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# BEHAVIOR OF STRENGTHENED RC BEAMS USING DIFFERENT TECHNIQUES UNDER FATIGUE LOADING

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## ABSTRACT

Strengthening and retrofitting are becoming available options for those important structures as bridges, factories and parking garages which are more economical to be strengthen than to be demolished. This paper presents an experimental program to investigate the behavior of strengthened RC beams using different techniques under fatigue loading. Experimental program including, four specimens, one acts as a control and the rest of specimens were strengthened with near surface mounted (NSM) CFRP strips, externally bonded (E.B) steel plate and casting of an ultra high performance strain hardening cementitious composite (UHP-SHCC) layer at the beam soffit techniques. Having the same criteria on the comparative study the area of each strengthening materials was deliberately chosen to obtain the same force gained at the tension side of the strengthened beam. The results indicated that strengthening NSM CFRP strengthening technique is the most convenient technique compared to the other strengthening techniques for strengthening RC beams subjected to fatigue loads.

**Keywords:** RC beams, Strengthening techniques, Fatigue life, Energy dissipation, stiffness degradation, Deflection.

## INTRODUCTION

Strengthening and upgrading of structures subjected to cyclic loads as bridges or parking garages is required for several reasons including extension of design life, functional change, mechanical damage and environmental effects, updated design requirements, and errors due to design and construction. It is both environmentally and economically desirable to upgrade structures rather than rebuild them, particularly if rapid, effective and simple strengthening methods are available [1]. There are many methods available for strengthening existing deficient structures under fatigue load. Significant works exist on the fatigue behavior of reinforced concrete beams with state-of-the-art reports by "ACI Committee 215 (1982) [2], CEB-FIP, 1988 [3] and Mallet, 1991 [4]". Susceptibility of a reinforced concrete beam to fatigue vary throughout the member as fatigue is dependent on the stress level of its components at each section, that are reinforcing concrete and steel. The dominant failure can vary depending on the design of reinforced concrete beam. Under reinforced members have their flexural fatigue performance dominated by the main longitudinal steel, but heavily reinforced members may fail in flexure or in shear. As reported by "ACI Committee 215 (1996) [5] the fatigue life of these beams is usually controlled by the fatigue life of the reinforcing steel. Bishara (1982) [6] studied the behavior of reinforced concrete beams under fatigue loads. He observed that the strain of the steel was increased under cyclic loading at the locations of the flexural cracks of 7%. He also found that the flexural rigidity of the beams was reduced as a result of a noted increase in deflection that accompanied concrete softening. Heffernan (1997) [7] and Heffernan and Erki (2004) [8] studied the fatigue behavior of RC beams. They noted that an increase of 2% to 6% in the stress of the reinforcing bars that was attributed to softening of concrete, which occurred in beams subjected to cyclic load. Hassanean etal. (2013) [9] study the strengthening and repairing of reinforced concrete beams subjected to short time repeated load by using mixed steel fibers concrete jacket (MSFCJ). Results showed that, strengthening and repairing with MSFC jacket decreased the number of cracks and they were concentrated in the middle third avoiding the forming of shear cracks. Oh et al. (2003) [10] investigated the behavior of RC beams that were flexure strengthened with steel plates under static and fatigue loads. The results showed that increasing the thickness of plates effectively decreases the mid-span displacements, tensile rebar strains, and compressive rebar strains. Christos et al. (2001) [11] performed a study to examine the effects on the fatigue performance of reinforced concrete beams of adding GFRP composite reinforcement. In all the fatigue specimens, a failure was initiated by a failure in fatigue of the reinforcing steel. Aidoo et al. (2004) [12] concluded that the fatigue life of the RC beams was increased by the application of FRP strengthening due to a reduction in the tensile stress carried by the steel. They also stated that the observed increase in the fatigue life was depending on the quality of the bond between the concrete and the composite materials. Wang et al., (2007) [13] made an experimental study of FRP-strengthened RC bridge girders subjected to fatigue loading. Carbon and glass fibre-reinforced polymers (CFRP and GFRP) saturated in an epoxy resin matrix were used to enhance the service load-carrying capacity of the bridge. The test result concludes that the bonded epoxy/FRP laminate shows better resistance to fatigue loading than the steel reinforcement. When the bar had fractured the CFRP laminate compensated for the strength lost by the ruptured. Badawi and Soudki, (2008) [14], studied the fatigue performance of the reinforced concrete (RC) beams strengthened with prestressed near surface mounted (NSM) carbon fiber reinforced polymer (CFRP) rods under cyclic loading at various load ranges. The test results showed that using NSM CFRP rod for strengthening increased the fatigue life with respect to that of the control beams at all load ranges. The primary mode of failure was by fatigue rupture of the internal steel reinforcement. A second mode of failure was a fatigue failure of the bond between the prestressed CFRP rod and the epoxy, which occurred in a few cases. Another observed failure mode was by fatigue rupture of the prestressed CFRP rod. Oudah and El-Hacha, (2012) [15], studied the fatigue performance of Reinforced Concrete (RC) beams strengthened using NSM CFRP rods. Test results showed that all strengthened beams experienced deflection increase lower than that of the un-strengthened beam which indicates the efficiency of the strengthening process in reducing the damage accumulation. Abd El-Hakim Khalil et al., (2017) [16] studied the behavior of RC beams strengthened with strain hardening cementitious composites (SHCC) subjected to monotonic and repeated loads. The recorded tests showed that use additional reinforcement embedded in the strengthening layer for beams strengthened with UHP-SHCC become sufficient at certain limit to eliminate the observed early strain localization and to gain adequate ductility under both monotonic and repeated loading. Another important conclusion is the strengthening of RC structures using an unreinforced UHP-SHCC layer may lead to a brittle failure especially in case of repeated loading.

## RESEARCH SIGNIFICANCE

Strengthening and retrofitting becoming available options for those structures which are more economical to retrofit than to demolish. In such circumstances, there are various methods for repairing and strengthening of R.C structures subjected to cyclic loading. All these available techniques can be successfully used but have some limits. So selection of strengthening technique is one of the most important factors to fulfil the successful strengthening. This research investigates the structural behavior of strengthened RC beams using different strengthening techniques under fatigue loads.

