THE FLEXURAL BEHAVIOR OF CORRODED OFFSHORE RC BEAMS REPAIRED BY DIFFERENT TECHNIQUES

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ABSTRACT

RC beams in a marine environment suffer deteriorations caused by the corrosion of reinforcement. Many researches have investigated this problem, yet few repairs techniques have been developed until now. The purpose of this research is to determine the effect of these deteriorations on the flexural behavior of RC beams. Also investigate rehabilitation of corrosion-damaged RC beams with different repair techniques. The flexural performances of the beams were studied. In this study, flexural test have been carried out to study the influence of reinforcing bar corrosion and cracking on ultimate strength of reinforced concrete beams, bearing capacity and ductility. There were totally 5 RC beams (300 mm × 150 mm × 2200 mm) constructed in this research, 4 of which were corroded by marine environment. The flexural behavior of these beams are studied at one level of corrosion of reinforcement bar. However, with progressive increase in corrosion, the ultimate strength, stiffness and toughness decrease rapidly for RC Beams subjected to flexural loading. The results show that the repair methods could provide better load carrying capacity for the beams with different techniques.

Keywords: Corrosion, cracking, bond strength, flexural behavior.

1. INTRODUCTION

From the mid-1900s and continuing to the present time, a vast number of reinforced concrete structures were in urgent need of repair and strengthening, due to either a change in use or structural degradation [1]. In addition corrosion and deterioration of reinforced concrete members in marine environment is a serious problem for structures [2]. Corroded reinforcement may cause cracking, delamination, and debonding of concrete [3]. A number of studies have been carried out on repair the corroded RC beams [4]. Widely used repair techniques for RC beams include concrete jacketing and carbon fiber reinforced polymer jacketing [4]. Some initially exploratory work has been done on the use of FRP to strengthen concrete structures in the early 1980s [5]. Lu and Xie exploited the wet laid-up CFRP sheets embedded to strengthen the RC beams, and the load effect of the strengthening RC beam has been assessed by the CFRP sheet measurements [6]. Ballim and Reid tested RC beams under simultaneous loading and reinforcement corrosion and noted that at 6.2% corrosion level, increasing the sustained load from 0.23 Pu to 0.34 Pu, the deflection ratio was increased from 1.42 to 1.7. They inspected for corrosion cracking at the end of the experiment and so they could not assess the evolution of cracking. [7]. In addition to its strong ability to increase the bearing capacity of the RC members, CFRP can also improve the durability of marine concrete members due to its effective prevention of chloride ion diffusion [8]. The corroded RC members involve the RC beams, [8], RC slabs [9] and RC columns [10], and the forms of the CFRP could be sheets, strips [8], or Near Surface Mounted (NSM) rods [11]. This paper reports the results of an experimental research of structural performance of reinforced beams with corroded reinforcements under service loads and their efficient repair techniques. Particular attention was paid to the bond between reinforcement and concrete. The effects of different strengthening method and CFRP layers on the behavior of the specimens are studied. The laboratory investigations were verified with the long-term (600 days) exposure investigations on reinforced concrete beams. Five beams were tested, each designed to investigate specific aspects of structural performance including stiffness and deflection under service loads and ultimate flexural loads at failure. The beams, having three bottom bars 12 mm and two top bars 10 mm were subjected to marine environment then corroded. After this process, a patch repair technique is proposed for the rehabilitation of the damaged concrete cover, while the corroded steel reinforcement was cleaned and protected with zinc or epoxy. Also, one beam was strengthened in flexure with CFRP sheet and tested to failure. The structural capacity of the repair corroded beam is compared with that of a non-corroded beam B1. The study addresses the following issue to contribute to monitoring and structural inspection: cracking load, peak load, beam stiffness, beam toughness as well as the corresponding deflections under serviceability loading. The excessive deflection of beams subjected to flexure due to the total steel-concrete bond loss in a high bending moment region and steel mass loss. The study concludes that CFRP repair technique has a significant influence on increasing the load carrying capacity of beams with corroded steel bars, this will be demonstrated through the following experimental program, which will describe and the results of each of them.

2. EXPERIMENTAL PROGRAM

Five RC beams were tested in current study. The tested specimens were 220-cm long RC beams with a 30 x 15 cm cross-section and reinforced detailing as shown in Figure 1. The experimental program consists of one beam as reference and remaining four beams were exposed to offshore climate and three of them were rehabilitated of corrosion with different techniques. The beams are as shown in Table 1.

