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Flexural Behavior of Post Tensioned Beams Damaged by Reinforcement Corrosion before and after Applying Patch-Repair

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Abstract Corrosion is the major source of degradation in reinforced concrete structures, particularly in harsh environmental conditions. In literature, a number of research works has been carried out on the performance evaluation of the post-tensioned (PT) beams with the corroded PT cables, but very few are related to the PT beams with corroded plain reinforcement before and after applying repair. In this paper, the flexural behaviors of intact, corroded/deteriorated, and repaired PT beams are examined. In particular, the effects of patch repair with and without applying epoxy at the repair-substrate interface have been investigated. Based on the results, using high performance concrete materials as a patch-repair without epoxy at the repair-substrate interface applying can significantly re-store the structural performance of deteriorated PT beams.

Keywords: *post-tensioning, corrosion, spalling, patch-repair, epoxy*

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1. Introduction

Design of prestressed concrete structures has allowed the construction of economically competitive bridges with significantly longer spans and higher load-carrying capacities. The increase in capacity, however, has not been accompanied by a corresponding increase in durability, especially in harsh environmental conditions. One of the major causes of deterioration in reinforced concrete (RC) structures, located in such conditions, is reinforcement corrosion due to the ingress of aggressive ions into the concrete. Corrosion deterioration most often shows up in the form of cracking, delamination and spalling of the concrete that can in turn accelerate the rate of corrosion and significantly reduce the service life of the structure [1-7]. Reports show that the annual direct cost of corrosion for highway bridges in US is approximately $8.3 billion with estimated indirect costs due to traffic delay and lost productivity as much as 10 times that amount [8].

Deterioration of RC structures in the south of Iran, in the severe environment of the Persian Gulf region, has also been noticeably observed in recent years that has resulted in reducing the service life of the structures and increasing the life cycle costs [9]. In particular, some of the RC structures in the Persian Gulf region are post-tensioned (PT) structures. Due to the insufficient thickness of concrete cover in these structures (about 4cm), the most prevalent form of deterioration in them is the spalling of concrete around the corroded steel rebars. Note that the consequences of plain reinforcement corrosion in PT members are much more serious than RC (reinforced concrete) members, since the prestressing tendons are typically stressed to as much as 80% of their ultimate strength. Accordingly, the spalling of concrete can increase the stress in the remaining prestressed tendons, leading to strand breakage without warning and subsequently release of force carried by tendons.

Furthermore, concrete cover spalling from a PT bridge can permanently increase stresses in the remaining original concrete and change both primary and secondary prestress moments [10]. Regardless of the type of corrosion (tendon or plain reinforcement), corrosion can also reduce the yielding strength and ductility of the rebars, and deteriorates the bond at the steel-concrete interface [11]. It is well noted that with increase of transverse cracks on the underside of the beams caused by the flexural tension and the expansive stresses induced by corrosion products, there is an increase in deflection of RC beams during the early stages of corrosion [12]. These effects can significantly reduce the service life of the structure and even change the failure mode of the structure from ductile to brittle. In literature, a number of research works has been carried out on the performance evaluation and durability design of corroded RC/PT structures.

In order to determine the strength capacity of the corrosion damaged RC sections subjected to bending, Capozucca [13] proposed two methods, which involves the characteristics of both direct method and inverse method. It has clearly shown that the damage caused by corrosion of the concrete around the bars produces a compressive softening effect of the concrete and it significantly decreases the strength and durability of RC elements. Capozucca and Cerri [14] described a theoretical model to analyze the influence of reinforcement corrosion in the compressive zone of RC beams considering a modified law for concrete. They compared theoretical and experimental data obtained from bending tests on undamaged and damaged RC beam models. Moreover, the static and dynamic tests were carried out on undamaged and artificially damaged prestressed RC beams by applying corrosion in the reinforcement located in compressive concrete [15]. To determine when corroded RC structures should be repaired, the reduction in both strength and stiffness must be known [16].

On the other hand, Coronelli et al. [17] evaluated the performance of corroded post-tensioned beams with bonded tendons and wire failure. Castel et al. [18] proposed two numerical procedures based on the micro finite element using cross-section analysis and nonlinear finite element analysis for structural performance prediction of corroded un-bonded posttensioned beams. Sajedi and Huang [19] proposed an analytical procedure to predict the nonlinear flexural behavior of intact and corroded RC bridges. The procedure is capable of predicting load-deflection behavior of RC beams with and without lap splices. Sajedi et al. [11] developed a reliability-based multi-objective optimization technique for design of cost effective RC structures in corroded regions. They utilized this technique to compare using three different materials including normal strength concrete (NSC), high strength concrete (HSC), and epoxy coated (EC) reinforcement in design of durable RC bridges and concluded that using HSC can significantly reduce the life cycle cost of the structure compared to NSC and EC. Few studies, however, has been conducted on flexural behavior of corroded/spalled PT beams before and after applying repair.

