Effect of Different Strengthening Techniques on the Structural Behaviour of Continuous Two-Way Slabs with Partially Corroded Reinforcement at Internal Support

Ammar K. AL-Najar 1* and Labeeb S. AL-Yassri 2*

1 Post Graduate Student, The University of Al-Qadisiyah, College of Engineering, Iraq
2 Lecturer, Ph.D. The University of Al-Qadisiyah, College of Engineering, Iraq

*Corresponding author: ms.ammar.kareem@qu.edu.iq

Abstract. This research studies the combined effects of partial corrosion of the negative reinforcement in two-way continuous slabs (two spans) with a construction joint at the internal support and investigates the efficiency of the proposed strengthening methods to overcome these effects. The partial corrosion in reinforcement comes from exposure to the corrosive outdoor environment due to an unplanned stoppage in concrete pouring for a long-time which left part of the negative reinforcement without protection. The construction joint formed at the internal support after resume the concrete pouring later. Three types of strengthening are proposed to overcome those effects. The study adopted an accelerated corrosion technique which is running an electrical current in the steel bars partially immersed in a 5% salt solution (elect-chemical cell). The proposed strengthening was: NSM CFRP bars, CFRP strips, and steel plates. All applied to the top face in the tension zone at the internal support. Six two-way continuous (two spans) concrete slabs were cast (2200, 1000, and 100 mm). The strengthening was: four strips of CFRP had 50 mm width, four of 6 mm CFRP bars applied using the NSM technique, and four steel plate 50 mm width and 5 mm thickness. All the strengthening extended to the third of span length and was aligned with the slab continuity axis. The specimens were tested and the deflection, first positive and negative crack width, and failure mode were recorded. Numerical analysis and comparison with the experimental results were conducted to verify the numerical model. The study found that the construction joint and corrosion had a clear influence on the structural behavior of the slab. Those effects were on the ultimate load, deflection, and the negative cracks. The proposed treatments were highly effective and efficient in increasing the ultimate load, delay the appearance of cracks, and reduction of deflection under the load points.
1. Introduction

Exposure steel structures to the natural atmosphere will suffer from corrosion and this can lead to extreme damage. Atmospheric corrosion is the corrosion that happens on the metal surface due to exposure to the outdoor environment, corrosion generally is an electrochemical process. The chemical part of the corrosion process includes a change in the chemical side of the reaction while the electric part contains electron and ionic interchange. The metal surface represents the electronic conducted and the water represents the ionic conducted (electrolyte). The main components of the cell are the anode, cathode sites, electrolyte, and an oxidizing agent [1]. Atmospheric factors that affect atmospheric corrosion are relative humidity, pH, dew, fog, dust and soot, wind velocity, corrosion products, pollutants, and distance from the sources [2]. Bates and Rakanta [3] reported study results conducted on four types of reinforcement steel bars suffered from corrosion due to the exposure to the atmospheric environment to investigate their performance. The study focused on the bond strength between the concrete and the corroded steel bars. The study underlined that corroded reinforcement bars exhibit a remarkable reduction in the pull-out strength. This reduction was highly affected by the thickness and the morphology of corrosion products, the reduction rate of the pull-out strength increases as a response to the increase of corrosion. Strengthening of the established concrete structure to endure higher design loads, or fixing damages and raise the ductility could be done conventionally by using common material and available techniques such as externally bonded steel plates, steel or concrete jackets, and external post-tensioning [3]. FRP became an accepted material for engineers from the mechanical properties point of view, CFRP is an anisotropic material that keeps high strength in the direction of its fibers [4]. Joint is essential when pouring concrete in two phases separated by time longer than the setting time of cement. There will be construction joints and when concrete incur volume changes due to shrinkage and temperature changes [5]. Construction joints: it placed where there is a stoppage in the process of concrete placing [6]. Abbas et al [7] Investigated the effect of different types and configurations of construction joints on the behavior of one-way reinforced concrete slab: slabs with a vertical joint at the shear zone failed by the shear while the others failed by flexural, slab with a vertical joint at mid-span had a reduction on the cracking capacity. Abdulah [8] studied the behavior of reinforced two-way lightweight concrete slabs strengthened by CFRP strips to resist punching load and compare the results with the finite element analytical model results. The CFRP strips dimensions were 80 mm length, 50 mm width, and 0.131 mm thickness and were diverse in shapes. The study found an increase in the ultimate loads compared with the not strengthened slabs, stiffness of slabs increased, as a result, there was a decrease in maximum deflection, delaying the crack appearance, and reducing the crack width. Research conducted by Sun et al. [9] studied the bond failure of the CFRP strip which is used to strengthen RC two-way slabs at tensile face. One of the two slabs strengthened with a CFRP strip on the tensile face and the other one wasn't strengthened (control). CFRP strips were 50mm in width and 1500 in length glued at the tensile face in x and y directions. The study conclusions were: CFRP strips were an efficient technic for strengthening Two-way RC slabs; de-bonding of CFRP strips take shape of a concrete wedge. Embed et al [10] Investigated the efficiency of strengthening five specimens of one-way slabs (1500, 600, 60 mm) by steel plates 353.7 MPa tensile strength. The parameters included in this study were the thickness of the plate (1, 1.5 mm) and plate length (600, 1200 mm). All specimens were cast with 30 MPa normal concrete. This research came up with that the steel plate enhanced the overall behavior of the slabs, increased the first crack load and the ultimate load, the most dominant parameter was the plate dimensions rather than the plate thickness. Axel et al. [11] Examined the effectiveness of replacing the steel reinforcement with exterior steel plates in reinforced concrete beams. The steel plate’s dimensions were, 1400x50x1.5, 1400x25x1, and 1400x75x1 mm. After the plates were applied to the tension face of the beams with a ratio of 33%, 67%, and 100%. The steel plate with a thickness of 0.5 mm gave better results. The ratio of replacement 33% gave the same load capacity of reference specimens.