## EXPERIMENTAL PROGRAM

This section describes the main characteristics of the tested specimens, the properties of their constituent materials, the experimental loading apparatus, the instrumentation and the data acquisition systems.

### Materials

The used concrete was made from a mix of ordinary Portland cement, natural sand, and gravel. The averaged compressive strength of the used substrate concrete was determined to be 30 MPa based on the compressive test results of six cylindrical specimens (150D × 300mm). Deformed steel bars 12mm were used for the tension reinforcement while steel bars 8mm were used for the compression reinforcement. Both types of steel bars had a yielding stress of 400 MPa and modulus of elasticity of 200 GPa. For stirrups deformed steel bars 10mm were used, which the yield stress and modulus of elasticity of 412 MPa and 200 MPa, respectively. Three different strengthening materials were used: (1) X-Wrap pp CFRP strips with tensile strength of 3000 MPa , and modulus of elasticity of 165 GPa, and rupture strain of1.8%, (2) high tensile steel plate with yield stress 411 MPa and (3) ultra-high performance strain hardening cementitious composite. X-Wrap plate adhesive was used to bond the CFRP strips into the grooves and bond the steel plate on the soffit of the beam. X-Wrap plate adhesive had a tensile strength and a modulus of elasticity of 23 MPa and 7 GPa, respectively. The mix proportions of the UHP-SHCC material was used in this study are listed in Table 1 .Water to binder ratio (W/B) was 0.20. Low heat Portland cement (density: 3.14 g/cm3) was used, and 15% of the design cement content was replaced by silica fume. Quartz sand with diameter less than 0.5 mm was used as a fine aggregate. Polypropylene (Pp) fiber was chosen for UHP-SHCC and its volume in mix was 2.0%. The diameter and length of the (Pp) fibers were 0.012 mm and 6 mm, respectively. The tensile behavior of the used UHP-SHCC was characterized by testing of three specimens using uniaxial tensile test. The dimensions of the practical size test specimens (60 x 150 mm) were selected similar to the average value of those used for beam's strengthening application. The averaged tensile strength and ultimate tensile strain (strain at ultimate load) of the used UHP-SHCC at the age of 28 days were determined to be 8.02 MPa and 1.48% respectively.

**Table 1: Mix proportions of the UHP-SHCC**

| **Material** | Portland cement | Silica fume | Fine aggregate | Super plasticizer | Expansion agent | Pp fibers (6mm) | Water |
|---|---|---|---|---|---|---|---|
| **Weight kg/m$^3$** | 1243 | 223 | 149 | 14.9 | 20 | 19.5 | 292 |

### Specimens

Four reinforced concrete beam specimens were designed, constructed, and tested under four point loading. The program contains four specimens, one unstrengthened beam BC and three strengthened beams with NSM CFRP strips, E.B steel plate with end anchors and UHP-SHCC techniques, BF, BS and BU, respectively. All specimens have the same dimensions are shown in Fig. 1. It had a total length of 3000 mm with a clear span of 2700 mm. Besides, the reinforcement details were fixed for all specimens. They were reinforced with 2D12mm bars as tension reinforcement and 2D8mm bars as compression reinforcement, which designed according to ACI 318M-14 [17], to make sure tension failure collapse. To prevent shear failure, stirrups diameter is 10mm were spaced 100 mm for length equal to 1000 mm from right and left support, however the mid length 700mm, the stirrups were spaced at 230mm. A summary of these beams is given in Table 2.

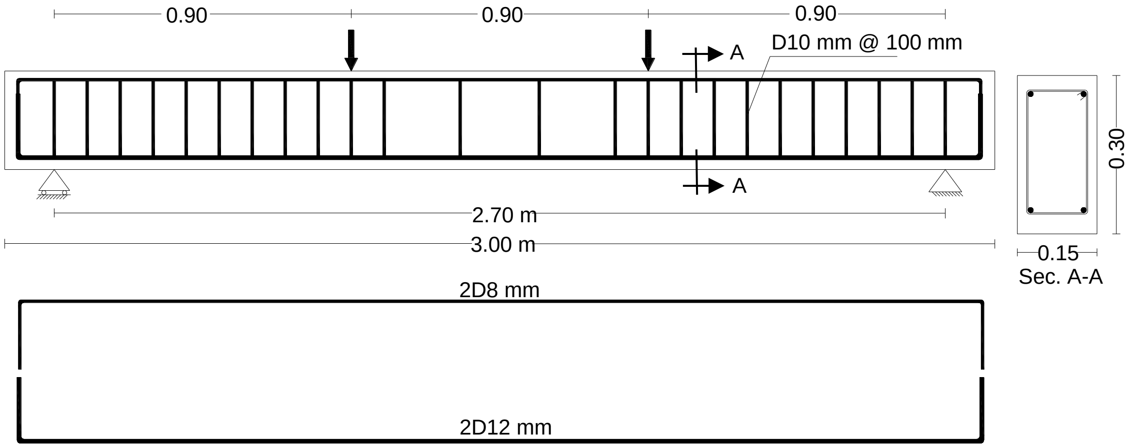



**Fig. 1: Beam details**

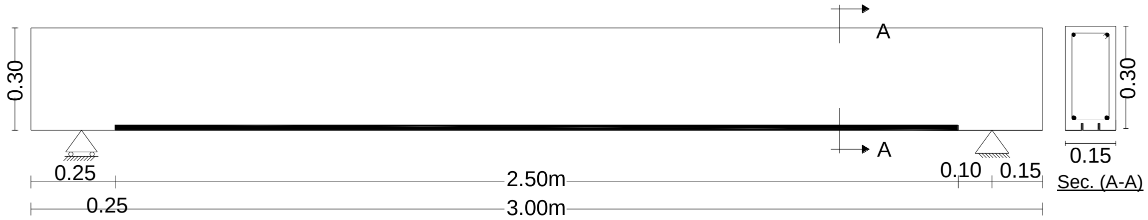
**Table 2: Description of tested beams**

| Specimen | Dimensions | Strengthening technique | Material | Dimension of material |
|---|---|---|---|---|
| BC | 150×300×3000 | — | — | — |
| BS | 150×300×3000 | Externally bonded plate with end anchors | Steel | 1.3×135 mm plate |
| BU | 150×300×3000 | Adding a layer of mortar | UHP-SHCC | 60×150 mm layer |
| BF | 150×300×3000 | Near surface mounted strips | CFRP | 1.2×10 mm strips |