The ingredients of the concrete are given in Table 2. Water / Cement ratio was 0.50. The average compressive strength and elastic modulus were 25 MPa and 22000 MPa at 28 days.

Figure 1. Reinforced detailing of RC beam.

Table 1: The details of tested specimens.

Beam	Beam description	Age at testing (days)
B-01	Control beam (unexposed to offshore climate)	28
B-02	Exposed to offshore climate without repaired	600
B-03	Exposed to offshore climate and repaired with Zinc and casting Grout	600
B-04	Exposed to offshore climate and repaired with Epoxy and casting normal concrete	600
B-05	Exposed to offshore climate beam and repaired with Zinc and strengthened with CFRP	600

2.1 Preparation of specimens:

The specimens were unmolded 24 hours after casting and kept in the laboratory atmosphere condition for 28 days, then 4 beams were transported beside offshore for one year before applying repair techniques. The compressive strength at 7 and 28 days were determined using standard cubic specimens. Each value represents the average of the results of three specimens.

2.2 Casting and curing:

The specimens were left in the port said university laboratory for 28 days and were cured continuously up to 28 days, by spraying water then transported for offshore climate in port said on Mediterranean Sea costal for 600 Days.

2.3 Repair Techniques:

Three different repairing techniques were investigated to find the most efficient one. Therefore, this part explains the three different techniques for repair using materials listed in the Table 1, corroded areas in the beams were repaired around steel bars after demolish old concrete and cast new suitable material. The steps of the three repairing techniques will be illustrated hereafter.

2.3.1 First Technique:

This technique (B-03) zinc rich epoxy was utilized to repair the corroded steel bars at defected areas as shown in Figures 2, the grout was cast to compensate the old concrete cover.

Figure 2. Beam B3 after it was exposed to offshore climate before being repaired.

The following steps were used,

1. Remove of the weak concrete cover at specified locations until reaching the reinforcing steel bars and remove corroded layer of the reinforcing steel bars using hand and power tools.

2. Apply two coats of anti-corrosion painting for the existing treated steel rebar and back of steel bars, Sika Zinc Rich -2 and clean the concrete surface at the treated areas by compressed air and water jet to remove any loose parts.

3. Apply the bonding agent and pouring Sika grout 214 as shown Figure 3 to compensate the concrete cover.

Figure 3. Beam B3 after being repaired with Sika zinc rich epoxy and Grout.

2.3.2 Second Technique:

In this technique (B-04) mastic epoxy was utilized to repair corroded steel bars at defected areas and new concrete was cast as shown in Figures 4.

The following steps were used,

1. Remove of the weak concrete at specified locations until reaching the reinforcing steel bars.

2. Remove corroded layer of reinforcing steel bars using hand and power tools. Then applying two coats of anti-corrosion painting for the existing treated steel rebar, (mastic Epoxy- Jota mastic 87) as shown in Figure 4.

3. Form work using plywood, pouring new concrete.

Figure 4. Beam B4 after being repaired with Sika zinc rich epoxy and adebond.

2.3.3 Third Technique:

In this technique using (B-05) zinc rich were used to repair corroded steel bars at defected areas and concrete was cast as concrete cover and then strengthened with CFRP.

The following steps were used,

1. Remove of the weak concrete at specified locations until reaching the reinforcing steel bars.

2. Remove corroded layer reinforcing steel bars using hand power tools, apply two coats of anti-corrosion painting for the existing treated steel rebar, mastic Epoxy- Jota mastic 87 as shown in Figure 5.

3. Form work using plywood, pouring new concrete.

4. Applying CFRP for strengthening the beams.

Figure 5. Beam B5 after being repaired with Sika zinc rich epoxy, adebond and CFRP.

2.4 Materials Properties

Measurements of mechanical properties of concrete and steel were conducted in accordance with the standard guidelines. Table 2 shows normal concrete mix ingredients.

2.4.1 Compressive Strength Test

The compressive strength of the concrete was determined by testing three 150 mm size cubes at ages 1, 3, 7, and 28 days for each mix. The test was carried out according to ASTM C 39 using a compression testing machine of 2,000 KN Capacity.

