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STRUCTURAL PERFORMANCE OF REPAIRED CORRODED REINFORCED CONCRETE BEAMS

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Abstract: The undertaken research work included in the paper presents the results of a laboratory investigation of the structural behavior of corroded and repaired beams in comparison with undamaged identical beams. The investigated structural behavior included shear and flexural strengths, ductility expressed by the middle span deflection, crack development and mechanical properties of the corroded reinforcing steel bars. The test program included fourteen beams; ten out of them were tested as simply supported beams subjected to two concentrated point loads. Two out of these ten beams acted as control beams, other two as corroded beams without repair and the remaining six beams were repaired after been corroded using different repair techniques. The other four were used to investigate the corrosion rate and the mechanical properties of the corroded steel bars. Special laboratory setup was assembled to allow fast corrosion of beam reinforcement. An electrochemical system was used to achieve the specified corrosion level. Repairing of beams was achieved by casting underlay inside which using additional longitudinal steel reinforcement bars to compensate the loss of steel reinforcement cross section due to corrosion. The deteriorated concrete bottom layer was replaced by a new underlay. To ensure monolithic behavior of the repaired beam the new reinforcement bars were fixed to the beam with shear connectors. Three repair techniques were investigated. Repair type 1 consisted of casting normal concrete overlay without bonding. In repair type 2 bonding agent was applied at the interface between the old and new concretes. Type 3 consisted of casting repair material as underlay bonded to the old concrete with bonding agent similar to repair type 2. The test results showed that the flexural capacity of the corroded beams have been reduced by 28% compared to the control beams. The ductility of the corroded beams was also adversely affected. On the other hand, the repair techniques succeeded in not only regaining the original strength, but have also resulted in a strength increase by up 47 % compared with the control beams. In general, the requirements of the serviceability limit state in terms of crack width and deflection were satisfied. Using different techniques in bonding new to old concrete did not affect the results significantly since all techniques have succeeded in preventing premature shear or laminar shear failure and thus allowed the beams to reach their full capacities which were enhanced using additional reinforcement and increased effective depth.

KEYWORDS: corrosion, repaired beams, underlay, bonding, structural performance.

التصرف الإنشائي للكمرات الخرسانية المعالجة من الصدأ

الملخص: تقدم هذه الورقة البحثية نتائج الفحوصات المختبرية للتصرف الإنشائي للكمرات الخرسانية المعالجة من الصدأ مقارنة مع مثيلاتها من الكمرات القياسية. حيث شملت هذه الاختبارات دراسة قوى القص وقوى عزم الانحناء وكذلك المرونة ممثلة بالهبوط أسفل منتصف الكمرات، بالإضافة لتحليل ظهور التشققات في الكمرات وقياس الخصائص الميكانيكية لقضبان الحديد الصدأة.

أربعة عشر كمرة خرسانية تم استخدامها في برنامج الفحوصات، عشرة منها تم اختبارها ككمرات خرسانية مرتكزة على دعائم بسيطة ومعرضة لقوتين مركزتين، كمرتين من العشرة ككمرات قياسية و كمرتين أخريين ككمرات تعاني من الصدأ بدون معالجة بينما ستة ككمرات معالجة إنشائياً من الصدأ باستخدام وسائل اصلاح مختلفة. الأربعة كمرات المتبقية تم استخدامها لفحص درجة الصدأ وكذلك لتحديد الخواص الميكانيكية للحديد المعرض للصدأ.

تم إتباع نظام كهروكيميائي للوصول لدرجة صدأ الحديد المطلوبة داخل المختبر لتسريع عملية صدأ الحديد داخل الكمرات. كما وتم معالجة الكمرات الصدأة بإضافة حديد طولي سفلي ليعوض نقص مقطع الحديد الأصلي نتيجة الصدأ وكذلك تم الاستعاضة عن طبقة الباطون السفلية المهترأة بطبقة جديدة. ولضمان تصرف إنشائي سليم للكمرات المعالجة تم تثبيت الحديد الطولي الإضافي مع الكمرات بقطع حديدية رأسية لتقاوم قوي القص.