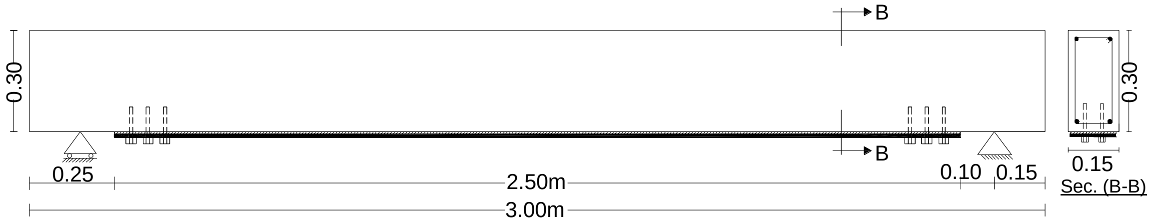
**Strengthening techniques**

Fig. 2 shows the used different strengthening techniques. Three different techniques (NSM CFRP, E.B. steel plate with end anchors and applying UHP-SHCC layer) are used for strengthening. Regarding to the different tensile strength of the used strengthening materials, the area of each strengthening material was deliberately chosen to obtain the same force gained at the tension side of the beam, to have the same criteria on the comparative study. Two carbon fiber strips with a cross section of 1.2 x 10 mm, a steel plate with cross section of 1.3x135 mm and a layer of UHP-SHCC with cross section of 60x150 mm were selected for the comparative study.
As shown in Fig. 3, the strengthening steps of NSM CFRP technique. After marking the layout the grooves were formed in the tensioned side of the beams by making two saw cuts and finishing off with a manual hummer to chisel any remaining concrete between cut paths, finally clean the groove and eliminate any residual dust with compressed air. According to ACI 440.2R-08 [18], grooves with size 6 x 20 mm$^2$ were cut for strips with a cross section of 1.2 x 10 mm$^2$. Epoxy resin was used to install the strips in the concrete a grooves. For externally bonded steel plate technique, Fig. 4 shows the main steps of the technique. The first step was surface preparation to clean the surface by removal of all possible surface contaminations, then drilling holes for anchors. Slightly larger clearance holes of 12mm diameter were provided for tolerances in fabrication and drilling of holes in the substrate and steel plates. Six bolts at each end of diameter 10mm, grade 8.8 are fastened in drilled holes. Number of bolts and spacing between centers of them are designed according to the Egyptian code of practice for steel construction and bridges (ASD code) [19] to avoid failure due to shear of bolts or bearing of the steel plate. A layer of high performance Epoxy adhesive with 1mm thickness is applied to the substrate concrete. Finally, the steel plate is placed on the layer, and hammered down slightly to remove air voids between the plate and the adhesive layer. Then the bolts are tightened sufficiently to ensure that sufficient contact is achieved between the different parts.

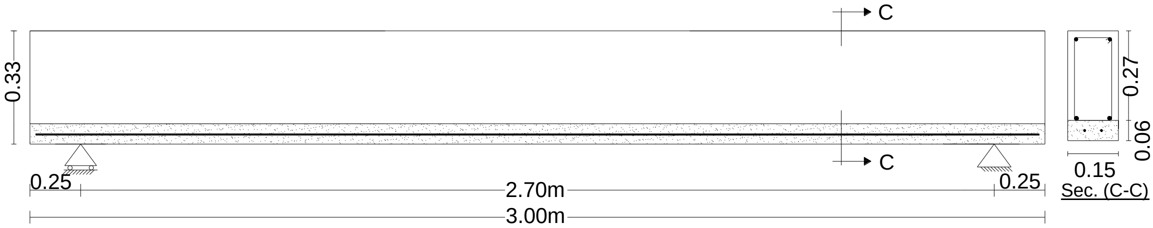
Applying of (UHP-SHCC) Layer technique was conducted through three main steps as shown in Fig. 5 The first step is surface preparation for the reason as mentioned before. Mixing the ingredients layer and installing the layer. The strengthening layer was bonded to the concrete beam surface, using the epoxy adhesive. After casting of all strengthening layer, curing is done by cover all specimens with wet burlap.




**(a) BF**




**(b) BS**




**(c) BU**

**Fig. 2: strengthening schemes for the different techniques**

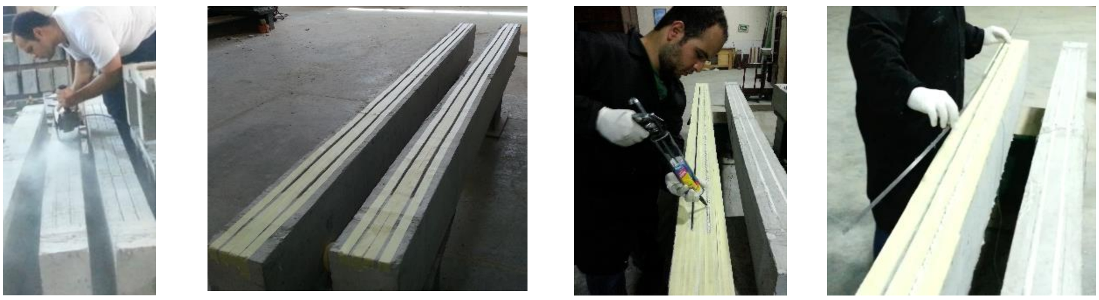


**Fig. 3: NSM CFRP application process**

**Fig. 4 E.B. steel plate with end anchors application process**

**Fig. 5: Applying UHP-SHCC layer process**

**Test setup and instrumentations**

As shown in Fig. 6 four point loading test setup, was used to evaluate all tested beams, the load is transmitted through MTS (Material Testing System) hydraulic actuator. The load from the actuator distributes into two points by spreader rigid beam. The load was applied using load-control mode. The vertical deflections at mid-point of the lower beam soffit and under the loading point were recorded using standard two different LVDT (linear variable displacement transducer) of sensitivity 0.01mm. Electrical strain gauges, of gage length 10mm and resistance 120 Ohms, were used to measure strains along the mid-span sections in longitudinal main reinforcement. Strain in substrate concrete and strengthening layer measured using electronically pi gauge with accuracy 0.001 mm and a gage length of 100 mm. All readings were recorded using data logger (sensor interface PCD-300A/320A).