2.4.2 Splitting Tensile Strength Test

The splitting tensile strength was determined following the procedure outlined in ASTM 496/C 496M-2004. Cylinders (150 mm x 300 mm) were used.

The average compressive strength of the normal concrete based on ASTM C 39 [3] is 25 MPa and the average tensile strength is 2.60 MPa. The average yield stress of steel reinforcement is 400 MPa with a modulus of elasticity of 22 GPa (DIN 50145) [13] and the ultimate strength is 600 MPa. The mix ingredient of the studied concrete is shown in Table 2.

Table 2: Normal concrete mix composition.

Sand (kg/m^3)	500
Gravel (kg/m^3)	1200
Cement (kg/m^3)	350
Water (L/m^3)	175
W/C	0.50

2.5 Test Set-up

The non-repaired RC beam (B1) was loaded at age of 28 days. The corroded RC beam (B2) was loaded at 600 days after corrosion was completed. And, the repaired RC beams (B3, B4, and B5) were loaded at 600 days after being repaired. All specimens were loaded under 4-points loading with a total clear span of 2000 mm. The distance between loading points and shear span were 600 mm. Displacement dial gauges were installed at the middle of the specimens and at the location of loading points. 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 10 KN of load. The loading continued until the specimens failed. The overall view of the flexural test setup is shown in Figure 6.

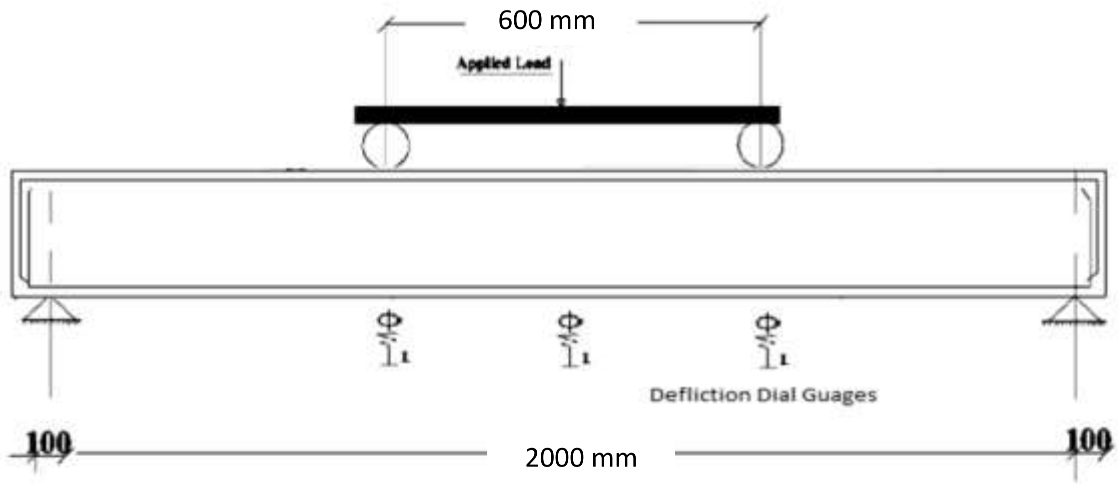

Figure 6. Test Set-up.

3. TEST RESULTS

This experimental program is conducted in order to study the behavior the flexural behavior of corroded and repaired reinforced concrete beams. Totally 5 RC beams for 28 and 600 days were used for finding flexural strength. The four tested beams have been carried out as show in Table 3.

Table 3: Tests results applied on the beams.

Beam	*Repair Type*	*Age* (day)	*Cracking load (kN)*	*Peak load P $_{Ultimate}$ (kN)*	*Beam stiffness (kN/mm)*	*Beam toughness (kN.mm)*
B1	----	28	46	67	21	32.45
B2	----	600	38	53	15	35.88
B3	Sika zinc rich epoxy and Grout	600	59	75	20	32.45
B4	Sika zinc rich epoxy and concrete	600	45	55	22	32.45
B5	Sika zinc rich epoxy and CFRP	600	49	61	24	36.46

* Beam stiffness is calculated as follows:

Result of average slope of (Load – Deflection) Curve for each beam.

** Beam toughness is calculated as follows:

Result of area under (Load – Deflection) Curve for each beam.