In general, the techniques used for flexural performance evaluation of corroded PT beams can be categorized into two groups: experimental studies and numerical analyses (e.g., finite element modeling, analytical procedures, novel evolutionary algorithms, etc. [20,21,22,23]). In this paper, an experimental study is conducted to evaluate the effects of plain rebar corrosion and concrete removal on the load-carrying capacity and failure modes of PT beams. Five PT beams are tested, including one intact, one corroded, one corroded/deteriorated, and two repaired beams. Accelerated corrosion test is used to simulate the corrosion in plain reinforcement.

For two repaired beams, the contaminated concrete is removed beyond the corroded rebar and then they repaired using patch-repair procedure, one with and one without using epoxy at the repair-substrate interface. The structural performance of all beams is then studied and compared to the control and deteriorated (non-repaired) beams.

2. Research Significance

Despite the numerous studies on the effects of tendon corrosion on the structural response of PT beams, there are still many questions regarding the deterioration of PT components due to chloride-induced corrosion of plain reinforcement. In addition, repairing is an important factor in extending the service life of deteriorated structures. However, the effectiveness of patch repair on the structural performance and durability of such structures, which have vast amounts of concrete spalling, has not been fully evaluated yet.

3. Methodology

3.1. Materials and Testing

The cementitious materials used in this study for repair concrete and substrate were portland cement (PC) equivalent to ASTM Type I, slag blended cement and silica fume (SF). The chemical compositions and properties of these types of cementitious material are shown in Table 1. Crushed limestone aggregate with maximum size of 16 mm and river sand were used as coarse and fine aggregates. Polypropylene fibers (1%) were added into the repair concrete mix to control shrinkage. Table 2 shows the mixture proportions of the repair and substrate concrete used in this study.

Table 1. Chemical compositions (%) and properties of binding

Binder	SiO_2	Al_2O_3	CaO	MgO	Fe_2O_3	SO_3	Na_2O	K_2O	Specific gravity
Cement Type I	21.60	5.40	62.60	3.600	3.40	1.90	0.78	0.44	3.10
Slag	35.50	10.00	36.50	9.500	0.70	1.86	0.50	0.53	2.86
SF	93.16	1.13	-	1.60	0.72	0.05	-	-	2.11

Table 2. Mix properties of substrate and repair concrete

	Cement Type	Cement (kg/m3)	Silica fume (kg/m3)	Super-plasticizer (%)	Polypropylene fibers (%)	w/b	Fine Aggregate (kg/m3)	Coarse Aggregate (kg/m3)
Substrate	I	440	-	0.7	-	0.34	1161	720
Repair	Slag blended	420	29.4	0.6	1	0.38	980	793

The repair material was selected based on previous studies. The procedure consisted of the evaluation of three performance categories: 1) mechanical properties, 2) durability characteristics, and 3) dimensional stability (shrinkage and creep). In particular, dimensional stability is important in cracking tendency and therefore degradation of repaired reinforced concretes [24,25].

In lack of experimental data, one may use recently developed models for prediction of mechanical properties [26,27,28,29] or time-dependent deformations of concrete

[25]. Each parameter was investigated by carrying out appropriate tests on 4 types of concrete with different mix designs and proportions. A weighted performance index approach was used to rank each type of concrete in terms of overall performance. Based on their study, the combination of silica fume and slag showed the best performance among other materials and could improve the properties of repair concrete at both early and later ages.

Samples for mechanical testing of materials were prepared in accordance with ASTM C192 and then were evaluated through a set of well-known experimental procedures including: cylindrical compressive strength (according to ASTM C39), Brazilian splitting tensile strength (according to ASTM C496), flexural strength (according to ASTM C78), and modulus of elasticity (according to ASTM C469). Performance of bond between substrate and repair concrete was studied using Pull-off test (according to ASTM C1583) and Bi-Surface shear test method [30].

The proposed Bi-surface shear test included casting cubic concrete specimens in two steps: First, one-third of the mold was filled with Styrofoam and the remaing two-thirds of each mold was filled with substrate concrete (See Figure 1-a). The substrate specimens were removed from the molds after 24 hours and their surfaces were roughened using steel wire brush (Figure 1-b). Specimens were then cured in water for the next 28 days. Afterwards, they were kept in dry conditions (23 ºC temperature and 45% relative humidity) for 150 days in order to simulate conditions similar to existing substrate concrete [31].

Subsequently, the surface of substrate concrete was cleaned using a wire brush and water-jet a couple hours before placing the repair concrete. This step is carried out so as to be certain that the bond surface is clean and saturated (Figure 1-c). The remaining third of the mold was filled with repair concrete. After the curing process, the Bi-Surface test was conducted as shown in (Figure 1-d). The main purpose of this test is to investigate the bond strength between repair and substrate concrete and so, it was also carried out on integrated cubic samples with substrate concrete mix design.