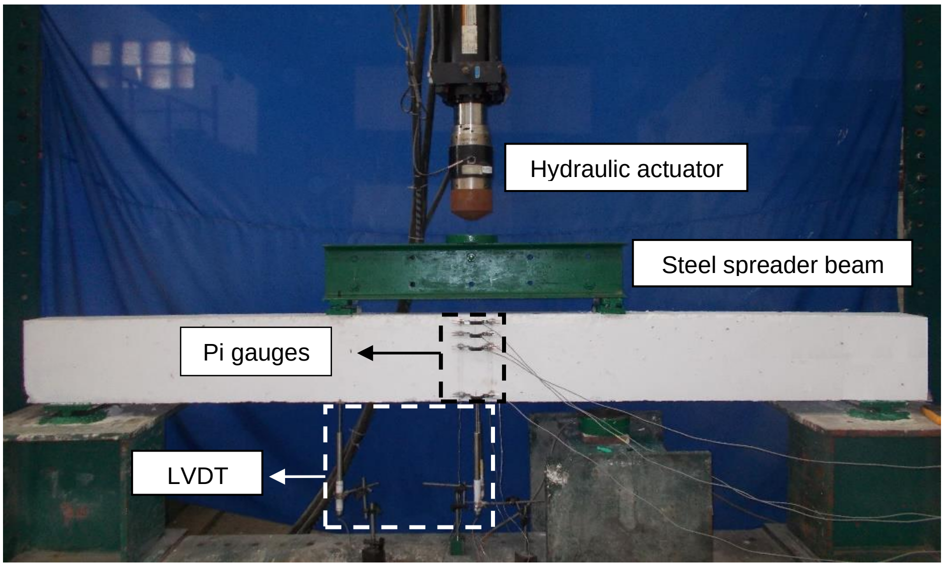



**Fig. 6: test setup and Instrumentations**

**Fatigue limits**

Fatigue tests were carried out in load control with sinusoidal applied loads being cycled at 2 and 3 Hz in the first 180,000 and following cycles respectively, as Barnes and Mays [20] suggested keeping the testing frequency below 3 Hz to avoid a hysteresis effect. A minimum load of 10% of the ultimate load was selected to avoid shifting of the beams and the effect of impact with cycling while generating stresses that could be considered representative of dead loads in beams. The maximum load was calculated to generate specified stress levels in the tension reinforcement as a fraction of the nominal yield stress (80% of nominal yield stress) according to ACI Committee 440 [18] since it would represents adequately the maximum stress that can be reached in real life applications. The load range was evaluated from static load test (paper static). The fatigue loading was applied for 180,000 cycles then the fatigue loading was stopped, and the load was released gradually for a rest period of about 10 h to count for the practical circumstances. At the end of the rest period, the beam was reloaded with the same sequence of loading for 360,000 cycles then the fatigue loading was stopped again for the next rest period, finally the beam was reloaded till fatigue failure. Based on field data and assuming that a linear relationship exists between vehicle speed and load frequency, the typical loading frequency of a truck varies between 0.159 and 8.724 Hz for a speed between 2 km/h and 110 km/h Lin JH [21]. Thus, a loading frequency which was implemented in this study since it would reasonably represent the actual vehicular loading frequency experienced in practice. Fatigue loads were applied using a sinusoidal variation between minimum and maximum values. A summary of fatigue loading limits is shown in table (3).

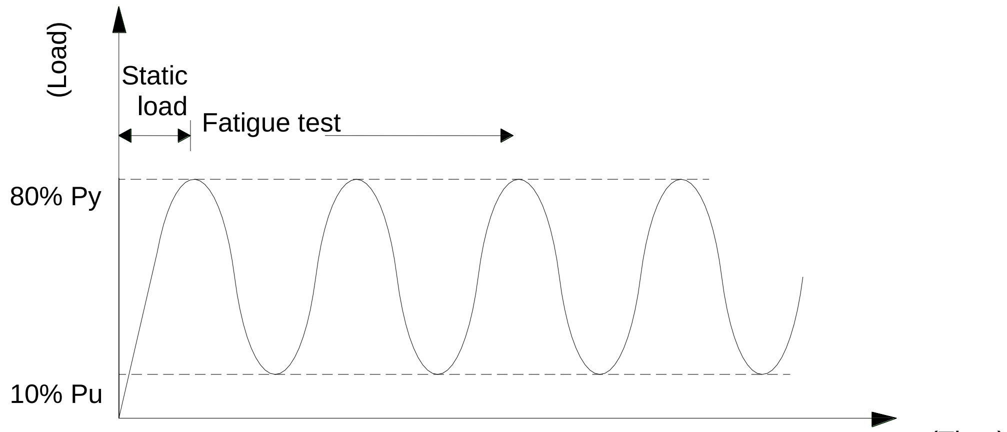



**Fig. 7: fatigue test procedure**

**Table 3: A summary of fatigue loading limits**

| specimens | Yielding load (kN) | Ultimate load (kN) | Fatigue limits | |
|---|---|---|---|---|
| | | | Min. load | Max. load |
| BC | 55.4 | 63.3 | 6.3 | 44 |
| BS | 94.1 | 108.8 | 10.8 | 75 |
| BU | 91.2 | 103.2 | 10.3 | 73 |
| BF | 91 | 115.6 | 11.5 | 72.8 |