3.1 Flexural Behavior and Cracking Pattern

3.1.1 B1–Control Beam.

The beam showed a normal bending behavior without any difference from what was expected, since at the beginning of the loading did not show any bending and up to 3.0 *kN*, cracks began to appear at loading at approximately 46 *kN*, this is a close and good level of the highest rate of failure of the beam then increased cracks in the loading areas with increased load, with the appearance of cracks in shear areas in general. The results appeared, all normal and expected, for a concealed reinforced concrete beam 28 days after pouring it, the crack pattern is also normal. It is distributed regularly on the beam until it began to crumble at about 67 KN. Figure 11 illustrates the crack pattern.

3.1.2 B2– Exposed to Offshore Climate.

This beam showed different results compare to B1, as that beam was exposed to offshore climate conditions for about 600 days without any method of repair. Therefore, deflection showed since the beginning of loading at 1.0 *kN*, as a result of being severely affected by the environment in which they were present and erosion of the reinforcing beam, and this became clearer with the appearance of the first crack at about 35 *kN*, a difference of 10 *kN* from the control beam, then cracks quickly followed, with the appearance of fractures in the concrete cover and the appearance of cracks in the shear area with the increased loading other than the first beam. This shows the great impact of the attack of chlorides on it, it began to collapse the beam is at 53 *kN*, which means that the bearing force is about 20% less than the control beam. The bending appeared convergent under the two loads and increased in the middle by about 5.0 mm, but the bending did not go beyond the control beam, which explains why the steel bars greatly preserves its tensile strength properties despite It goes through environmental conditions that expose it to rust. In the following chart in figure 12 shows the crack pattern.

3.1.3 B3– Repaired with Sika zinc rich epoxy and Grout

This beam, despite being subjected to the same conditions as B2 it was exposed, showed different behavior, due to its repaired using grout, as the deflection started from the beginning of loading, but slowly increased unlike the beam 2 until cracks began to appear at approximately 60 *kN*, and it is considered a result that exceeds the control beam and the corpus exposed to rust but repaired by zinc rich, due to the advantages of characterized by the grout, as it is highly strengthening , and then cracks follow, including areas of shear. However, there was no break in the concrete cover as in B2, and there were no cracks in the area between the old concrete and the grout, which shows the efficiency of the bonding material. The beam endured up to 75 *kN*, increased on the control beam, at 11%. In the following figure 12 shows the crack pattern.

3.1.4 B4 – Repaired with Sika zinc rich epoxy and Concrete

In this beam, the bending behavior is similar to the behavior of the control beam, greatly in the way of bending and the appearance of cracks, but with the difference in the load of the crack and collapse, the load increased even with an increase in the deviation until the appearance of the first crack At 45 *kN* in the middle of the beam, cracks continued under the two loads in the middle and a link to the shear area until the collapse load was at 55 *kN*, approximately 18% less than the control beam, which means that the beam is greatly affected by exposure to coastal conditions despite the use of a maintenance method that includes Epoxy for steel exposed to rust in addition to concrete that did not reach the same efficiency as the control beam, and it proves that this method is less effective than its predecessor in B-03 with regard to the bearing strength of the beam and its resistance to collapse. In the following figure 12 shows the cracks pattern.

3.1.5 B5 – Repaired with Sika zinc rich epoxy and CFRP

This beam showed good performance when loading, as it showed different performance of cracks in terms of where they appeared, where the first crack appeared in the middle of the beam at about 50 *kN*, which means its preference over the control beam, in which the first crack appeared about 45 *kN*, by 10%, and then the cracks followed. It was concentrated in the middle of the beam and under the loads without any cracks appearing in the Shear area due to its reinforcement using CFRP which showed good and expected performance in the bearing strength of the beam and its resistance to cracks in the shear area. The beam began to failure at about 60 *kN*, which is not significantly less than the control beam despite the conditions experienced by the beam before the repair process with zinc for steel bars and casting concrete, and then reinforcing with fibers, where I decreased by approximately 6.0% with no cracks in the shear zone the collapse load was at 61 *kN*, In the following figure 12 shows the cracks pattern.

4. DISCUSSION

4.1 Deterioration due to Corrosion

By examining the corroded un-repaired RC beams, one longitudinal crack appeared at the bottom soffit of the specimen and the other crack appeared on the sides as shown in Figure 12. The average longitudinal crack width before sealing of the corrosion cracks was 0.4 mm. When the repaired specimens were exposed to further corrosion, the sealed cracks did not open, and there was hardly any other longitudinal cracking observed. Figure 5 clearly shows that the FRP repair process reduced the crack opening by about 88% at the end of corrosion process. This implies a significant enhancement in appearance of FRP repaired corroded specimens by reducing crack opening due to further corrosion.