This study aims to investigate the behavior of R.C. two continuous slabs with a partial corroded reinforcement bar in the negative zone due to the stoppage of construction for a long time and the presence of construction joint. The study investigates the efficiency of three types of strengthening to compensate for the losses in force due to the partial corrosion and the construction joint.
2. Experimental Program

2.1. Corrosion of Steel Reinforcement Bars
To make the study simulates the real-life scenario in which the uncovered reinforcement steel bars are suffered from partial corrosion due to exposure to a corrosive environment also corrosion in construction sites takes a long time to happen, the study adopted accelerated corrosion technique. The accelerating corrosion technique can describe as an electrochemical technique that depends on immersing steel bars in 5% salt solution and electric DC flow through the bars in setups which knows as an electric cell to gain clear signs of corrosion within a reasonable time (see Figure 1). The bar's corrosion process in the electric cell needed thirty days to gain clear signs of corrosion. After the corrosion process, complete bars cleaned by sandblasting technique and weighted to determine the weight loss percentage of bars. Table 1 illustrates steel losses for the specimens. The difference in the corrosion for steel bar should be a reflection of the real-life situation where the exposure condition for steel bars not expected to be identical in addition to the effect of pitting corrosion which is different from bar to another according to the manufacturing, exposing, and handling conditions.

![Figure 1: corrosion accelerating technique](image)

| Specimen  | Losses in Steel Area | Proposed Treatment          |
|-----------|----------------------|-----------------------------|
| S-T-1     | 0.00%                | Not strengthened, No construction joint |
| S-T-2&3   | 48.89%               | Reference                   |
| S-T-4     | 55.86%               | Steel Plate                 |
| S-T-5     | 51.60%               | CFRP Bras                   |
| S-T-6     | 65.37%               | CFRP strips                 |

2.2. Specimens Details
The study was conducted on six specimens of two-way continuous (two-span) slabs with dimensions of 2200 x 1000 x 100 mm. The specimens were cast using normal concrete with an average compressive strength at age 28 days equal to 32 MPa. To form the construction joint for the specimen tested, the framework was separated at 1100 mm (at the internal support) with a wooden barrier extend along the framework width. The woody barrier makes the framework consist of two parts. The concrete had been poured in the framework parts in two stages, the second part (post-casting span) poured after the first
Reinforcement details were configured in such a way that guarantees flexural failure. The positive reinforcement for both spans was 9 bars of 6 mm diameter for both directions. The negative reinforcement was 3 bars of 8 mm diameter and 2 bars of 10 mm diameter in the longitudinal direction and 9 bars of 6 mm in diameter for the transverse direction. Five specimens out of six suffered from partial corrosion (corrosion in one span) in the negative reinforcement and had a construction joint at the internal support while the last one of those specimens was cast fully without neither construction joint nor corrosion, Figure 3 illustrates reinforcement details of specimens.

