**The effects of shear stud distribution on the fatigue behavior of steel-concrete composite beams**

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**Abstract:**

Over the past few years, composite steel-concrete beams saw numerous applications in bridge construction because of their competitive cost in comparison to non-composite steel or reinforced concrete structures. Bridges, in particular and industrial structures in general, are commonly subjected to cyclic loading of vehicles or operating machines. This made it important to study and investigate the behavior of composite structures under fatigue. In this study, the cyclic loading behavior of simply-supported composite beams was analyzed numerically using ANSYS finite element software. Following analysis validation using the experimental literature load-deformation results, the numerical models were used in a parametric study to investigate the response under cyclic loading to varying the degree of shear connection between the beam’s concrete slab and steel section. According to finite element simulation results, there were consistent compatibility in terms of stiffness changes with fatigue life, low deflection values, increased fatigue life and a clear delay in reaching the failure under achieving more than 80% of composite action in order to have the optimum performance under fatigue loading. Shear lag phenomenon - which is a good indication of the continuity in strain between concrete slab and steel beam - was also observed and it had an influence effect on studying the weld region between shear connectors’ body as a target element and top flange of the steel beam in order to achieve the required degree of composite action.

**Keywords**

Composite Beams - Cyclic Loading - Fatigue Loads – Numerical Model - Partial Interaction

**Notations:**

**F.E.M.:** Finite Element Modeling

**"S-N" Method:** Relationship between the shear stress range and fatigue life of shear connectors

**Δτ:** Shear stress amplitude (MPa).

**Δ:** Deflection of composite beam (mm).

**N:** Number of cycles (×104).

**δ:** Slippage of concrete slab on steel top flange (mm).

**η:** Number of cycles at first shear stud failure (×104).

**1. Introduction**

Recently, there has been a wide use of steel-concrete composite beams in buildings and bridge construction. Their advantages include high bending capacity and stiffness due to the benefits of composite action and high speed of fabrication and construction.

Despite improved understanding of their behavior, several composite structures failed in satisfying their structural and functional demands due to stud shearing off or concrete crushing as a direct result of fatigue[1]–[3]. In order to prevent these failure modes, Slutter and Fisher[1] conducted experimental tests focused on shear connectors’ mechanical behavior under fatigue loads. They observed two main types of failure modes, the most common was an initial damage in the welding zone at stud root, gradually extending to the steel section's top flange. The second was a failure of reinforcement that appeared and extended to the stud welding zone. The test results led to a logarithmic function relating the shear stress amplitude to the number of loading cycles that can be experienced before failure takes place.

Δτ =1020 n−0.186  (1)