Figure 7. Corrosion pattern in RC beam B2.

4.2 Effect of Corrosion on Flexural Behavior

The RC beams had corrosion along the whole length of the specimen. The flexural performance, with the exception of ductility, of the strengthened and corroded beams was improved. The behavior of specimens strengthened with transverse CFRP wrapping only as shows in Figure 5 . The continuous transverse laminate provided a small increase in the yield and ultimate strength as a result of the transverse strength of the laminate. The difference in yield and ultimate load between B1 and B5 is small. Thus, the continuous transverse sheet was successful at maintaining the majority of the yield and ultimate strength of beam B5. The yield and ultimate strengths of beam B5 was 9% less, respectively, than the corresponding values for beam B1.

Figure 9b shows the behavior of repair beam B3 with Grout. The tensile steel reinforcement of the strengthened specimens was corroded. The load-deflection response of the control beam B1 and the strengthened beam corroded B3 are shown for comparison. The strengthened beam exhibited increased stiffness over the strengthened beam, and marked increases in the yield and ultimate strength. However, the ductility was reduced in comparison to strengthened beam B3. The increase in yield and ultimate strength of the strengthened beam were on average 8% and 12%, respectively. The ultimate strengths of beams B2 and B4 were almost identical. The ultimate strength of beam B4 was only 4% greater than the ultimate strength of beam B2. Thus, the adding new concrete did not have any significant effect on the ultimate strength of the beam B4. Beam B4 exhibited ultimate strength, which were 18%, less than the ultimate strength of control beam B1.

4.3 Deflection of Beams

The load deflection relations for the beams are shown in the following Figures. Figure 8, shows compare the load deflection diagrams of control beams B1, B2 and beam B3 repaired with Epoxy and Grout technique. The beams behaved linearly until cracking load and the deflections were almost the same until the start of yielding of the tension reinforcement. In this post-yield phase, the greater the percentage of reinforcement corrosion B2, the greater was the deflection for a given load. Deflection values were less for repaired beam than for the beams without repaired at the same load.

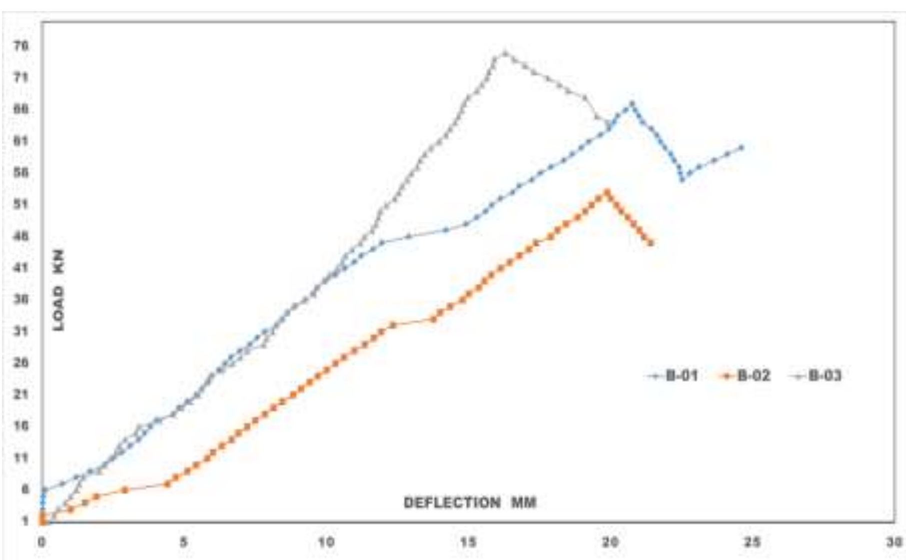

Figure 8. Load-Deflection relation for B1, B2 and B3.

From the Figure 9, it seems that adding the Epoxy and Concrete has had very little effect on the member stiffness. Also, the B4 showed similar behavior, the beam behaved linearly up to the cracking load, and later the yielding point was clearly detected.