أثناء عملية معالجة الكمرات الصدأة تم استخدام ثلاث تقنيات للإصلاح. التقنية الأولى اعتمدت على صب طبقة باطون طبيعي أسفل الكمرات الصدأة بدون مادة رابطة بينما اعتمدت الطريقة الثانية على اضافة مادة رابطة بين سطح الباطون القديم و طبقة الباطون الجديدة. التقنية الثالثة اعتمدت على استخدام مادة إصلاح بديلاً عن الباطون مع إضافة مادة رابطة كما في الطريقة الثانية.

أظهرت النتائج أن القوة الإنشائية لعزم الانحناء للكمرات المعرضة للصدأ كانت أقل بنسبة حوالي 28% من الكمرات القياسية كما و أظهرت ضعف في مرونتها أثناء الفحص. من جهة أخرى عملية الإصلاح نجحت ليس فقط في إعادة الكمرات لقوتها القياسية بل زادت القوة الإنشائية للكمرات المعالجة بنسبة 47% مقارنة بالكمرات القياسية و أظهرت مرونة جيدة و تصرفا إنشائياً مماثلاً للكمرات التي لا تعاني من أي مشاكل بالنسبة لعرض التشققات و الهبوط الناتج عن الفحص وفق المعايير المطلوبة في حالة حد التشغيل. كما و أظهرت النتائج أن استخدام تقنيات إصلاح مختلفة لضمان ربط طبقة الباطون القديمة بالجديدة لم تؤثر بشكل ملحوظ على النتائج حيث لم يحدث أي تشققات مبكرة أو أفقية أثناء عملية الفحص. حيث نجحت الكمرات المعالجة في الوصول لكاملقوتها المكتسبة نتيجة الحديد الطولي السفل المضاف ونتيجة العمق الإضافي الحاصل بعض إضافة الطبقة السفية الجديدة.

INTRODUCTION

Corrosion of reinforcement steel bars is the most serious problem that results in deterioration of reinforced concrete structures. More than 80% of reinforced concrete structural damages around the world are caused by the corrosion of steel [1]. The problem of corrosion is also found in most Middle East countries [2]. The high temperature combined with concrete mixing water with high salt concentration of ground water, have added to this problem, especially in the Gulf Countries. In the period from 1960 to 1970 it was acceptable in Europe and Middle East countries (due to scarcity of fresh water) to make concrete with sea water. Also, calcium chloride was used extensively as a set-accelerating admixture. During the following 20 years it was discovered that this practice has resulted in deterioration of the structures due to corrosion of reinforcement and thus resulted in huge economical and investment problems.

In Gaza Strip, many structures show signs of deterioration due to corrosion of reinforcement bars. This could be attributed to inappropriate construction practices, (e.g. inadequate concrete cover, high w/c ratio, insufficient compaction, segregation, etc.) and environmental factors since Gaza is a coastal area. This location with the associated environmental conditions has a considerable influence on the deterioration of existing concrete structures, especially steel corrosion. A survey of forty case studies for assessment of existing damaged structures in Gaza Strip showed that the main cause of defects in existing buildings was reinforcement corrosion with about 31% of the causes [3].

The corrosion problem reduces significantly the lifespan of structures and requires more financial resources for reconstruction. Corrosion can be seen as an economical problem, which requires proper assessment and involvement in the right time to minimize further deterioration. The right time is controlled by safety and economical constraints. As with time, corrosion, repair cost and failure risk are increased.

The research work in this paper is concerned with the problem of corrosion of reinforced concrete beams. The aim was to evaluate the efficiency of various repairing techniques for solving this problem from a structural point view. To overcome the problem of deterioration due to corrosion, the causes and mechanism of corrosion needs to be fully understood. Therefore, it is useful to review main issues related to corrosion damages and repairing techniques of reinforced concrete structures.

CORROSION OF STEEL REINFORCEMENT BARS

Mechanism of Corrosion

Corrosion is an electrochemical process where a metal undergoes a reaction with chemical species in the environment to form a compound. The chemical species are principally oxygen and water. The corrosion of steel is the process in which steel is oxidized at the anode and the electrons are released and flow to the cathode for the oxygen reduction reaction [4]. Fig.1 shows a diagram of rust formation on steel reinforcement in concrete.