2.3. Specimens Preparation for Strengthening
Since the efficiency of the installation of CFRP strips on the concrete surface was depends on the effectiveness of adhesive force between the concrete and the CFRP strips, it was necessary to remove the weak layer of concrete on the top surface and make the aggregate visible to some degree by grinder machine. Some scratches were made on the concrete surface to increase the bond between the CFRP strips and concrete (see Figure 4). The same procedure which was followed in the preparation of the specimen's surface for CFRP strips installation was adopted for steel plate installation except for the holes that were drilled in the slabs and the plates for bolts fixation. The 6 mm CFRP bars (Near Surface Mounted technique) installation needed a different type of preparation which was a slot with 15 mm width and 16 mm depth carved in the tension top face located above the internal support and extend to 350 mm in both spans. The slot was done by using the cutting machine to make two parallel lines in concrete separated by 15 mm and 16 mm. After removing the concrete between the two lines and after cleaning the slots were ready for use.
Figure 3: Reinforcement Details

Figure 4: Specimen Preparation
2.4. Application of Strengthening

Four strips of CFRP with 700 mm long and 50 mm width were used (see Table 2). The strips were separated from each other by 160 mm. The CFRP strips were applied on the top face at the tension zone (negative moment) above the internal support. They were glued to the surface by Sikadur®-330 using the required equipment in such a way that guaranteed there were no gaps between the CFRP strips and concrete. Four of the 6 mm diameter of CFRP bars (see Table 3) with 700 mm long were used, these bars were laying in the slots separated by 200 mm from each other which prepared for this purpose. After making sure that the slots were clean from dust and concrete remains a layer of the Sika AnchorFix®-2 was applied in the slots and then the bars were laid inside the slot and apply a light pressure on it to make the bar surrounded by the epoxy (bonding material), then filled the slot with the same epoxy. Four steel plates (700 x 50 x 5 mm) were fixed into the tension zone at the top face above the internal support with Quickmast SB®, each of the steel plates separated from each other by 160 mm. Properties of steel plates were tabulated in Table 4.

Table 2. CFRP Strips properties

| Property                        | Value   |
|--------------------------------|---------|
| Density (g/cm³)                | 1.82    |
| Laminate Nominal Thickness (mm)| 0.167   |
| Laminate Tensile Strength (N/mm²) | 3500     |
| Laminate Modulus of Elasticity in Tension (kN/mm²) | 225 |
| Laminate Modulus of Elasticity in Tension (kN/mm²) | 220 |

Table 3. CFRP bars properties

| Property                      | Value |
|-------------------------------|-------|
| Nominal diameter (mm)        | 6     |
| Nominal Area (mm²)           | 28.26 |
| Ultimate tensile strength (MPa) | 2241 |
| Modulus of Elasticity (GPa)  | 124   |

Table 4. Steel Plate Properties

| Property                     | Value   |
|------------------------------|---------|
| Nominal thickness (mm)       | 5       |
| Nominal Area (mm²)           | 250     |
| Ultimate tensile strength (MPa) | 250 |
| Modulus of Elasticity (MPa)  | 2.1E5   |
2.5. Bonding Materials

- Hard- Fresh concrete epoxy used in the construction joint which was commercially known as Sikadur®-32 LP which can be made from mixing two-component A and B in the ratio of 1:2 volume or weight.
- Sikadur®-330 is a bond agent, it used with CFRP strips. Two components of the material have to be mixed in the ratio of 1:4 to use.
- Sika AnchorFix®-2 which is used as a bonding agent between the concrete and the CFRP bars (Near Surface Mountain).
- Quick mast SB which is commercially known as Prebuild SB that used to fix the steel plate to the concrete surface.

3. Testing of Specimens

After specimens were painted with white emulsion to highlight the first appearance and development of cracks, they were tested with 1000 KN hydraulic test machines, they were laid on the test steel frame by crane carefully in such a way that prevents any unnecessary movement (see Figure 5). Load points were located in the mid-distance between the supports and the LVTD were fixed under the specimens to indicate the deflection. The vertical load was divided into four equal point loads and was applied in an increment of 5 KN each time, the crack width and deflection were recorded.

![Figure 5: Testing Frame and Hydraulic Test Machine](image)

4. Results and Discussion

Ultimate load, deflection in the two spans, the width of cracks, and failure modes all these parameters were measured and followed in the test. Cracks could be classified according to appearing zones which is a negative crack that appears in the negative moment zone on the top face above the internal support and positive cracks at the positive moment zone on the bottom face in the middle of the spans.