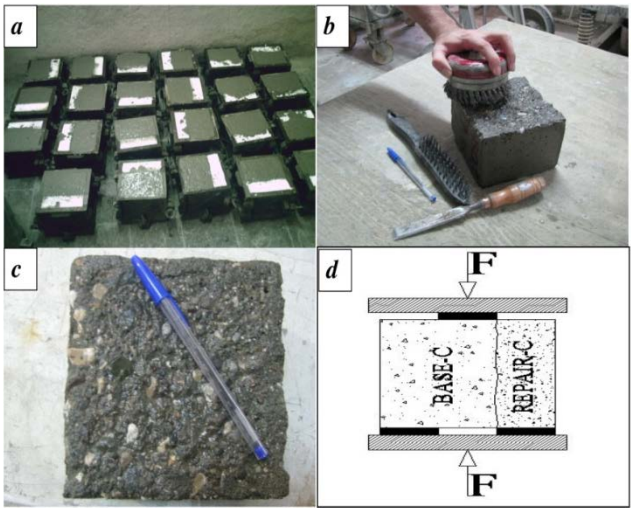

Figure 1. Photos and details of the Bi-Surface shear test procedure

3.2. Specimen Preparation and Reinforcement Arrangement

All beams were prestressed with two 7-wire low-relaxation strands with nominal diameter of 12.7mm, yield strength of 1600 MPa, and ultimate strength of 1860 MPa. Cement grout was used to fill galvanized ducts of prestressing strands. The deformed bars with 14mm and 12mm diameter with yield strength of 400 MPa were used as tensile and compression reinforcements, respectively.

The shear reinforcement was 12mm diameter round bar. Cover thickness from specimen surface to rebar was 4cm which is similar to the concrete cover in constructed PT structures of Persian Gulf region. The beams were cast in wooden molds with equal width, height and length of 300mm, 300mm, and 2500mm, respectively, and de-moulded after 24 hours. They were then moist cured for seven days and subsequently the strands were pretensioned up to 139.5 KN which is about 75% of their breaking load. The geometry of the PT beams and reinforcement layout are shown in Figure 2.

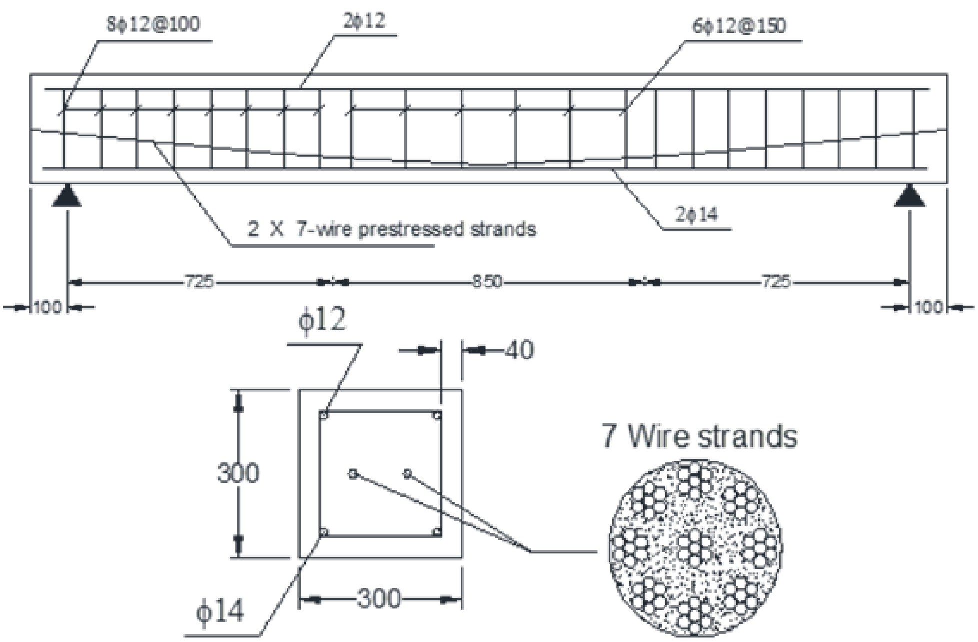

Figure 2. Geometry of the PT beams and reinforcement layout (mm)

3.3. Experimental Plan of Beams

The flexural behavior of deteriorated and repaired PT beams were investigated and compared with the control beam. Table 3 shows a list of tested specimens and their specifications.

Table 3. List of PT specimens

Beam ID	Category
F1	Un-corroded (Control Beam)
F2	Corroded
F3	Corroded, deteriorated and cleaned
F4	Corroded, deteriorated and cleaned, repaired without using epoxy
F5	Corroded, deteriorated and cleaned, repaired using epoxy

The corrosion was induced for beams F2 to F5 by an electrochemical corrosion technique to accelerate the corrosion of steel bars embedded in the specimens. For beams F3 to F5, the contaminated concrete was then removed beyond the reinforcement until sound concrete was reached. The F4 specimen was then repaired with high performance concrete by patch repair method without the use of epoxy at the interface between the base and repair concrete. The F5 specimen was patch repaired after applying epoxy at the repair-substrate interface. Subsequently, non-corroded, corroded and repaired PT specimens were tested to investigate their flexural performance under monotonic loading. Detailed descriptions of each process are given in following sections.