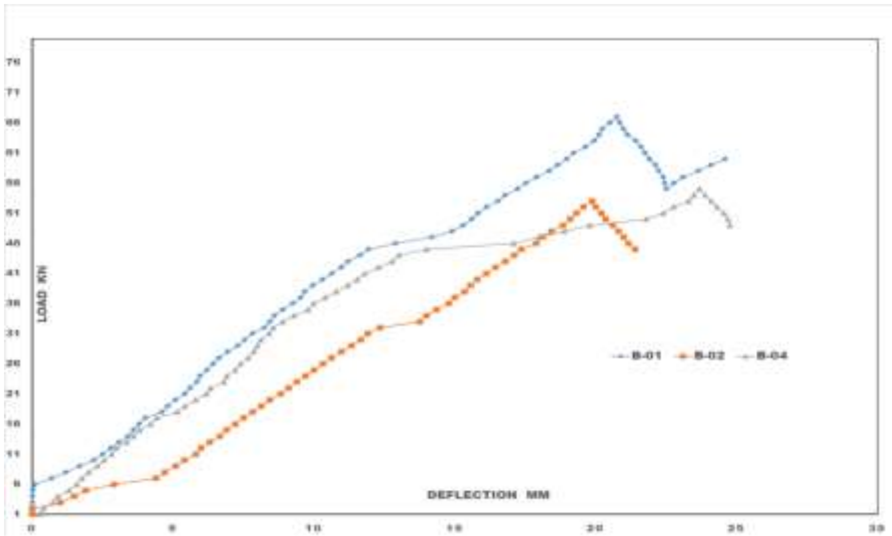

Figure 9. Load-Deflection relation for B1, B2 and B4.

The effect of bonding a CFRP to the beam is introduced in Figure 10. The beam was strengthened with CFRP, exhibited almost linear behavior. Deflection values were less for all beams than for the beams without strengthening at the same load. It may be concluded that composite action between the CFRP and the beam was achieved. This resulted in higher stiffness for the strengthened beam than the reference beam leading to an approximately elastic behavior of the strengthened beam up to failure.

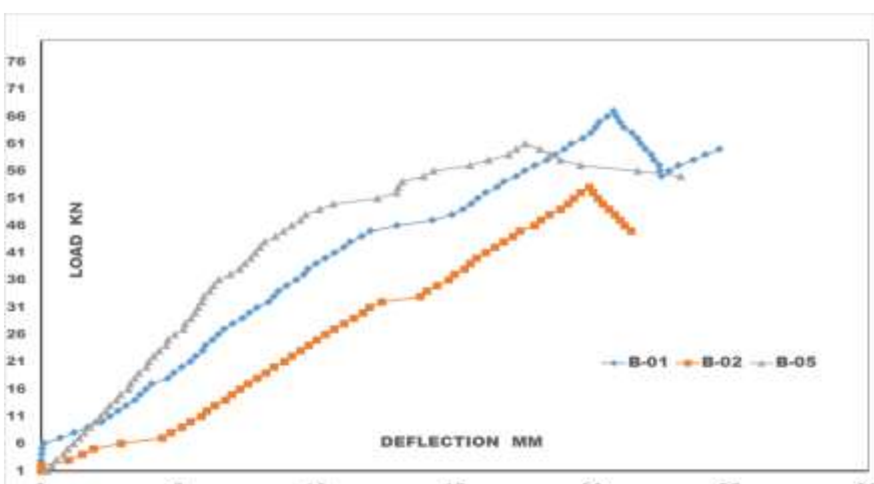

Figure 10. Load-Deflection relation for B1, B2 and B5.

4.4 Failure loads

Since one of the objectives of this research was to find the corrosion level affected the failure load and identifies the effect of strengthening the beams using different techniques on the failure loads. Figure 11 shows the comparing the failure load. It is seen that strengthening beams exhibited higher value of failure load than the unstrengthen beams at corrosion level.