4.1. Specimen S-T-1

The two-way continuous slab was cast fully without construction joint and losses in the reinforcement due to the corrosion and this specimen wasn’t strengthened. About 35 kN load, negative crack initiated with a fine and short crack that had 0.05 mm width in the top face at the internal support and started at the slab right edge in a direction perpendicular to the slab axis. At load 143 kN the crack combined with the first negative crack coming from the right side formed one obvious crack 0.4 mm width extended along the slab width, at this stage the continuity of the slab has vanished and, this crack kept getting wider with the load increasing till reached 0.85 mm wide at load 240kN. When the load reached 281kN the crack width was 2.1mm (see figure 6), and this loading stage considers as the ultimate load (failure load). The negative crack showed in internal support at mid of slab length because of the symmetry of loading on both sides of the internal support, and the direction of the crack was due to the direction of the internal support. Figure 6 illustrates the Negative crack width propagation verse loading.
At load 55 KN the first positive crack showed at the bottom face under the load points. The crack location was closer to the slab's centre than the slab's edge, the crack moved in direction of the slab's centre. With load increased cracks that had the same pattern appeared. At a load of 155 kN, new cracks appeared in the area in which its location can be described as around the first cracks zone and longer than the first cracks. At a load of 199-200 KN, new cracks appeared in the area closer to the edge of the slab in direction of the centre and combined with first zone cracks. It was clear there weren't any crack formed near the corners close to the outer supports, or cracks could be referred to it as shear cracks. At the late stages of loading, there were one or more of the positive cracks which were spread in the bottom face of the slab and extend to the sides edges and appeared there. The cracks started in an area where was the max deflection at the slab centre with load increased new cracks under the loading point appeared and extended to the slab sides. The mostly uniform distribution of cracks in the bottom face reflects the symmetrical four-point load and four sides support. It was expected in the load-deflection curve (see Figure 7) to notice that deflection increased as long as the load increased until the failure of the specimen. There was a clear difference in deflection increment before 110kN load and after it which could be explained with: after 110kN more cracks showed and these cracks reduce the resistance of the slab to load and increase the deflection. As mentioned before, two LVTDs used each one of them was at the slab centre and touched the bottom face which where was expected to find the maximum deflection. As a result, we have two load-deflection curves and theoretically, they should be identical as long as there weren't any lose in steel reinforcement and there was symmetry in loading and supports. There was a little of a difference around +-0.1 mm, which could be explained due to the difficulty of attaining the complete symmetry or undetected initial values or due to accidental movement for the test frame. According to the load-deflection curves, the construction joint and corrosion reduce the specimen's ductility and loading capacity. The effects of corrosion and construction joint were manifest after 65 kN which indicates the remaining steel and the epoxy in the construction joint material were sufficient till pass a certain load point.
4.2. Specimens (S-T-2 and S-T-3)

These specimens had 48.89% average losses in the steel area due to corrosion and had a vertical construction joint. About 43 kN load, the crack in the construction joint was initiated with a fine and short crack that had 0.15 mm width in the top face and started at the slab right edge in a direction perpendicular to the slab axis. At a load of 85 kN, a new crack showed on the left side of the slab and extend up to the top surface of the slab. At load 95 kN the crack combined with the first negative crack coming from the right side formed one obvious crack 0.5 mm width extended along with the slab width, at this stage the continuity of the slab has vanished. This crack kept getting wider with the load increasing till reached 1.5 mm wide at load 134 kN increment of crack’s width was different before the 85 kN load and after it, the crack width at load 62 kN was 0.25 mm and it was 0.4 mm at 85 kN but it became 0.75 mm at load 113 (see figure 10), this increase in crack’s width increment indicates that the continuity of the slab was over somewhere around 85 kN. The negative crack showed at internal support in mid of slab length because of the symmetry of loading on both sides of the internal support, and the direction of the negative crack was due to the direction of the internal support. by conducting a comparison between the S-T-1 and the reference, it’s clear that negative cracks initiation wasn’t highly affected by the presence of the construction joint while the propagation and the extend of it were more influenced by them. At load 134 the increase in the crack width in this specimen was 275% compared to the S-T-1 specimen which was without construction joint and corrosion.  

At load 52 kN the first positive crack showed at the bottom face under the load points, this crack was closer to the centre than the edge of the slab and take the direction of the slab centre. With load increased, new cracks with the same pattern appeared. At a load of 95 kN, new cracks appeared in the area in which its location can be described as around the first cracks zone and longer. At a load of 134 kN, new cracks appeared in the area close to the edge of the slab in direction of the centre and combined with previous cracks. It’s clear the cracks in the corroded part were denser and generally take radial direction or tree-like, this pattern for cracks distribution reflects the location of the loading points on the top surface. When the load did increase, the crack width also got bigger which affect the serviceability of the member. That was clear there weren't any crack formed near and parallel to the supports which could be referred to it as shear cracks. Cracks spread over the tension face of the slab in zones, first cracks appear in the early stages of load were located close to the centre of the slab. With load increasing both the length and width of cracks were also increased and other cracks appeared surrounding the first zone. With load increased cracks got wider and became closer to the edges and the closer cracks to supports were coming from bigger loads. By taking into consideration the corrosion was partially and in one span, this reduces the effect of corrosion on the crack’s appearances in the corroded span compared to the not corroded span. At the late stages of loading, there were one or more positive cracks that were spread in the bottom face of the slab and extend to the sides edges and appear on the sides (see Error! Reference source not found.). The cracks started in an area where was the max deflection at the slab centre with load increased new cracks under the loading point appeared and extended to the slab sides. The mostly uniform distribution of cracks in the bottom face reflects the symmetrical four-point load and four sides support. The failure mode in this species was a tensile flexural failure due to the yielding of the steel reinforcement.