The aim of constructing and testing beam F3, which is deliberately and manually deteriorated after corrosion, is to consider extreme effect of corrosion (spalling of concrete cover) on the performance of the structure. Based on previous evaluation of structures in Persian Gulf region, in some cases, the concrete cover is completely destroyed and the reinforcement is exposed as shown in Figure 3. In order to create similar conditions in the lab, the concrete cover on beam F3 is completely removed until the reinforcement can be seen.

Figure 3. Deterioration of beams and destruction of concrete cover in a structure situated in the Persian Gulf

3.4. Accelerated Corrosion Process

An electrolyte corrosion technique was used to accelerate the deformed steel corrosion. The specified portion of beams F2 to F5 were submerged in the 5% NaCl solution as the electrolyte in a plastic tank. Power supplies with adjustable voltage and direct current of 600 mA were chosen for the electrolyte corrosion process. The positive side was connected to reinforcement cage and the negative voltage was connected to stainless steel plate. The corrosion was limited to the central zone with length, width and height of 85cm, 30cm and 7.5cm, respectively so that the prestressed strands were not corroded. In actual structures under severe corrosion the whole beam would have usually corroded. However, the focus of this study is on the flexural capacity of the beams and so only the middle section (which has maximum moment) of each beam was corroded.

The desired degree of corrosion was 15% for the deformed bars which can occur in a period of 40 days according to Faraday's law. However, based on the experiments carried out by Rinaldi et al. [32], the desired corrosion level in reinforced concrete is reached in a period of time that is twice as much that is required by Faraday's law.

So, the corrosion period for the beams was extended to 75 days. Figure 4 shows a photo of the experimental setup of accelerated corrosion test and Figure 5 shows details of the accelerated setup.

Figure 4. Photo of setup for accelerated corrosion test

3.5. Patch Repair Procedure

Different techniques can be used for repair of damages structures (e.g. [33]). In this study, damaged concrete was removed to a depth of approximately 8-10cm from 85-90cm mid-section of the F2 and F3 beams. The reinforcement surface was then cleaned using a water jet in order to remove surface micro-cracks and dust.

Finally two of the beams (F4 and F5) were repaired with patch repair method. The patched portion was then wrapped by plastic sheets for seven days in order to control restrained shrinkage in repair concrete. Figure 6 shows a photo of beams F4 and F5 before and after applying patch repair.

3.6. Flexural Loading Test

PT beams were subjected to a four point flexural testing, 28 days after the application of patch repair. Load was applied in increments by a hydraulic jack and measured with a load cell as shown in Figure 7. Two LVDTs were placed on both sides at the middle of the beams and two LVDTs were used to measure the deflection under the two loading points. The output data was recorded every 3 seconds using a data logger.

