrosion is 22% and 36%, respectively. In this study, one CFRP layer with 150 mm width and two CFRP layers with 100 mm width are used respectively to compensate for the loss of the moment capacity due to corrosion.
- Preliminary retrofit design targeted at overall beam moment capacity of 33.9 and 32.8 kN.m for beams with 20 % and 40% degrees of corrosion respectively, which are affirmed by detailed numerical simulation.
- It can be concluded that fracture energy and ultimate shear stress between CFRP plate and the concrete has an inverse relationship with FRP strip width. It was observed for the beam with 20% degree of corrosion, in which CFRP strip width was 150 mm, the ultimate shear stress was 3.9 MPa and fracture energy was 0.4 N/mm. It was observed for the beam with 40% degree of corrosion retrofitted with 100 mm CFRP strip, the former was 4.5 MPa and the latter was 0.53 N/mm. This finding can be attributed to the relationship of these parameters to the slip between concrete and CFRP strip.
- CFRP strip de-bonding took place in later stages for the retrofitted beam with 40% degree of corrosion. The width of CFRP strips is smaller for this specimen. This width has an inverse relationship with developed ultimate shear stress and fracture energy between concrete and CFRP strip. In other words, a decrease in the width of CFRP strip increases slips demand to have it de-bonded from the concrete.
- Incorporation with U-shaped CFRP strip for shear retrofit can reduce slip between underneath flexural CFRP strips and concrete resulting in higher moment capacity. 3.0 MPa increase in capacity was observed for retrofit variant B (shear CFRP strip on both sides).

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