The two main reasons for corrosion of steel in concrete are the chloride attack and carbon dioxide penetration. The concrete is a porous material containing water in the voids due to the process of curing or because of rainy weather or any weather with high relative humidity. It is not necessary that steel bars embedded in concrete to be corroded because concrete has a high concentration of the oxides calcium, sodium, and magnesium. These oxides produce hydroxides that have a high alkalinity when water is added resulting in pH between 12 and 13. This alkalinity will produce a passive layer on the steel reinforcement surface; consisting of oxides and hydroxides for iron and part of cement. This layer is dense and prevents the occurrence of corrosion [2]. This passive layer can however, be broken when carbon dioxide enters the concrete and reaches the steel-concrete interface. This is called carbonation. Another a powerful destroy of the steel passive layer is the present of chloride salt in concrete. Chloride ions are introduced into the concrete by marine spray, industrial brine, deicing agents, and chemical treatments. The chloride ions can reach the reinforcing steel by diffusing through the concrete or by penetrating cracks in the concrete [5]. Some acids, such as sulfate, will attack the concrete and cause concrete deterioration and corrosion of steel. On the other hand, significant corrosion does not occur for steel in concrete that is either very dry or continuously saturated, because both air and water are necessary for corrosion to be initiated. Steel will remain corrosion-resistant in concrete if the concrete cover prevents air and water from reaching the embedded reinforcement [6].

Corrosion Rate

Corrosion rate is considered one of the most important factors in the corrosion process from a structural-safety perspective and in the preparation of the maintenance program for the structure. This factor is considered an economic factor of structural life, when the corrosion rate is very high, the probability of structure failure will increase rapidly and structural safety will be reduced rapidly [2]. Prediction of corrosion rate is thus necessary to take right decisions concerning the right time for intervention to prevent excessive damages to concrete structures due to corrosion. Many physical and mathematical models have been proposed to estimate the time of corrosion initiation and propagation. Tutti's two-stage model shown in Fig. 2 was proposed to predict the service life of reinforced concrete structures [7]. Stage 1 is reached when the chloride or carbonation has reached the reinforcement. Stage 2 is reached when corrosion due to oxidation of the

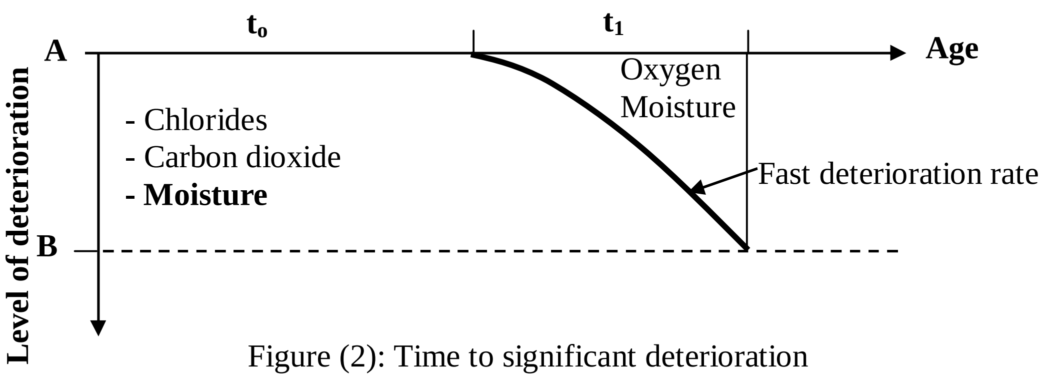


Figure (2): Time to significant deterioration

steel has been developed sufficiently for spalling and cracking of the cover to occur. The concept of this model is to divide the service time of the structure into t_o, as the time to corrosion initiation, and t_1, as the time taken for deterioration of a sufficient level to occur to be of concern.

A useful and simple, 3-component model shown in Fig. 3 was developed for defining whether corrosion damage would occur:

a. Chlorides or carbonation have to be present to destroy the alkali protection to the reinforcement provided by concrete.

b. A low concrete electrical resistively is required to permit electrochemical corrosion. Values of less than 5,000 to 10,000 ohm.cm may be critical.

c. Oxygen from the atmosphere has to get through the concrete cover.

Very dry conditions, as exist in many parts of the Middle East, can result in high concrete resistivity greater than 100,000 ohm.cm, inhibiting the corrosion process until it rains where high chlorides exist in the concrete (dry concrete is very porous to oxygen). High relative humidity, daily cyclic condensation, evaporation and high temperatures are the preliminary environmental conditions which result in low concrete resistivity. Unless the concrete is saturated, the oxygen can get through the cover to the steel in a matter of hours. The worst combination for corrosion to proceed is when the concrete is slightly drier than saturated, i.e. around 85-95% RH with a low resistivity and when the oxygen can still penetrate to the steel.