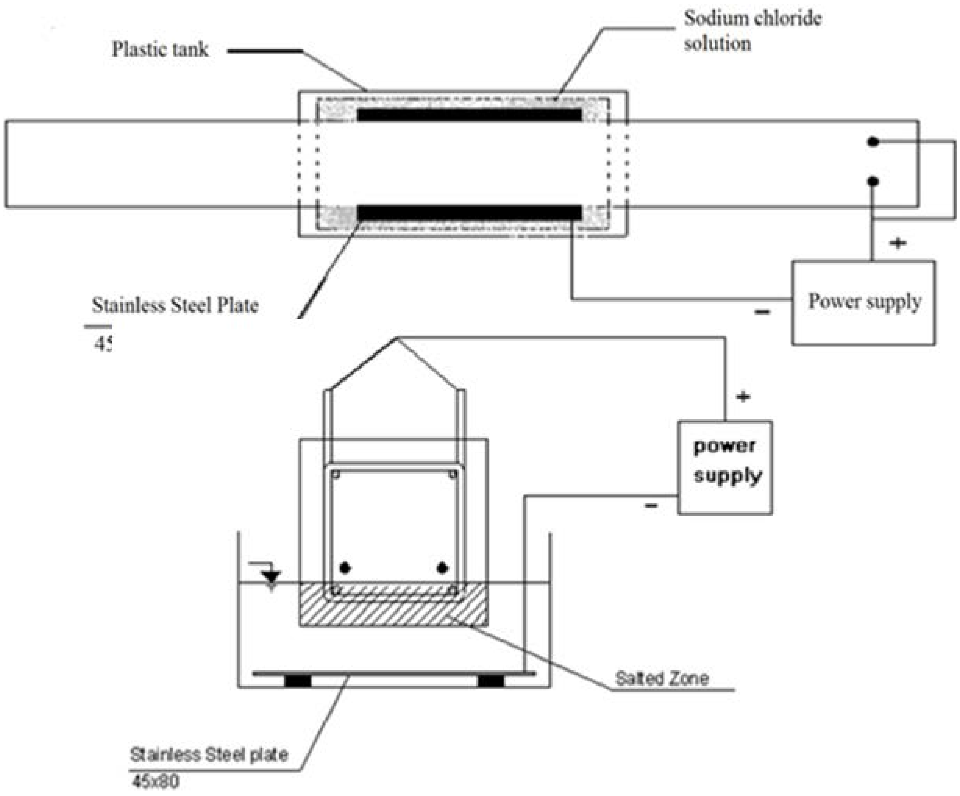

Figure 5. Details of accelerated corrosion test setup

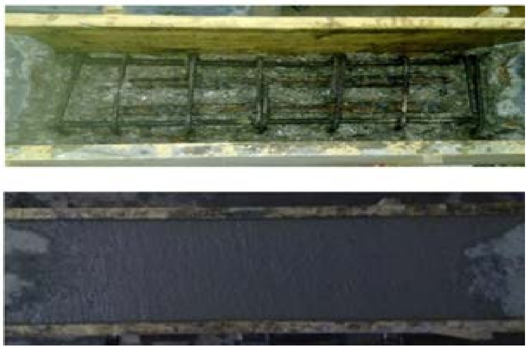

Figure 6. Photo of corroded PT beam before and after repairing

Figure 7. Flexural PT beam test set

A total clear span was 2300mm and the distance between two loading points was 850 mm. The loading continued until the specimens failed. Figure 8 shows details of the loading test setup. In order to obtain a displacement for the mid-span of the beam, the values obtained from the two midspan LVDTs were averaged for each record. The other two LVDTs, under each loading point, were used to calibrate results with numerical simulations.

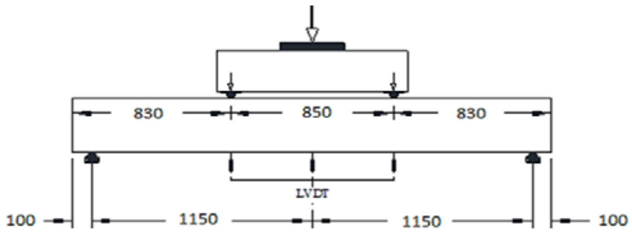

Figure 8. Flexural load test details (in mm)

4. Experimental Results and Discussion

4.1. Mechanical Properties

Table 4 shows the mechanical properties of repair concrete and substrate and Table 5 shows the bond strength test results for the interface between the repair and substrate concrete. Based on the results, in spite of effect of w/b, the mechanical properties of repair concrete are better than the substrate concrete. This is due to the synergistic effect of silica fume and slag on the mechanical properties of repair concrete [2].

Table 4. Mechanical properties of repair and substrate concrete

	Substrate	Repair
Compressive strength (MPa)	60.0	64.0
Flexural strength (MPa)	2.90	3.7
Tensile strength (MPa)	4.4	5.3
Modulus of elasticity (GPa)	37.0	35.0
Tensile bond strength (MPa)	5.2	
Shear bond strength (MPa)	3.0	

Table 5. Bond strength test results for repair and substrate concrete

	Bond strength between repair and substrate concrete (MPa)	
	Without epoxy at interface	With epoxy at interface
Pull-off test	5.2	0.6
Bi-Surface shear test	3.0	0.3

Sufficient bond strength between repair and substrate concrete is crucial in preventing delamination and spalling of concrete at an early age. The addition of silica fume to the selected repair concrete can improve the microstructure of the interface between repair and substrate concrete at early ages. The addition of slag can also improve long-term bond strength between repair and substrate concrete.

It is worth noting that, based on the results shown in Table 5, using epoxy resin adhesive can immensely reduce the bond strength between new and old concretes. Preparing patch area generally involves removing loose particles, cleaning up the concrete faces and reinforcements, setting up formwork, and placement of new concrete. The method of epoxy application and its type can have a great effect on the bond strength at various interfaces. Nevertheless, in this study, the epoxy was applied on completely dry concrete substrate according to the instructions of its provider. The results of research from various studies also back up the point that epoxy can sometimes reduce bond strength at interfaces for concrete.

4.2. Degree of Corrosion

A 15% degree of corrosion was desired in this study. In order to investigate the exact degree of corrosion, after conducting the flexural tests on PT beams, the corroded rebars were cut and extracted from the corrosion area and then were cleaned as shown in Figure 9.

Figure 9. Corroded reinforcement, extracted from specimens after conducting all tests. Top: Longitudinal deformed rebars. Bottom: shear deformed rebars

Exact weight and length of the extracted corroded deformed rebars were measured and the degree of corrosion was determined. Table 6 shows the exact degree of corrosion in the deformed rebars of beams F2 to F5. As seen from the table, the average degree of corrosion for longitudinal and shear reinforcement was 15.37% and 15.75%, respectively. This degree of corrosion is very close to the predicted corrosion from the corrected form of Faraday's law.