## TEST RESULTS AND DISCUSSION

### Loads, crack pattern and failure modes:

The failure modes of all beams are shown in Figs. 7–10. All beams failed suddenly due to fatigue failure of the tension steel reinforcement due to cyclic fatigue loading. This mode of failure was expected since the stress range in the tension steel reinforcement was high enough to cause a fatigue failure in the steel. No indications of imminent failure were observed as the failure was brittle. Fatigue failure of the reinforcing bar was accompanied by a sudden propagation of the flexural cracks and crushing of concrete in compression. It is also noted a sudden increase in deflection and a significant drop in the beam stiffness followed the failure of the bars. The control specimen beam (BC), (BS), (BU), (BF) survived 783,571, 762,964, 720,865, and 730,746 cycles respectively. For beam BS a typically cracks formed at the substrate in the anchors region of the steel plate during the last stage of loading. Thus when a steel reinforcement had fractured, a fatigue fracture of the concrete layer below the tension reinforcing steel (concrete peel off) was occurred as shown in figure (4.33). For beam (BU) that was strengthened with UHP-SHCC layer, flexure cracks started to appear in substrate concrete and strengthening layer at the tension side of the beam during static loading stage. After 700,300 cycles, fatigue fracture occurred within the additional reinforcement in UHP-SHCC layer. This generated high stresses in the strengthening layer at the position of the cracks in the concrete substrate which led to decrease the fatigue life of the beam (BU) compared to other strengthened beams. Beam (BF) failed during fatigue loading due to fatigue fracture of the steel reinforcement with subsequent of fatigue rupture of CFRP strips as shown in Figure (4-35), as after the reinforcing bars had fractured the excess force generated in the CFRP strips. This is indicated that CFRP reinforcement exhibits a higher fatigue life than that of the main steel reinforcement. It has been show that the crack spacing in the strengthened beams was smaller than in unstrengthened ones. Moreover, the fatigue life of strengthened and unstrengthened beams is almost the same, even though the maximum applied fatigue load of the strengthened beams was 68 % higher than the maximum fatigue load of the control beam and this due to applying a same stress level on the main tensile steel bars at loading. With respect to the first steel bar rupture, the fatigue life of steel plate-strengthened beam (BS) was greater than that for their NSM CFRP strips counterparts (BF). This implies that the beams strengthened with NSM strips had higher local concentrated stresses than beams strengthened with steel plate, probably because the steel plate had larger contact areas on the lower faces of the beam and were thus more effective at restraining crack growth.

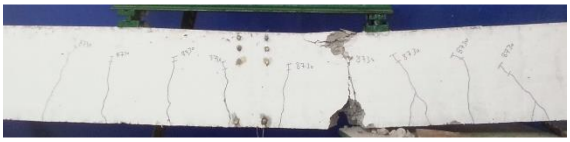


**Fig. 8: failure mode of beam BC**


**Fig. 9: failure mode of BS**

**Fig. 10: failure mode of BU**

**Fig. 11: failure mode of BF**

**Fig. 12: Fatigue failure due to a rupture in the reinforcing steel bar**

**Load Displacement Hysteretic Response:**

An important figure that must be generated to evaluate the structural performance under fatigue loading is the load-deflection hysteresis loops. For Figures (13-14-15-16), which show typical cyclic behavior for the unstrengthened and strengthened beams. Under fatigue loading, all beams demonstrated an increase in their midspan deflections in the early stages of their life during approximately the first 10 thousand cycles. The measured deflection of the beam remained relatively constant with an increasing number of cycles until approximately 95% of the life of the beam was attained. At this point, the rate of deflection increased sharply during last

5% of the beam life before failure occurred. The last stages in the life of all beams were characterized by the development of new cracks in the moment and shear zones and the widening of the existing cracks.

The unstrengthened concrete control beam, (BC), survived 783,571 cycles of fatigue loading under the specified live load with stiffness and displacement degradation. This can be seen in Figure (13), the rupture of one of the reinforcing bars became apparent after cycle 782,100. At this point, the stiffness of the beam changed significantly and an increase in mid-span deflection was evident. The width of the crack increased significantly near mid-span due to the energy release associated with a fatigue rupture and the greater beam deformations. The test was continued until cycle 783,571. The fatigue test was terminated automatically following the reinforcing steel fracture due to the displacement control input detector. For beams BS, BU and BF it was observed that the slope of the hysteresis curves (secant stiffness) degrades after rupture of the reinforcing bars and the failure occurred at cycles 762,964 ,720,865 and 730,746 respectively. In general, the load deflection behavior under cyclic fatigue load seems to vary according to the strengthening technique. This may be manifested by description the load deflection hysteresis loops of all beams as shown in Figure 17.