Corrosion Damage

The corrosion damage can be classified into three main groups [8]:

1. Those, which affect the reinforcement section, reducing the effective area and ductility.
2. Those, which are related to concrete integrity.
3. Those, which affect the interaction concrete – reinforcement due to the bond reduction.

The most problems that occur because of corrosion of steel are due not only to the reduction in the steel section but also to fall of the concrete cover that in turn results in bonding loss. Many studies have been conducted to calculate the amount of corrosion occurring and causing the concrete cover to fall. It has been found that cracks may occur in cases of reduction of 0.1 mm from steel reinforcement sections and, in some cases, much less than 0.1 mm, depending on the distribution of oxides and the ability of concrete to withstand the stresses, as well as the distribution of steel [2]. The reason of concrete cover failing refer to the fact that the rust occupies a much larger volume than the original steel and causes the buildup of bursting forces at the surface of the reinforcement. Because concrete is weak in tension these bursting forces quickly cause the concrete to crack parallel to the reinforcement direction and eventually, to spall away from the rebar. However, corrosion is a complex mixture of oxides, and hydroxides and hydrated oxides of steel have a volume ranging from twice to about six times that of the steel consumed to produce it. The magnitude of the rust incremental are various according to various steel oxides generated. For example, the volume of FeO, $Fe(OH)_2$ and $Fe(OH)_3 3H_2O$ will increase by 2, 3.5 and 6.5 times the volume of Fe they originally occupied [5].

It should be noted that the concrete cover in the corner is more prone to falling because it is a largely exposed area for the penetration of carbon dioxide or exposure to chlorides, as well as oxygen. Therefore, concrete cracks often happen faster in this situation. Stirrups may be the first to crack or spall due to their lower cover. The significance of corrosion damage to the concrete depends on the structure, function and location of damage. Cracks and delamination to the raft slabs and foundations will result in a reduction in the strength of the structural members. Delaminations which cover large areas may well reduce the flexural capacity of a slab, particularly as corrosion of the exposed steel may then accelerate.

REPAIR OF CORRODED REINFORCED CONCRETE STRUCTURES

Many alternatives are available to the maintenance engineer to combat corroded deterioration buildings and keep the structure in its original condition [9]. These may be grouped into three broad categories such as protection, patching and rehabilitation [10,11]. However, before approaching corroded reinforced concrete structures repairs, consideration must first be given to the cause of the problem. There are several regular steps in the repair of all structures exposed to corrosion. The very critical first step in strengthening of a structure is the use of a suitable temporary supports. The second step is to remove the cracked and delaminated concrete. It is important to clean the concrete surface and also the steel bars by removing rust. To remove the cracked and delaminated concrete, many methods can be followed such as manual method, pneumatic hammer method, water jet and grinding machine. After this step, a new patch concrete or repair materials are applied. There are many execution methods for patching concrete such as manual method, grouted preplaced aggregate and shotcrete. The final step is to apply a protective coat to the concrete surface against environmental factors [2].

TEST PROGRAM

Objectives

This research study focuses on the problem of structural performance of the repaired reinforced concrete beams. The laboratory work presented included carrying out an accelerated corrosion tests on reinforced concrete beams to understand and make reliable assessment of the effect of corrosion on the beams. Repairing techniques are then applied to the corroded beams which were tested to evaluate their structural performance.

Test Beams

The test program consisted of fourteen reinforced concrete beams that were designed, constructed, corroded, repaired and tested in flexure to achieve the research objectives. Two of the beams were tested as control beams, two as corroded beams and six as repaired beams. The remaining four beams were used to investigate the corrosion rate for the corroded beams. The beam dimensions were 1200 mm × 150 mm × 100 mm as shown in Fig. 4. The beams were designed in accordance with ACI 318-08 [12] with nominal bending moment M_n = 9.5 kN.m. The shear failure was prevented by using stirrups Φ6@50mm. The specified steel yielding strength was 412 MPa. 10mm nominal diameter deformed steel bars were used for the longitudinal reinforcement. The test results of steel reinforcement gave f_y = 545MPa, f_u = 738 MPa and 30% elongation. The stirrups and secondary reinforcement were fabricated from 6mm nominal diameter plain round steel bars. All of the beams have longitudinal reinforcement ratio r = 1%, with an overall length of 1200mm and width to depth dimensions of 100mm × 150mm. To prevent anchorage failure, all of the beams were extended beyond the