From the load-deflection curves of the two specimens, it’s noticed that at the early stages of loading the two curves were similar which is expected behaviour since the response of the member to the applied load depends on the concrete before the applied load exceeds the strength of the concrete. Even when the load exceeds concrete strength the two curves were much similar. With the load increased, the difference between the two curves became clearer and the curves took a different path. The difference between the two curves in the increment of deflection highlights the effect of steel lost (see Figure 10). The construction joint and corrosion reduce the specimen's ductility of the corroded part. The effects of corrosion and construction joint were manifest after 65 kN which indicates that the remaining steel and the epoxy material were sufficient till pass a certain load point.
Figure 8. Load Deflection Curves for S-T-4

Figure 9. Cracks in the S-T-2

Figure 10. S-T-3 Load-Deflection Curves and Negative crack width of S-T-3 Specimen
4.3. Specimen S-T-4

The specimen had a 55.86% loss in the negative reinforcement steel area due to corrosion and had a vertical construction joint and strengthens with steel plates. At a load of 60 kN, negative cracking started with a fine crack 0.05 mm in the top face at the internal support and started at the slab left edge in a direction perpendicular to the slab axis. At a load value of 87 kN, the previous crack combined with another crack that came from the right side, both of them formed one obvious crack 0.25 mm extends along the slab width. This crack maintained the same width. At the ultimate load, the crack width became 0.7 mm. At load 126 KN, there was a new crack formed on the slab top surface and was extended along the slab width at the same line where the steel plate’s bolts were fixed. The crack kept getting wider as a response to load increase and at load 215 it became 0.55 mm width (see figure 12). This kind of crack was not familiar in the previous specimens and appeared here due to the holes of the bolts which were used to tie the steel plate to the concrete that became a weakening point in the slab which encouraged the formation of this kind of a negative tension crack. The negative crack showed at internal support in mid of slab length because of the symmetry of loading on both sides of the internal support, and the direction of the negative crack was due to the direction of the internal support. The strengthening in this specimen method increased the first negative crack load and constrain the crack from expanding, and compensate for the joint present regarding the negative cracking.

At load 60 kN the first positive crack showed at the bottom face it was short and under the load point with load increase, more cracks appeared and the older cracks became longer. As long as the load became bigger the cracks extend more and combined. The cracks concentrated in the middle of the slab bottom and it's clear the cracks in the corroded part were dense. At the late stages of loading, there were one or more positive cracks that were spread in the bottom face of the slab and extend to the side edges. It's clear the cracks in the corroded part were denser and generally take radial direction or tree-like. This pattern for cracks distribution reflects the location of the loading points on the top surface of the slab. It was clear there weren't any crack formed near and parallel to the supports which could be referred to it as shear cracks. The failure for this specimen was de-bonding of the steel plats followed by a flexural tensile failure due to the yielding of the reinforcement steel bars at an ultimate load of 240 kN. This failure mode was identical to other specimens (see figure 11).

From the load-deflection figure (12) curve can notice that at the early stages of loading the two curves were similar which expected behavior is since the response of the member depends on the concrete before the load exceeds the concrete resistance. Even when the load exceeded concrete resistance the curves were much similar. With the load increased, the difference between the two curves became clearer and the curves took a different path. The difference between the load-deflection curves which were related to the fact that the deflection of the corroded span was higher was highlighted the effect of steel loss. The difference between the two curves in the increment of deflection highlights the effect of steel lost. The construction joint and corrosion reduce the specimen's ductility of the corroded part. The effects of corrosion and construction joint were manifest after 65 kN which indicates that the remaining
steel and the epoxy material were sufficient till pass a certain load point. This method of strengthening increased the ductility of the specimen by increasing the deflection at the ultimate load up to 50 % compared to the reference. This method of strengthening was very efficient regarding the ultimate load compared to the reference especially if know that this specimen has 6.97% losses more than the reference specimen and this specimen showed the lower deflection compared to other specimens

![Figure 1](image1.png)

**Figure 12.** Load-Deflection Curves and Negative Crack Width of S-T-4 Specimen

### 4.4. Specimen S-T-5

The specimen had a 51.6% loss in the negative reinforcement steel area due to corrosion and had a vertical construction joint and strengthened with the CFRP bars NSM technique. At a load of 27 kN, negative cracking started with a fine crack 0.05 mm in the top face at the internal support and started at the slab left edge in a direction perpendicular to the slab axis. At load 85 kN the previous crack combined with another crack that came from the right side formed one obvious crack 0.3 mm width extended along the slab width. When the load increased, the crack width also increased. the crack width became 1.1mm at a load of 230kN (see figure 14). The negative crack showed at internal support in mid of slab length because of the symmetry of loading on both sides of the internal support, and the direction of the negative crack was due to the direction of the internal support. The load at the initiation of the negative crack in this specimen was below the reference and this behaviour could be explained on the base that the slots that carved in the tension face of the specimen for CFRP bars laying purpose weakened the specimen.