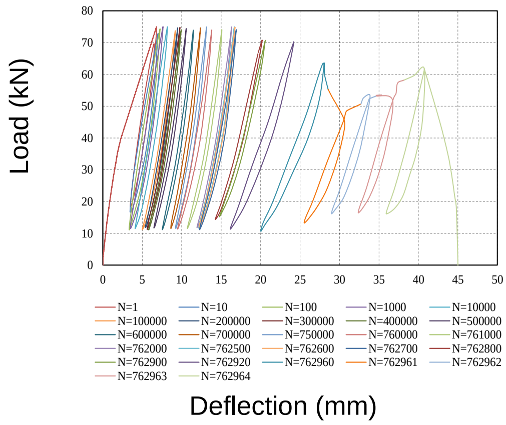



Fig. 13: Load-deflection relationship for BC

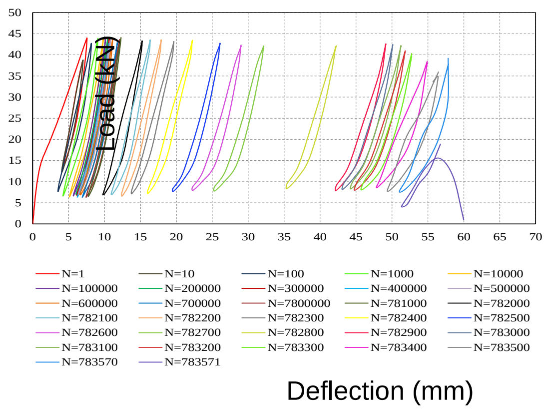



Fig. 14: Load-deflection relationship for BS

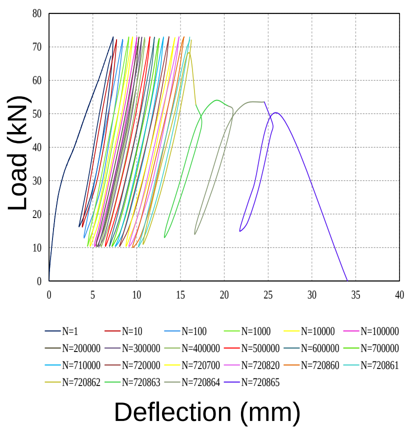



Fig. 15: Load-deflection relationship for BU

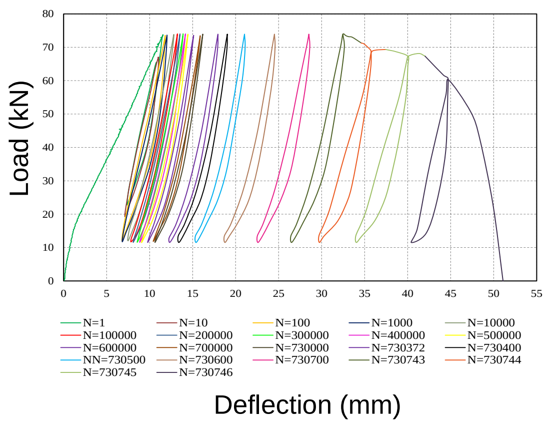



Fig. 16: Load-deflection relationship for BF

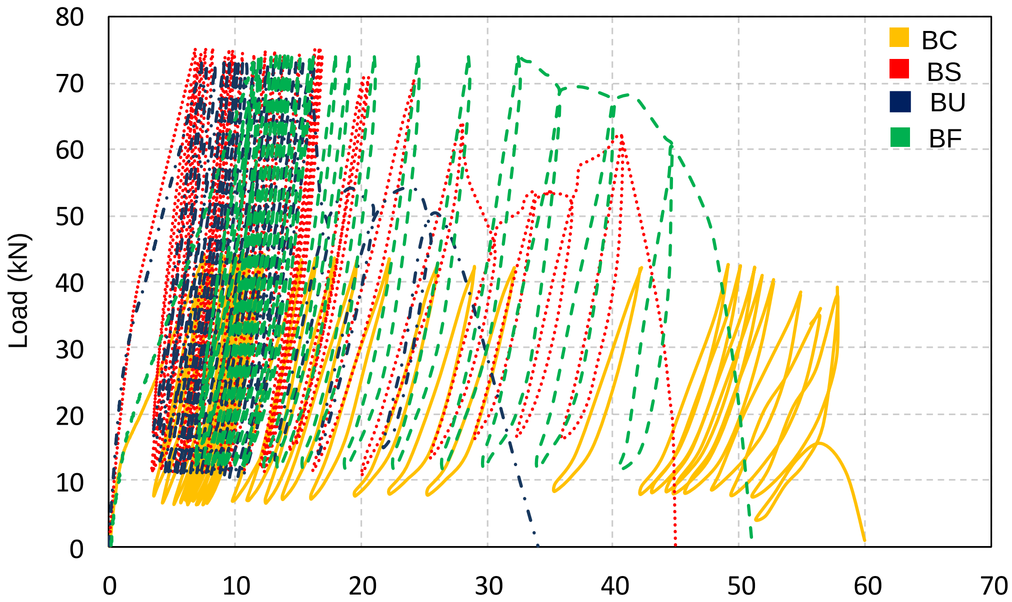



**Fig. 17: load-deflection relationship**

**Damage accumulation**

The damage accumulation was assessed by examining the deflection and stiffness variations during the fatigue loading, a definition used by several researchers [22-24].

**Deflection Measurements**

For each beam, the deflection was recorded until failure occurred. In Fig. 18. It can be seen that in all beams there was an initial increase of the mid span deflection, followed by a stable region where the deflection remained relatively constant through many cycles, followed by an abrupt increase of deflection just before failure. There is a more gradual increase in the deflections as the beam approaches its fatigue life. This may be caused by a greater number of cracks resulting from increased cycling. Moreover Strengthened beams exhibit a deflection versus number of cycles behavior similar to that of the control beam, as the strengthening techniques had a limited effect on the rate of increase of deflection during the stabilization stage of the beams. Both unstrengthened and strengthened beams encountered similar rates of increase in their mid span deflections under fatigue loads during the stabilized period. However, the percentage deflection increase, at the end of fatigue loading, was almost the same for strengthened beams (BS) and (BF). For beam (BU) the percentage deflection increase, at the end of fatigue loading was lower than other strengthened beams. Also the percentage deflection increase at the end of fatigue loading of the strengthened beams was lower than that of un-strengthened beam. This indicates the efficiency of the strengthening system in lowering the damage accumulation. It is worth mentioned that the the fatigue life of the additional reinforcement used in the strengthening layer for beam (BU) is lower than the fatigue life of the main reinforcement. When the additional reinforcement was fractured after 694,000 cycles, the load deflection behavior under fatigue load seems to vary according to the different beam. The number of cycles and the fatigue life of the control and strengthened beams are shown is Fig. (19) and (20) respectively.