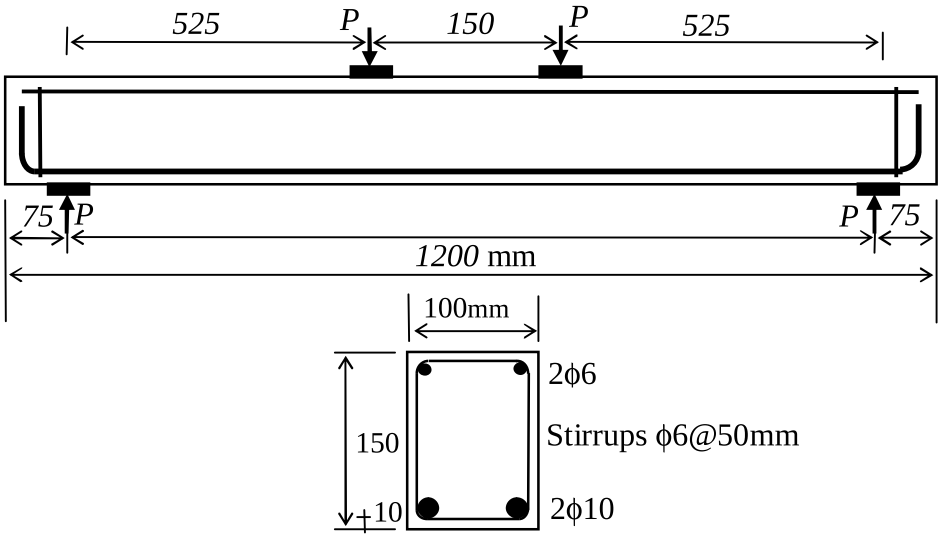


Figure (4): Beam details

supports for a distance of at least 75mm and were bent up a distance of 100mm at their ends.

The design cylinder concrete compressive strength " f_c' " was 25 MPa. The concrete mix design is shown in Table 1.

Table 1: Mix design*.

Material	Weight/m^3 (Kg)
Coarse Aggregate	930
Fine Aggregate	892
Cement	336
Water	241

- w/c = 0.61 and maximum aggregate size = 19 mm.

Corrosion Process Set-Up

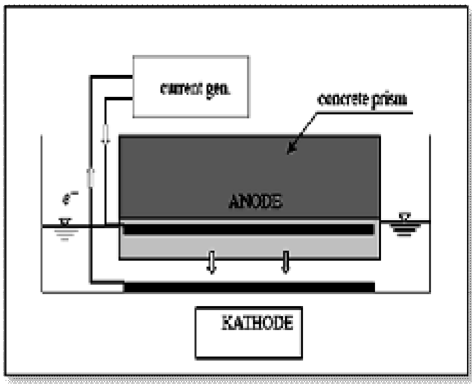


Figure (5): Corrosion technique [13]

In order to accelerate the corrosion process, an electrochemical system based on the concept of Faraday's second law was used. The concept of accelerating the corrosion was to force steel reinforcement to act as anode in galvanic cell. This was done by immersing the test beams in aqueous solution and connecting the steel reinforcement bars with positive DC current generator to act as anode. Fig. 5 shows a schematic diagram of the accelerating corrosion process [13].


Figure (6): Test beam

Fig. 6 shows a photo of the test beams inside the basin during the corrosion process. The negative power supply was connected to external steel rods immersed in the aqueous solution to act as cathode. The resulted electric circuit forced steel ions to translate from anode to cathode. A basin was filled in the laboratory with a sodium chloride solution (5 percent NaCl by weight of water) to act as the aqueous solution. The solution covered 30mm of the beam height in which the flexural reinforcements was placed. This was to ensure that enough oxygen needed for the corrosion mechanism exists in the vacancy of the reinforcement. The electric wires were attached to the reinforcement bars. The beams were connected in parallel to +5 Volts with a capacity of 25 Amperes electric DC current generated by power supply, which impressed an equal voltage on each beam. The negative power supply terminal was connected to twelve 10 mm steel rods immersed in the aqueous solution between the beams in order to facilitate the opening and closing of the electric circuit. The required corrosion level was achieved (crakes appear along longitudinal reinforcements) in about three months.