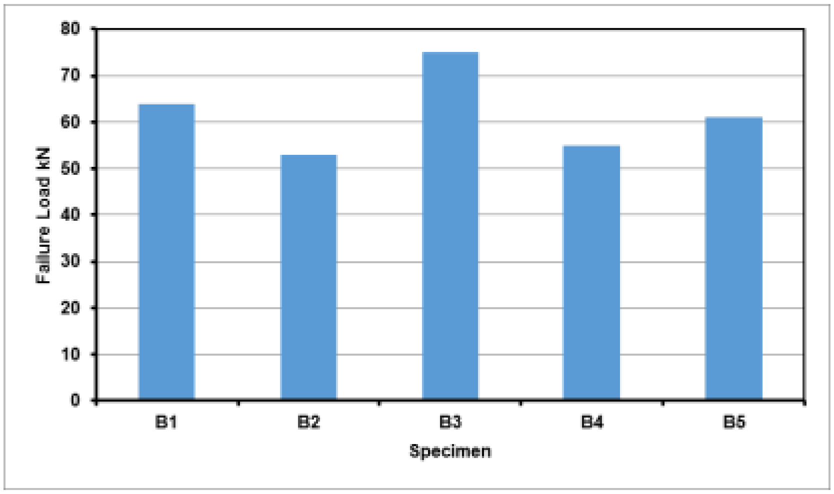

Figure 11. Effect of Different Techniques on Failure Loads.

4.5 Efficiency of Repairing Techniques

4.5.1 First Technique using Zinc Rich and Grout

In Table 4 shows the influence of the grout, as it made a increasing the ultimate load, as well as stiffness and toughness This proves the effectiveness of using grout as a repair method for concrete beams beside offshore. As for the cracks, they showed a pattern similar to B1. Upon increasing the load more flexure cracks starting just outside of repaired area the splice length and propagated towards the applied loads region as shown in Figure 11 and beam was failure at load 75 kN.

4.5.2 Second Technique using Epoxy and Concrete

Table 4 shows, the less effectiveness of the concrete repair technique compare to grout technique was used in B3. Figure 12 shows, B4 was showed similar cracking behavior as B1 and better than B2. Fine flexural cracks were formed at the middle of the beam and under loads as shown in Figure 12, and the maximum load was the 55 kN at failure.

Table- 4: Cracking and maximum Load of the beams.

Beam	Classification	Cracking Load kN	Maximum load kN	Beam stiffness kN/mm	Beam toughness kN.mm
B-01	Control Beam	46	64	2.86	988
B-02	Exposed to Offshore Climate	38	53	2.16	560
B-03	Repair Technique No.1	59	75	3.44	810
B-04	Repair technique No.2	45	55	3.81	868
B-05	Repair technique No.3	49	61	3.81	868

4.5.3 Third Technique Strengthening with CFRP:

This explains effectiveness of the concrete repair method strengthening with CFRP. As for the cracks in beam No. 5 .It showed the first crack observed at 5 ton. Fine flexural cracks were formed at the middle of the beam as shown in figure 12, the shear zone was almost free from cracks, and the maximum load is the 6.1 ton and starting failure , and completely different from the behavior of cracks in Beam No. 1, which showed cracks in shear area and in the middle of the beam, and this is the opposite of what we saw in Beam No.5 which did not show cracks in the shear area , it is also different from Beam No. 2, which showed cracks in the shear area with deterioration in the concrete cover .