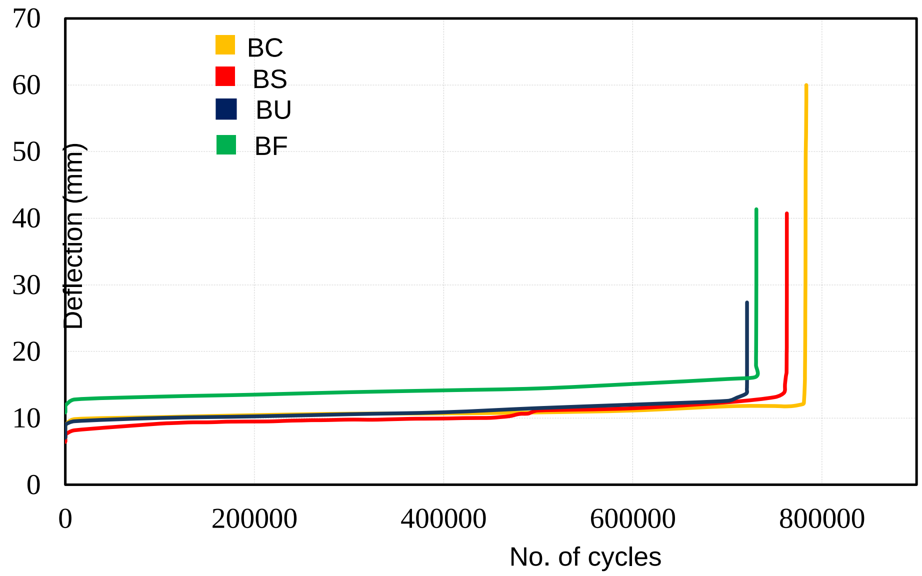
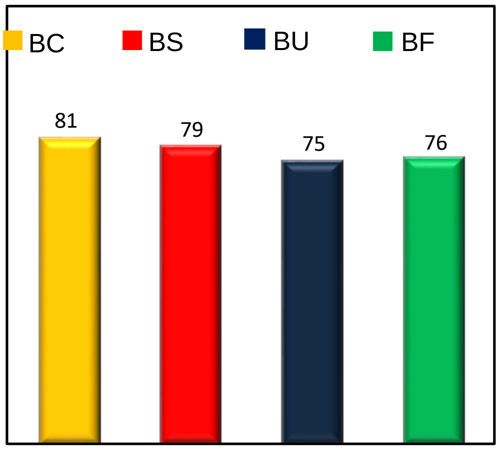



*Fig. 18: Variation of mid-span deflection during fatigue loading.*



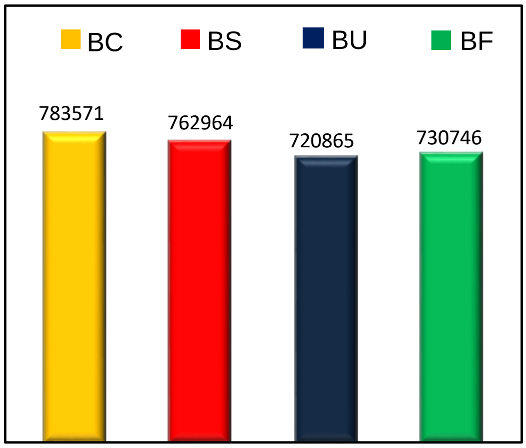




**Fig. 19: Number of cycles**

**Fig. 20: fatigue life**

**Stiffness degradation**

In order to provide a quantitative measure to the stiffness degradation in the beams, the stiffness after each fatigue loading cycle was calculated. The method of calculations involved recording the load and deflection for each specimen after the specified cycle, then the secant stiffness was calculated. Stiffness gets reduced when the beam is subjected to fatigue loading. This reduction in stiffness is due to the following reasons: during fatigue loading, the materials, viz. concrete and steel, are subjected to loading and unloading processes. This will cause initiation of micro-cracks inside the beam and will sometimes lead to the low fatigue limit of the materials. This, in turn, increases the deformations of the beam, thus resulting in increasing the damage level of the beam, which lead to reduction in the stiffness. Hence, it is necessary to evaluate degradation of stiffness in the beam subjected to fatigue loading. The degree of damage in beams can be measured from the degradation in stiffness at different cycles. In order to determine the degradation of stiffness, the values of the secant stiffness obtained for each cycle (Shannag etal, 2005) [25]. The stiffness is computed by taking the slope of the load versus mid-span deflection curves. To compare the reduction in stiffness up to the end of fatigue life with respect to the 1st cycle, the normalized stiffness curves are plotted in Fig. 21 for all beams show a deterioration in stiffness that would have signified immediate failure. In all tested beams most of the stiffness degradation took place in the initial 500 cycles after which the trends were stabilized with respect to the increase in number of cycles until an abrupt stiffness degradation happen during the fatigue failure of the reinforcement. The stiffness of beams (BC), (BS), (BU) and (BF) decreased by 35%, 40%, 44% and 29%, respectively, at the end of the fatigue loading. It is noticed that beam (BF) has a lowest stiffness degradation compared to beam (BS) and (BU).