This method provided constrain which prevent the crack from free expanding. At load 60 kN the first positive crack showed at the bottom face, it was short and under the load point. On both sides, when the load increased more cracks appeared and combined and the older crack got longer became and approached the slab edge. It’s was clear the cracks in the corroded part were denser and take radial direction or tree-like pattern and this distribution reflected the load points' location in the slab top surface. At the not corroded part, some cracks had the same pattern as the corroded part’s cracks but less density. There were longitudinal cracks parallel to the continuity slab axis of the slab under the CFRP bars which indicates that grooves which used to put the CFRP bars weakened the slabs. At the late stages of loading, there were one or more positive cracks that extend to the sides edges and appeared on the sides of the slab.
This pattern for cracks distribution reflects the location of the loading points on the top surface of the slab. It was clear there weren't any crack formed near and parallel to the supports which could be referred to it as shear cracks. The failure for this specimen was de-bonding of the CFRP bars followed by a flexural tensile failure due to the yielding of the reinforcement steel bars at an ultimate load of 243 kN. This failure mode was identical to other specimens.

From the load-deflection curve (see Figure 14) can notice that at the early stages of loading the two curves were similar. This similarity is expected behavior since the response of the member depends on the concrete before the load exceeds the concrete resistance. Even when the load exceeds concrete resistance and steel starts to take the role the two curves were generally similar. With load increase, the difference between the two curves starts to become clearer and took a different path which clarifies the effect of steel loss. The failure in treatment was de-bonding of the CFRP bars out of the concrete, this kind of failure similar to all other strengthening methods deflection. This type of strengthening led to an increasing compared to the reference specimen with 81.3% regarding the ultimate load, 67.4% regarding the construction joint crack load, 13.2% regarding the first positive crack load. This strengthening method able to compensate for the losses of the steel and also improving the structural behavior of this member. This method of strengthening increased the ductility of the specimen by increasing the deflection at the ultimate load up to 83.3% compared to the reference.

4.5. Specimen S-T-6
The specimen had a 65.37% loss in the negative reinforcement steel area due to corrosion and had a vertical construction joint and strengthened with the CFRP strips. At a load of 73 kN, negative cracking started by a fine crack 0.05 mm in the top face at the internal support and started at the slab right edge in a direction perpendicular to the slab axis. At load 107kN the previous crack combined with another
crack that came from the left side forming one obvious crack 0.35 mm width extended along the slab width. With the load increased, the crack width also increased, it became 1.75 mm at a load value of 248 KN. The negative crack showed at internal support in mid of slab length because of the symmetry of loading on both sides of the internal support, and the direction of the negative crack was due to the direction of the internal support. The strengthening in this specimen method increased the first negative crack load and constrain the crack from expanding, and compensate for the joint present regarding the negative cracking (see figure 16)

At load 59 kN the first positive crack showed at the bottom face, in this specimen, we notice that the positive crack appears before the negative crack due to the CFRP strips efficiency in delating the negative cracks appearing, it was short and under the load point. On both sides with the load increased, more cracks appeared and the older became longer, as long as the load became bigger the cracks extended more and combined in the bottom face, the cracks concentrated in the middle, and with the load increase cracks approached the edge of the slab, it’s was clear that the cracks in the corroded part were denser and take radial direction or tree-like, this pattern for crack distribution reflected the location of the loading point on the top surface of the slab. At the late stages of loading, there were one or more positive cracks that extend to the sides edges and appeared on the sides of the slab. This pattern for cracks distribution reflects the location of the loading points on the top surface of the slab. It was clear there weren’t any crack formed near and parallel to the supports which could be referred to it as shear cracks. The failure for this specimen was de-bonding of the CFRP strips followed by a flexural tensile failure due to the yielding of the reinforcement steel bars at an ultimate load of 271 kN. This failure mode was identical to other specimens.

From the load-deflection, (see figure 16) curve can notice that at the early stages of loading the two curves were similar. This similarity is expected behavior since the response of the member depends on the concrete before the load exceeds the concrete resistance. Even when the load exceeds concrete resistance and steel start to take the role the two curves were generally similar. With load increase, the difference between the two curves starts to become clearer and took a different path which clarifies the effect of steel loss.