Table 6. Degree of corrosion in PT beams

Type of rebar	Beam F2	Beam F3	Beam F4	Beam F5
Longitudinal	14.5%	14.0%	16.1%	16.8%
Shear	16.6%	14.9%	15.5%	16.0%

4.3. Flexural Behavior of Investigated Beams

Table 7 shows the ultimate load-carrying capacity of beams and their relative mid-span displacement and Figure 10 shows a graph which compares the ultimate load capacity of each beam. Figure 11 shows the comparison of the load- deflection responses of PT beams.

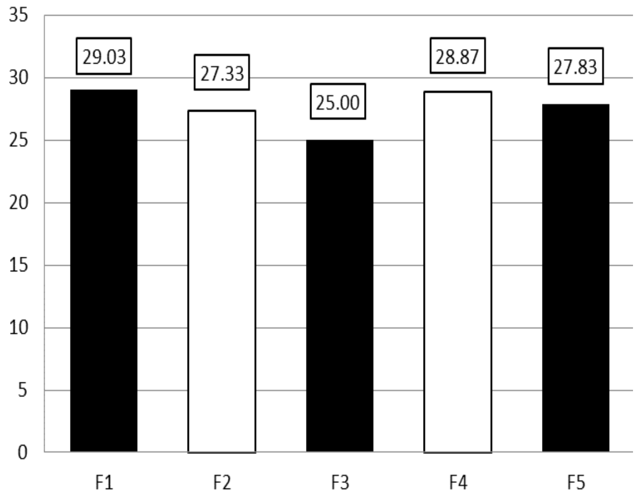

Figure 10. Ultimate loads of PT beams (tons)

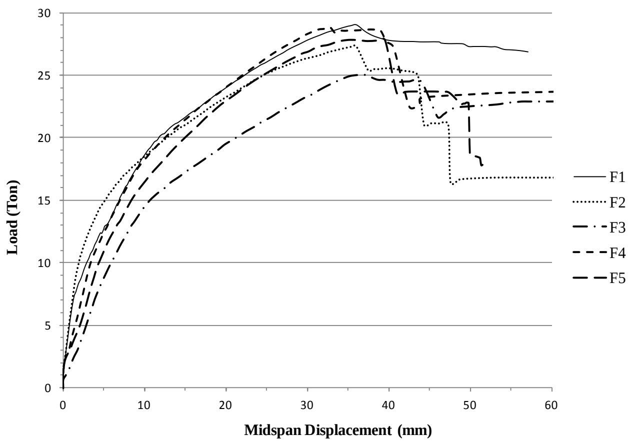

Figure 11. Comparison of load-displacement response of PT beams

The description of failure of each beam as follows:

Table 7. Ultimate loads and maximum mid-span displacement of each specimen

Beam ID	F1	F2	F3	F4	F5
Ultimate Load (tons)	29.03	27.33	25	28.87	27.83
Max mid-span displacement (mm)	36	36	36	33	40

4.3.1. Beam F1 (Control Beam)

In beam F1, flexural cracks were formed in constant moment region in tensile zone and extended upward followed by the formation of shear cracks and crushing of concrete in the compressive zone. The length and width of cracks were increased dramatically by increasing the flexural load until eventually the deformed bars yielded.

The displacements of the beam began to increase at a higher rate as the loads increased and greater deflections occurred at the mid-span of the beam when the presressed cables yielded. By applying higher loads, the width of flexural cracks increased and subsequently the failure of tendons occurred as the beam approached failure.

4.3.2. Beam F2 (Corroded Beam)

Table 7 shows that the ultimate capacity of the corroded beam F2 has decreased by 2 tons in comparison with the control beam F1. In PT beams, the portion of load sustained by the PT tendons is much larger than the longitudinal rebars. As a result, a significant decrease in ultimate load capacity is not expected from the beams under investigation because the tendons were not corroded and fully maintained their tensile strength after the corrosion process.

Surprisingly, there is a periodic increase in flexural capacity between 7 and 17 ton loads for the corroded beam F2 in comparison with the control beam F1 (See Figure 11). This difference can be explained by an increase in surface roughness due to corrosion, which can cause an increase in bonding between corroded reinforcement and concrete. Furthermore, this change can be relevant to the increasing the curvature and decreasing the stiffness of the corroded beam section [15]. This phenomenon is usually observed before cracking occurs and disappears with the increase in the number of cracks. In any case, the ultimate energy absorption of the corroded beam is less than the control beam**.**

4.3.3. Beam F3 (Corroded and deteriorated)

Figure 11 shows that there is a change in graph gradient for beam F3 at a load of 15 tons, where the tensile longitudinal rebars have yielded. The rupture mechanism of PT tendons for beam F3 differed from other specimens. In beams F1 and F2 beams the wires within each strand ruptured simultaneously, however the wires of strands in beam F3 ruptured one by one as shown in Figure 12.

Figure 12. Right: Rupture of strands for F3 beams, Left: Other specimens

As seen in Figure 11, steel corrosion and removal of concrete from beam F3 have lead to reduction of flexural stiffness and the yielding load of deformed bars and prestressed tendons. There is a 13.7% and 7.5% reduction in flexural capacity of beam F3 in comparison with F1 and F2, respectively. This reduction is due to a decrease in cross-sectional area and moment inertia at corroded segments of this beam. Furthermore, in this zone, the bonding between concrete and reinforcement had been reduced by manual deterioration. In addition, stress release within the tendons, due to the reduction in local cross-section reduction of beams, has also lead to a decrease in flexural capacity.