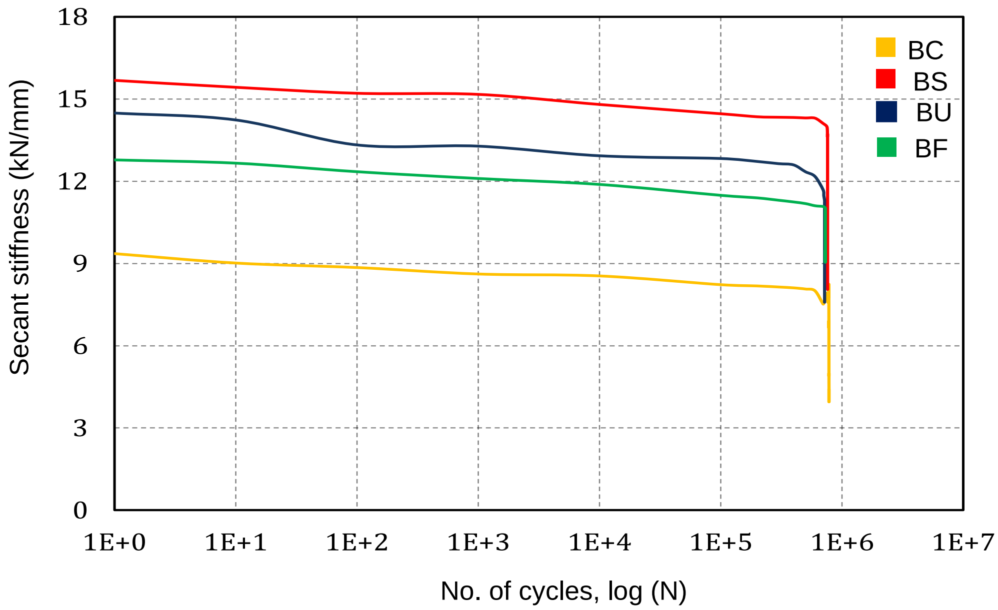



**Fig. 21: Relation between secant stiffness and number of cycles**

**Dissipated energy**

Energy dissipation is a fundamental structural property of RC elements when subjected to cyclic loading. For RC structures the input energy can be dissipated through RC element's hysteretic response, without a significant reduction in strength. The dissipated energy is the area enclosed by the hysteresis loop and represents the structural element capacity to mitigate the fatigue loading effect which causes excessive cracking and permanent deformation. Fig. 22 compare the total dissipated energy obtained from the test results. This total dissipated energy corresponds to the energy dissipated from the start of the test until conventional failure is reached. As shown in Fig. 22 it can be observed that, during first 100 cycles, for all beams the cumulative dissipated energy increased by increasing number of cycles. Cumulative energy dissipation for the control beam (BC) is lower than all strengthened beams. For strengthened beams the increasing of the dissipated energy up to conventional failure is approximately the same with differences lower than 10%. That enhancement in energy dissipated occurred in strengthened beams was due to the contribution of the strengthening systems, which presented in the fibers of the CFRP strips or the fibers inside the matrix of UHP-SHCC layer and the additional steel bars used in this layer, which act as energy dissipaters but for short period in the fatigue life (during the first 10% of the fatigue life) the energy dissipation of beam (BS) and (BU) was smaller than the control beam which was able to made large deflection and lead to widely cracks compared to other strengthened beams that helps to dissipate more energy. During the last 10 % of the fatigue life, it can be observed that all strengthened beams were able to dissipated more energy than the control beam and beam (BC), this is because that the great contribution gained from the additional strengthening system, which obtained larger strain which gained large dissipated energy, however beam (BU), strengthened with UHP-SHCC layer, had rapidly localized stresses due to the rupture of the additional reinforcement after 700,300 cycles which lead to high strain concentration points in the strengthening layer produced by the cracks developed in the concrete substrate. It can be seen that the total energy dissipation by beams (BF), (BS) and (BU) compared to beam BC is 30%, 24% and 23%, respectively. It is important to know that, from the observation of the hysteresis loops, the energy dissipated in the first cycles of the fatigue life was greater than in the subsequent cycles. This is due to the crack development, crack propagation, stiffness degradation and permanent deflection occurred in the first stage of the fatigue life. The possible explanation of greater value of dissipated energy in the first cycles of loading is that when deflection is increased, crack is propagate and the strengthening systems which restrict the crack propagation causing much energy dissipation.

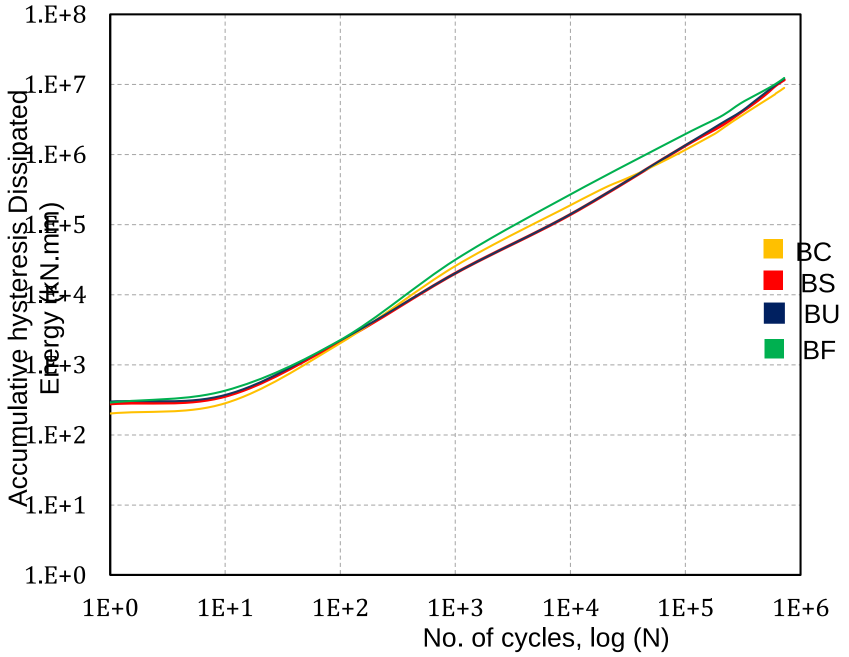



**Fig. 22: Dissipated energy for specimens**

## Conclusion

Based on The experimental test results of RC beams strengthened with different techniques, the following conclusions can be drawn:

- The fatigue life of strengthened beams less than that of unstrengthened beams at the same stress level of the tensile steel bars. This is because the load applied to the strengthened beams will be much higher than those applied to the unstrengthened beams, causing high levels of local shear stress in the reinforcing steel at the locations of cracks. Fatigue life of beam BS is slight higher than the fatigue life of beams BF and BU as the steel plate control the crack widening during the fatigue cycles.
- All beams failed by a fatigue failure in the tension steel reinforcement due to cyclic fatigue loading, so fatigue limit of steel RFT is the major factor contributing to the failure of the strengthened beams.
- All strengthened beams experienced less deflection increase and narrower flexural crack widths lower than that of the un-strengthened beam which indicates the efficiency of the strengthening process in reducing the damage accumulation under fatigue load.
- The stiffness of beams BS, BU and BF decreased by 40%, 44% and 29%, respectively, at the end of the fatigue loading. It is noticed that beam BF has a lowest stiffness

degradation compared to beam BS and BU, which show that strengthened beam using NSM CFRP technique has a good behavior during its fatigue life.

- Near surface mounted CFRP strengthening technique is the most convenient technique compared to the other strengthening techniques for strengthening RC beam subjected to fatigue loads.

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