Repair Technique

The repair technique comprised of adding a new underlay to replace the delaminated bottom concrete layer which covered the corroded steel bars. Two new supplementary steel bars of Φ8mm were added to compensate the corroded flexural steel bars at the bottom of the beam. Φ8mm anchoring steel bars of U shape were anchored to the old concrete to replace the corroded stirrups and to prevent laminar shear failure at the interface between the old concrete layer and the new underlay.

Two types of bonding agents were used to bond anchor bars inside concrete holes (bonding agent Type 1) and to bond wet cementitious material to existing cementitious surface (bonding agent Type 2). Bonding agent Type 1 is a polyester anchoring resin used for installing a high strength corrosion resistance heavy duty anchoring. This bonding agent was used for the purpose of fixing the steel anchoring bars to the old concrete. Bonding agent Type 2 is an epoxy resin concrete bonding agent used for bonding wet cementitious material to existing cementitious surface. This repair material was used for the purpose of bonding the new underlay to the old concrete.

Three types of repair materials and techniques shown in Table 2 were investigated for the new underlay. These are:

Repair Type 1: Normal concrete directly cast to old concrete without bonding agent. Repair Type 2: Normal concrete bonded to old concrete using bonding agent Type 2. Repair Type 3: Repair material shrinkage-compensated cementitious precision grout bonded to old concrete using bonding agent Type 2.

Table (2): Underlay types and applying steps

Repair Type	Underlay Description	Beam Type	Underlay Applying Steps
1	Use of normal weight concrete according to mix design proportions shown in Table 1, without any additions.	(R1-B1) & (R1-B2)	• Concrete surface was cleaned from dust and unsound concrete. • Surface of concrete was flooded with water and then free water was removed. • Beams were put in water after 24 hours from casting.
2	Using of normal weight concrete according to mix design proportions shown Table 1, adding bonding agent type 1 at the interface between the new underlay and the old concrete.	(R2-B1) & (R2-B2)	• Concrete surface was cleaned from dust and unsound concrete. • Surface of concrete was cleaned with water before applying the repair material. • Bonding agent Type 2 was well mixed and brushed to the concrete surface. • The new concrete underlay was caste after two hours from brushing. • Beam was put in water after 24 hours from casting.
3	Shrinkage-compensated cementitious precision grout.	(R3-B1) & (R3-B2)	• Concrete surface was cleaned from dust or unsound concrete. • Surface of concrete was flooded for 5 hours before casting the new underlay and then free water was removed. • Repairing material was prepared and cast to the existing concrete. • Beam was put in water after 24 hours from casting.

General Repair Steps

Repairing process was carried out in accordance with the following general steps:

1. The concrete covering and loosely concrete was removed.

2. The surface of concrete was cleaned well until reaching a sound concrete surface.

3. The corroded steel reinforcing bars were cleaned from rust using steel brush.
4. Twenty holes of 12mm diameter and 100 mm depth were drilled into the bottom surface of the beam at the location of stirrups.
5. The drilled holes were cleaned with air compressor and then injected with the bonding agent Type1.
6. The 8 mm anchored steel bars of length of 170 mm were placed inside the holes before drying of the bonding agent (100mm into the hole and 70 mm into the new underlay).
7. The beams were left for 24 hours, and then the two 8 mm steel reinforcing bars were placed in the flexural zone inside the U shape anchored bars.
8. After installing the shear connector anchored bars and the steel bar reinforcement, 40 mm new concrete underlay was added to the beam.

Casting and Testing

After the end of repairing process, beams were kept in water for 28 days and then had been tested. Before testing the beams were whitewashed for easier identification of cracks during the loading sequences. The beams were tested using the four point loading system shown in Fig. 4. Two 30 mm in diameter steel rods were used as roller supports while two steel bars with a semi-half circular section with radius of 40 mm were used to apply the middle concerted loads. A mechanical dia1 gauge was located at the mid-span to measure the deflection. The load was increased with 0.785 KN increment. However, deflection readings were recorded for each 4 kN load increment. During the test process the crack development was recorded.