4.3.4. Beam F4 (Corroded, deteriorated and repaired without using epoxy)

After applying patch repair material to the deteriorated areas, it was found that the load- carrying capacity and flexural rigidity of deteriorated beam can be greatly restored (about 99.7% of un-corroded beam's capacity). The load-bearing capacity of the repaired beam (F4) was slightly lower than that of the uncorroded control beam(F1), but higher than the deteriorated beam(F3).

The reduction of cross-section for the reinforcement of this study (15%) was not high enough to greatly affect the flexural capacity of these beams. Instead, it was the reduction of bond strength between the reinforcement and concrete that reduced flexural capacity in reality. The corrosion smoothened the surface of deformed bars which lead to a reduction in bond strength. The cracking of concrete, due to expansive corrosive substances, also helped degrade the concrete and its bond with reinforcement.

In this study, the corroded deformed bars were not replaced by sound reinforcement before patch repair. So it seems that the main cause of reduction in the flexural stiffness and load-carrying capacity of beam F3 was the removal of concrete from the tensile zone, not the corrosion of deformed bars. However, the repaired beam F4 did behave differently after the yielding of prestressed steel in comparison with beam F1. It is probably due to the abrupt rupture of corroded rebars which has led to the brittle failure of the beam after the yielding of prestressed tendons.

4.3.5. Beam F5 (Corroded, deteriorated and repaired using epoxy)

Based on the frequent application of epoxy in repair and rehabilitation projects, it was thought to have a positive effect on the interface between the repair and base section of the beam. The results showed that, not only did epoxy fail to improve the concrete interface, in fact it helped to degrade the interface quality between the repair and substrate concrete. Figure 13 shows that cracking began at this interface in the concrete and after further loading, this part of the concrete completely opened up, and the beam showed shear failure. The results of pull-off tests and Bi-Surface shear test method [23] in this study, have previously confirmed the poor performance of epoxy.

Figure 13. Ultimate cracking and failure of beam F5\

4.4. Crack Patterns

The crack lengths of beam F2 at 60% of its ultimate load are longer in comparison with the control beam (F1).

It is worth noting that, during the accelerated corrosion process, cracks due to corrosion expansion were induced within corroded sections of beam F2. Due to the degraded quality of the corroded concrete, flexural cracks can form easier along the lines of previous corrosion cracks.

Similar crack patterns were observed for beam F4 in comparison with the control beam F1 and no significant cracking/debonding was observed at the repair/substrate concrete interface.

This is probably due to the higher mechanical properties of the applied patch repair and sufficient bond strength at the interface. Figure 14 shows the final crack pattern of beam F4, which represents the flexural failure mode. This was not the case for beam F5, in which the crack at the interface completely opened up after yielding (See Figure 13).

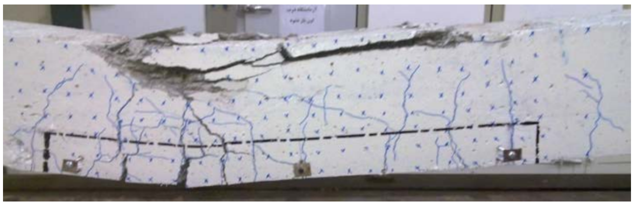

Figure 14. The final crack pattern and failure of beam F4

The schematic ultimate crack patterns of the PT beams are shown in Figure 15.