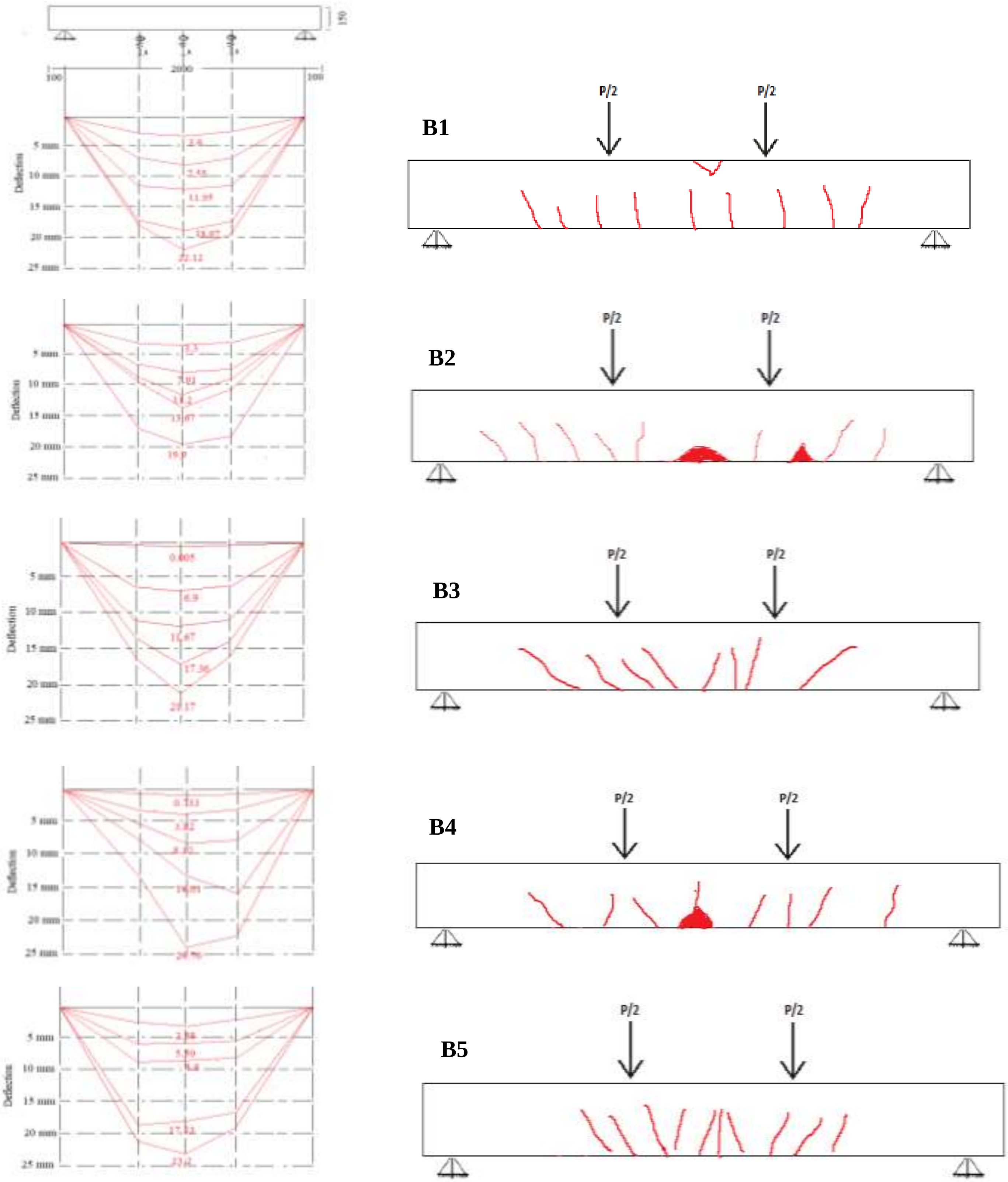

Figure 12. Load-Deflection Curves and Cracking Pattern.

5. CONCLUSIONS

Based on the study results, the following conclusions may be presented:

- According to the results obtained on B2 beam, concrete cracking created by the corrosion of steel reinforcements dose not significantly influence the mechanical characteristics of the reinforced concrete beams in service.
- However, results obtained on B2 beam ultimate behaviours show that the corrosion of steel has influence on reinforced concrete beams capacity. So, the residual bearing capacity can be assessed by taking into account the reduction in the tensile steel cross-section.
- Moreover, a ductility reduction by about 15% was observed on B2 beam. This effect is at least as unfavorable as is the reduction in ultimate strength to the safety of the RC beams. This loss of ductility could be attributed to a loss of ductility of the tensile steels due to corrosion damage.
- Corroded beam B2 showed cracking pattern different compared to another all beams, it involves heavy cracking and during loading the concrete cover fall out.
- A significant increase in the flexural strength of the RC beams can be achieved by bonding a CFRP sheets to the beam B5.
- Repair using Grout and zinc rich epoxy can efficiently reinstate or improve both strength and stiffness - B3.
- The CFRP sheets could affect the mode of failure of the beams as it reduces strengthened beam’s ductility - B5.
- The flexural stress appears to be the governing criteria for the failure of the strengthened beams B3, B4 and B5.
- When applying repair techniques on the defected beams B2, B3 and B4, the problem of corrosion reinforcement could be overcome, which then enabled the restoration of the structural integrity.
- By applying concrete repair, panting and cleaning steel bars for a beam B4 with corrosion, a non-significant amount of the load-bearing capacity could be restored.
- Grout repair and panting steel bars by sika zinc rich epoxy for a beam B3 with corrosion almost completely restored the load-carrying capacity.

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