TEST RESULTS

Flexural Capacity

Table 3 shows the test results which include ultimate measured load, middle-span deflection at failure and the ratio of measured load to the theoretical control beam load. It can be noted that the average flexural capacity of the two corroded beams has been reduced by 28% compared to that of the control beams. This was due to the loss in the cross section area of steel reinforcement. Fig. 7 shows typical corroded reinforcement steel bar where it was difficult to measure the reduction in the cross section due to its irregularity. On the other hand, the flexural capacity of the repaired beams has increased by 47% compared to that of the control beams. The increased flexural capacity is due to the increase in the steel cross area of the two additional supplementary reinforcement bars. The increase of effective beam depth has also contributed to the added strength. It can be concluded that the three repair techniques succeeded in strengthening the corroded beams and had resulted in an increase in their flexural capacitates. However, Repair Type 1 in which casting of the concrete underlay was carried out without applying bonding agent was less effective compared to the other two Repair Types 1 and 2. However, the difference was insignificant since all beams were able to reach their flexural behavior.

Table (3): Measured ultimate load and deflection

No	Beam Type	Beam Description	Load (kN)	Average Load (kN)	Deflection (mm)	Ult. Load/ Theo. Control Load (%)
1	CB1	Control Beam 1	43.20	44.0	2.50	102.2
2	CB2	Control Beam 2	44.80		2.54	106.0
3	Co.B1	Corroded Beam 1	31.20	31.6	2.13	73.8
4	Co.B2	Corroded Beam 2	32.00		2.07	75.7
5	R1-B1	Repaired Type 1 Beam 1	62.27		2.53	147.3
6	R1-B2	Repaired Type 1 Beam 2	62.76		2.48	148.5
7	R2-B1	Repaired Type 2 Beam 1	67.00		2.55	158.5
8	R2-B2	Repaired Type 2 Beam 2	66.69		2.53	157.8
9	R3-B1	Repaired Type 3 Beam 1	65.20		2.39	154.3
10	R3-B2	Repaired Type 3 Beam 2	64.70		2.62	153.1

Figure (7): Corroded and Non-corroded steel reinforcement bars

Shear Strength

The stirrups in the control and corroded beams and the anchored steel bars of U shape succeeded in preventing shear failure in all beams. The anchored steel bars succeeded also in preventing the laminar shear failure at the interface between the old concrete and the new overlay. The use of bonding agent at the interface did not result in significant enhancement since the mechanical bonding provided by the stirrups was adequate alone. It is thus concluded that the use of bonding agent at the interface is not a prerequisite in the case of mechanical bonding provided by anchored bars.

Requirements of the Serviceability Limit State

The requirements of the serviceability limit state can be evaluated based on the ductility and cracking obtained from the test beams. The elongation tests carried out on the corroded steel bars have shown a significant reduction in the measured deformation up to 83% compared to non corroded steel bars. Despite of this, the repaired beams have achieved comparable deflection level as the controlled beams as shown in Fig. 8. The enhanced ductility of the repaired beams is attributed to the additional supplementary steel bars.