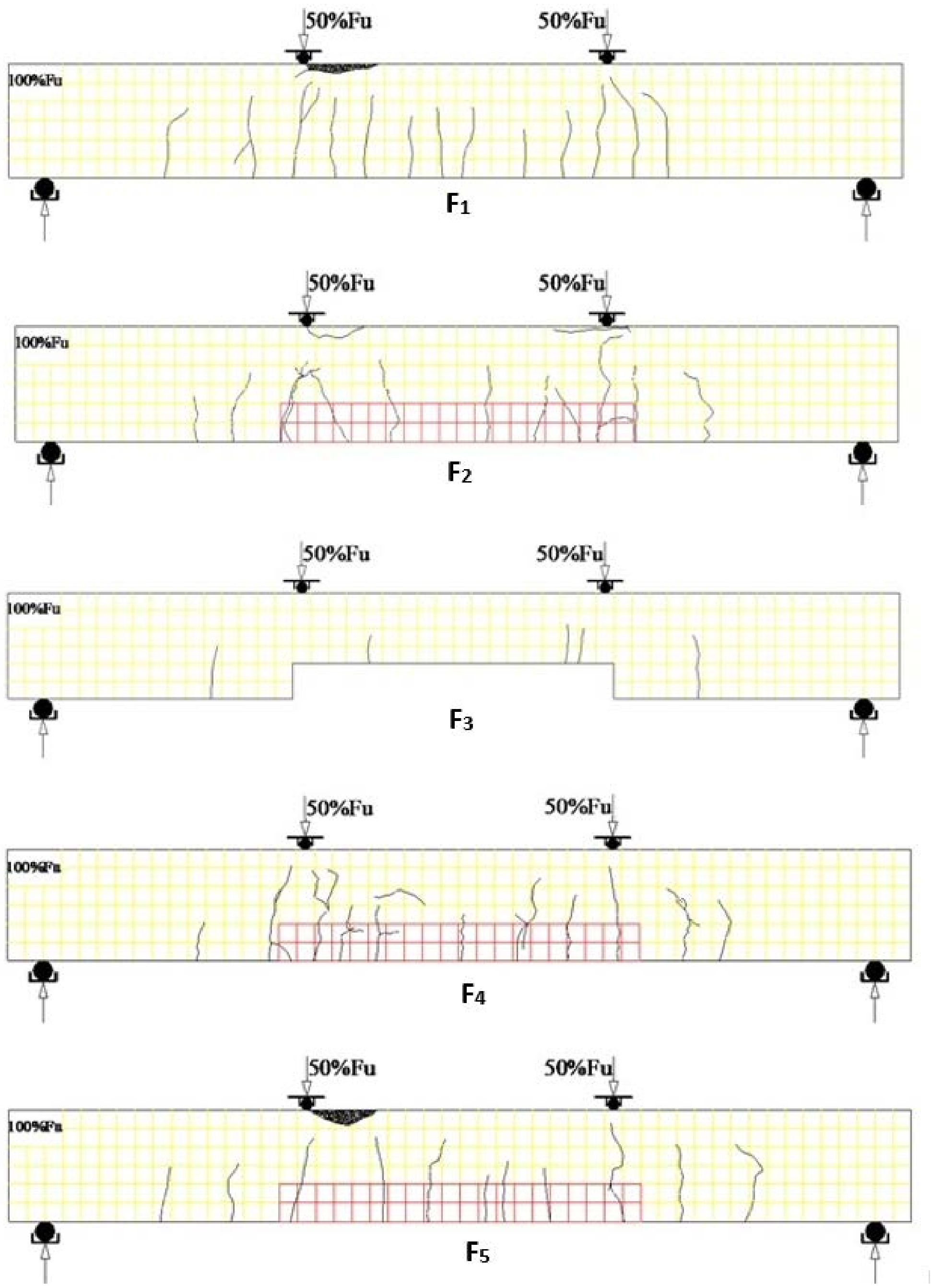

Figure 15. Schematic crack patterns for PT beams at ultimate loads From the top: F1, F2, F3, F4, F5

Based on Figure 12 to Figure 14 and on closer inspection of the beams, it can be seen that beam F4 (which doesn't use epoxy) has more cracks with smaller widths within the patch repair area in comparison with F5 (which used epoxy). This shows that the stress transfer between the reinforcement and concrete is more uniform and so, is carried out more efficiently compared to beams with fewer cracks and larger crack widths. Based on the flexural loading tests, there is little difference between the beams with or without using epoxy in terms of flexural capacity. However, the repaired beam F5 that used epoxy, showed large crack width under service loads and shear failure under ultimate loads. Note that the wider cracks under service loads can accelerate the corrosion process in the marine environments.

5. Conclusions

The aim of this study was to investigate the effect of concrete removal and deformed bars corrosion on the flexural response and load-deflection behavior of post-tensioned beams and the effectiveness of patch repair in restoring the rigidity and load-bearing capacity of the deteriorated beam. The main conclusions drawn from the results are given as follows:

- After a period of 75 days, 15.37% and 15.75% average degree of corrosion were obtained for longitudinal and transverse reinforcement of the beams, respectively. It was found that the corrected form of Faraday's law is more suitable for prediction of the corrosion period in reinforced concretes.
- The main cause of reduction in flexural rigidity and load-bearing capacity was due to the removal of concrete from the corroded PT beams. Whereas the corrosion of deformed rebars alone, only affected the flexural behavior of beams after the yielding of prestressed tendons.
- By applying patch repair in the deteriorated areas without replacing the corroded rebars (F4), the load carrying capacity and flexural behavior of the beam can be significantly restored.
- The use of epoxy on substrate concrete as an under layer for repair concrete, not only failed in improving interface conditions between the substrate and repair concretes, in fact, it considerably reduced the bond strength between these two segments.
- Beam F4 endured greater number of cracks with smaller crack widths compared to beam F5 under service loads, which is desirable in marine environments.
- Under ultimate loads, beam F4 showed flexural failure (which is desirable), while beam F5 showed shear failure due to debonding between repair and substrate.
- In corroded PT beams with non-corroded tendons, it is recommended to remove the contaminated region beyond the reinforcement before the spalling of concrete begins. After cleaning the corroded reinforcement, high performance repair material can be used as an appropriate repair approach.

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