This type of strengthening led to an increasing compared to the reference specimen with 102.2% regarding the ultimate load, 69.7% regarding Negative crack at construction joint crack load, 11.3% regarding the first positive crack load. This strengthening method able to compensate for the losses of the steel and also improving the structural behavior of the member. This method of strengthening increased the ductility of the specimen by increasing the deflection at the ultimate load up to 124.5 % compared to the reference.
Figure 16. Load-Deflection Curves and Negative Crack Width of S-T-6

Figure 17. Load-Deflection Curves for Tested Specimens

Figure 18. Cracks Width of Tested Specimens
Table 5. Ultimate load, load at First Negative and Positive Crack

| Specimen | Strengthening                      | Load at First Negative Crack (kN) | Load at First Positive Crack (kN) |
|----------|------------------------------------|-----------------------------------|-----------------------------------|
| S-T-1    | No Corroded, No strengthened, No Construction Joint | 35                               | 55                                |
| S-T-3&2  | Corroded and Construction Joint (Reference) | 25                               | 52                                |
| S-T-4    | Strengthened with Steel Plates     | 60                                | 60                                |
| S-T-5    | Strengthened with CFRP NSM Bars    | 27                                | 60                                |
| S-T-6    | Strengthened with CFRP Strips      | 73                                | 59                                |

The failure mode for all strengthened specimens was similar which was de-bonding between the concrete and the strengthening followed by reinforcement yielding. According to figure (17), it clear that at the early stages of loading the deflection behavior was similar for all specimens, with the load increased the differences between the curves started to manifest indicating the role of corrosion and how far the strengthening was efficient especially CFRP which gave a high ultimate load with low deflection associated with highest corrosion percent. Figure 19 illustrated the ultimate load for each specimen associated with losses in the steel which indicates the efficacy of each strengthening method in regards to the losses percent.

5. Numerical Analysis

5.1. Introduction

This part investigates numerically the combined effects of a construction joint and reduction in the cross-section area of negative reinforcement on the structural behavior and the efficiency of strengthening technique on two-way continuous slabs. All the structural details which were in the experimental part was adopted in the numerical analysis. S-T-3 and S-T-6 were selected for the numerical analysis. For the previous reason, the ABAQUS (software) is used to model the specimen and simulate the loading.
The program outcomes are represented visually as stresses distribution diagrams, load-deflection curves, and cracks pattern. The results from the numerical analysis were compared to the experiment results.

5.2. Parts, Properties, and Assembly

The finite element analysis program ABAQUS is used to create the model which represents the specimen in the experimental part. The model creation process required drawing the component of the specimens using the toolbars provided by the software which are the concrete and, the reinforcements, the strengthening, loading, and supporting parts with its dimensions and exact location. After drawing all model parts, the properties of the material either elastic or inelastic which is gained from the control test were assigned to the parts. To make the numerical analysis represent the actual structural behavior of the specimens the model should be as far as possible is identical to the experimental specimens' bars of negative reinforcement (see Figure 20 & 21).

5.3. Interaction Between the Parts

To model a composite member and after drawing each element separately and gave each one of them a proper section it was necessary to connect the parts by assigning the suitable connect type. (see Table 6).

| Parts Type of Interaction |
|---------------------------|
| concrete-steel bars       | embedded |
| concrete-loading plate    | Rigid body |
| concrete-CFRP bars        | embed |
| concrete-steel plate      | Surface to surface (cohesive) |
| Old Concrete-New concrete | Surface to surface (cohesive) |

5.4. Finite Element Modeling

Concrete, loading plate, and support, and other parts were modeled in the finite element program (see Table 7). The concrete elements were modeled using (T3D2) 3D 8nodes bricks elements to gain sufficient distribution of stress while the steel bars meshed to linear truss element (2 nodes elements). Depending on the convergence study for mesh 35mm mesh size was adopted.

| Part      | Element Type          |
|-----------|-----------------------|
| concrete  | Linear hexahedron, type C3D8 |
| support   | Linear hexahedron, type C3D8 |
| loading Plate | Linear hexahedron, type C3D8 |
| steel Bars | Linear line, type T3D2 |
| CFRP Bars  | Linear line, type T3D2 |
| steel Plate  | Linear hexahedron, type C3D8 |
5.5. Loading and Boundary Conditions
The loading plate was a square steel plate with dimensions of 10 mm thick and 100mm side length. It was placed at a distance of L/4 between the supports in both directions for each span, the load presented in the program by a uniform pressure after divided the load by the area of the loading plate, the boundary conditions were applied at the same location of supports in the experimental work which restrained the slab in four sides in addition to the internal support. The vertical displacement was constrained in all supports, the third supports in z-direction that aligned with slab width was restrained in three directions X, Y, and Z (see Figure 22).