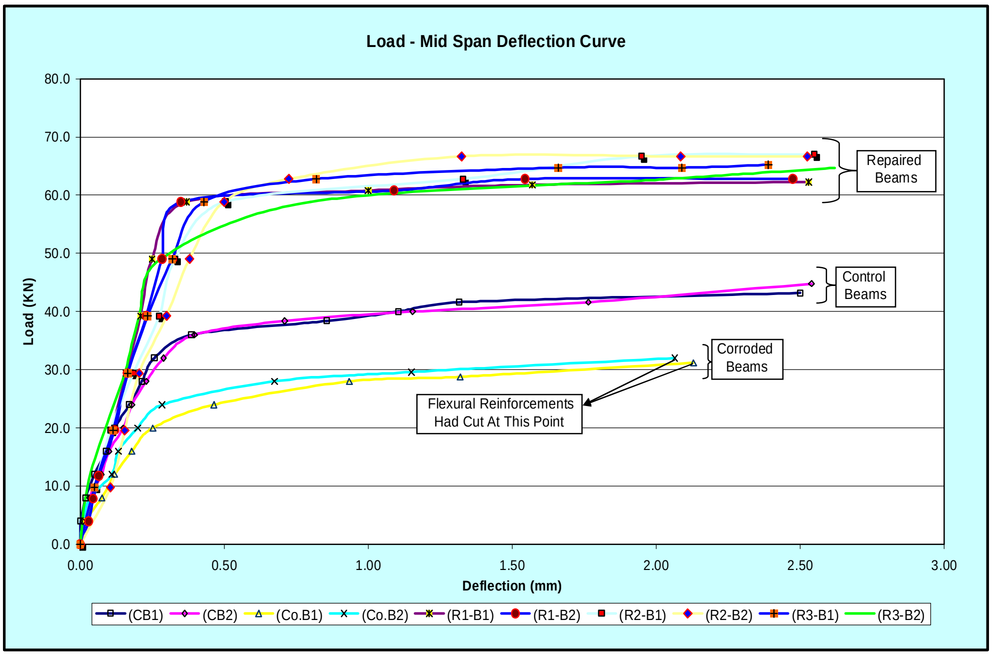
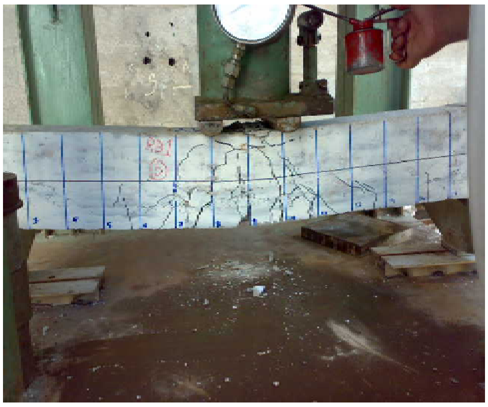


Figure (8): Ultimate load middle-span deflection curves

Fig. 9 includes photos of typical cracked patterns obtained from controlled, corroded and repaired beams. Regardless of the repair type used, the repaired beams have shown cracking widths and patterns similar to that obtained from the controlled beams. The corroded beams exhibited excessive crack widths, especially at the flexural reinforcement level.

Control beam C B1

Corroded beam Co B1



Repaired beam R1 B2

Repaired beam R2 B1

Repaired beam R3 B2

Figure (9): Crack patterns

CONCLUSIONS

1. Corrosion of concrete structures is common in the coastal area of Gaza Strip, Palestine. In the undertaken research work, it was possible to investigate the influence of the corrosion process on the flexural capacity of corroded beams and the structural performance for the repaired corroded beams. An experimental test program consisted of 14 beams (4 out of which were used to investigate the level of corrosion) was carried out. Two beams were used as controlled beams, two beams were tested after corrosion and the remaining 6 beams were tested after repairing of the corrosion.

2. In order to accelerate the corrosion process, an electrochemical system which depends on the concept of Faraday's second law was used. The induced accelerating corrosion process succeeded in producing corrosion in the reinforcement steel bars in about three months only; which has made this investigation possible!

3. It was possible to strengthen the corroded beams by casting new underlay to replace the deteriorated bottom concrete in the vacancy of the corroded flexural reinforcement. Additional supplementary flexural steel bars and anchored bars of U- shape were used to compensate the loss in the cross area of the corroded steel reinforcement. The anchored bars have acted as stirrups and prevented laminar shear failure at the interface between the old concrete and the new underlay.

4. The flexural capacity of the corroded beams has decreased by 28% and the corroded steel bars have lost 83% of their elongation capacity.

5. Three types of repair techniques were investigated. Repair Type 1 consisted of casting concrete underlay without any bonding to old concrete. The anchored bars alone succeeded in preventing the laminar shear failure. Repair Type 2 was similar to Type 1 except that a bonding agent was used at the interfaces. Repair Type 3 was similar to Type 2; however a repair material was used as underlay instead of normal concrete. Regardless of the repair type employed, all repaired beams behaved in a ductile flexural manner up to the failure.

6. In comparison with the controlled beams, the repaired beams achieved similar ductility, crack pattern and 47% increase in their flexural capacity. The increase in the flexural capacity was due to the addition of flexural reinforcement and the increase in the effective depth. The variation in the flexural capacities obtained from the various repair types was insignificant since the mechanical bonding provided by the anchored bars was adequate to prevent the laminar shear failure and forced the beam to act as one unit.

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