![Figure 22: Loading and Boundary Conditions](image)

5.6. Analytic Results
After completing all the requirements and all the data that needed to run the simulation, the program starts to analyze, and the result was interpreted visually by the load-deflection curve and deformed shape. According to the changes in the parameters such as the corrosion percent and the treatment the ultimate load and deflection will vary. The convergence between the numerical and experimental results is indicated to the reliability of the model (see Table 8&9).

| Specimen | Losses in Steel Area % | Ultimate load (kN) | 
|----------|------------------------|--------------------|
|          |                        | Exp. | Num. | Num. Exp. *100 |
| S-T-3    | 48.89                  | 134  | 136  | 101.5        |
| S-T-6    | 65.37                  | 271  | 300  | 110.7        |

Table 8. Experimental and Numerical Result of S-T-3 & S-T-6 Specimens

| Specimen | Deflection at Ultimate-Load (mm) | Failure Mode |
|----------|---------------------------------|--------------|
|          | Exp. | Num. | Num. Exp. *100 | Exp. | Num. |
| S-T-3    | 6    | 6.4  | 106.7          | Flexural Tensile Failure |
| S-T-6    | 13.4 | 20.3 | 151.5          | CFRP Strips De-Bonding Followed by Flexural tensile Failure |

Table 9. Numerical &Experimental Deflection and Failure Mode of S-T-3 & S-T-6 Specimens
5.7. **Cracking Pattern**

According to the observations from the experimental test for the specimens, the crack pattern reflected the load point location on the top face of that slabs which was radical lines pointing the slab center concentrated in the bottom face and extended through the depth of the slabs in addition to the tension area in the negative moment at the internal support which results in the opening of the joint. The analytical part showed tension damaged areas which can be developed into cracks identical to the experimental part.
5.8. Failure Mode

Figs. 27 and 28 shows the experimental an numerical failure modes obtained. Good correlation can be noticed.

![Figure 27: Failure Mode of S-T-3 Specimen](image1)

![Figure 28: Failure Mode of S-T-6 Specimen](image2)

6. Conclusions

- The combined effect of the presence of construction joints and losses in negative reinforcement due to the corrosion has a tremendous effect on the ultimate load and the deflection and serviceability in addition to the appearance of cracks and their propagating in the positive and negative moment zones.
- The losses in the steel area of negative reinforcement at internal support which was responsible for resisting the negative moment, affect the slab's continuity.
- The similarity of crack's patterns and failure mode in all strengthened specimens could be related to the that all the corrosion happened on one side of the slab and there weren’t large differences in the corrosion percent between specimens.
- All the proposed strengthened technique was able to compensate strength losses due to corrosion and the presence of construction joint and improving the structural behavior of the member.
- The simplicity and ease of proposed strengthening that doesn't necessarily need professional workers for implementation make them a good choice among the other alternatives which can be used to compensate for the losses in steel.
- The cross-section of CFRP bars (NSM) was 59% higher than CFRP strips and despite this fact, the specimen that strengthened by CFRP strips provide a higher ultimate load value than NSM bars which can clarify by since the failure did not happen in the CFRP itself but it was debonding between the treatment and the concrete so the determinate factor was the contact surface between the two parts and the sheets have a higher contact surface compared to the bars.
- The specimen with (0%) corrosion and without construction joint showed the highest ultimate load and lowest deflection compared with all other specimens.
- Regarding ultimate load, the specimen who strengthened with CFRP strips gained the highest ultimate load values compared with other strengthening methods (102.2% compared to the reference) even it has the highest losses percent between other specimens (65.37%).
- At load 240kN the specimen which strengthened with steel plates showed the lowest deflection compared to other treatments.
- Specimen strengthened with CFRP bars showed the highest deflection value.
- All strengthened specimens had almost the same load value for the first positive crack (60kN 15% increase compared to the reference).
The specimen which strengthened with CFRP strips showed the highest load value for the first negative crack between all other specimens (192% increase compared to the reference).

The created model by ABAQUS predicts to an acceptable degree the structural behavior of a member and highlighted the potential failure regions with high accuracy.

A good convergence between the numerical and experimental works regarding the ultimate load and deflection.

The stiffer behavior in numerical analysis is related to many factors such as the assumption of homogeneous material for concrete while it is heterogeneous material; neglect the microcrack which formed due to drying shrinkages or thermal exchanges or microbore poured in concrete and full-bond assumption between the concrete and steel bars regarding material properties dimensions and loading conditions. All specimens' parts resemble in one figure (see Figure 20). The losses due to the partial corrosion in the negative reinforcement in the experimental study represent in the numerical study as a reduction in the cross-section area in the steel.

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