In recognition of this function’s inability to describe the cause and time of failure, or consider important parameters such as the concrete compressive strength, stud dimensions and degree of composite action, Hirokazu et. al.[4] provided arithmetic equations to estimate the fatigue life of shear studs based on regression methods and results of static and cyclic tests. Their recommendations mostly related to the studs’ geometric shape and maximum spacing, based on which they determined the static capacity of the shear stud. In 1997, N. Gattesco et. al. [5] tested 8 specimens exposed to low-cycle fatigue load and it was then widely considered that using “S–N” method was limited at elastic state in structural elements. However, the observations and results showed that in long-span composite beams there were cracks that begins in the concrete near shear studs, which in turn failed as a result of the accumulated slip in the inter-surface between the concrete slab and steel beam and they concluded that inelastic deformation of studs existed. Also Yen et. al.[6] presented experimental results of 44 composite beam structures tested under static and fatigue loads. The main variables which considered during design of the specimens was: shear studs number; method of reinforcing (deformed bars or welded wire fabric); and stress rang of fatigue load. Results showed that cracks were initiated in the concrete slab; propagation and connection of cracks led all specimens to fail; the shear studs become stressed after formation of the cracks; crushing of concrete near the shear studs was observed in some fatigue tests; and the flexural stiffness of the specimens decreased through fatigue tests. Moreover, no practical difference in structural behavior was found between the full composite and the 80% composite specimens. Later, Johnson [7] used fatigue tests on composite beams to predict the fatigue life of shear studs. This expression was later adopted by Eurocode 4 [8].As this equation was based on the traditional "S-N" method. Based on that and in 1997, Grundy and Taplin [9] investigated the effect of cyclic load (monotonically) and fatigue load (reversely) on cumulative slippage at the surface between a steel top flange and a concrete slab. They concluded that monotonic cyclic loading caused much slower growth in slippage than cyclic reversed fatigue loading After 10 years, Hanswille et.al.[10] Conducted practical tests on more than 90 standard samples of push out test and on two full scale composite beams. The main focus was on the growing cracks on studs’ foot and the local damage of the concrete around the studs under the influence high-cycle loading. They came out from these tests with an improved mathematical equation, including a logarithmic function to calculate the number of cycles to failure. Dawood et. al.[11] investigated the fundamental behavior of steel– concrete composite beams strengthened using high modulus CFRP materials. The findings of the experimental program provided comprehensive evidence that HM CFRP materials can be used to increase the elastic stiffness, yield load, and nominal capacity of steel flexural members which were typically used for most highway bridge structures. The presence of the CFRP helped to reduce the residual deflection due to overloading conditions which could help reduce or eliminate the need for repair or replacement of a structure. They also demonstrated that externally bonded HM CFRP materials represent an effective strengthening system for steel–concrete composite highway bridge girders. A later experimental study by Yu-Hang et.al.[12] confirmed the significance of shear stress amplitude on studs on the fatigue life of composite beams. In 2017 Xu et.al.[13] Presented a parametric study on push out test specimens which was mainly to illustrate the effect of fatigue loads (particularly low cyclic loads which are less than 2 million cycles) on a grouped shear connectors .Twelve static and dynamic models of push out test models were used for this experiment. In parallel, they conducted another group of laboratory tests to compare and confirm the compatibility of the results of the theoretical models. The unequal stud shear forces in the grouped studs, which were suggested by analyzing the static failure mode, resulted in reduced stud shear stiffness and strength. For low cyclic loading results, the damage of fatigue initiated at the foot of shear stud section in the direction of push load, which they called it critical point of fatigue. Newly in 2018 Elzohairy et.al. [14] Presented a paper to evaluate the fatigue behavior of composite beams at the sagging moment regions. The fatigue response was predicted using a numerical model for each component of the composite beam. Also, they developed a transformed section method in order to evaluate behavior of the composite section including residual deflection after fatigue loading, bottom flange strain, degree of loss of static strength of the shear studs, and composite beam residual capacity. The main observations were that at starting and the end of the fatigue loading there were a rapid growth in the residual deformations with the number of cycles while a linear increase in the remaining part of the fatigue life occurs. In addition, a decrease in the shear connectors ' static strength is produced with the number of cycles and consequently induces a decrease in the monotonic beam capacity that can be measured based on the new strength of the shear connector.

However, these studies were mostly based on push out tests, which cannot realistically simulate the redistribution of shear after the failure of a number of shear studs, neglect the beam span effect and cannot consider the effect of degree of composite action. This point was considered in this study, where full-scale, composite beams were modeled under fatigue load with multiple degrees of composite action (between 40% and 100%), to predict the failure modes, fatigue life, stiffness changes, strains in the steel section and residual deflection following loading cycles.

**2. Material and Methods**

**2. 1. Verification Study:**

To investigate the fatigue life and rigidity of steel–concrete composite beams that are subject to continuous fatigue load, Tests were carried out by Yu-Hang et. al.[12] on seven specimens, numbered FSCB-1 to FSCB-7, as shown in **Figure 1**. The tensile stress of the steel beams' bottom flange was strictly regulated to avoid the steel's tension fatigue failure, which had been already fully studied in the past. All the specimens were simply supported beams, and subjected to two-point symmetrical load. The load was applied by a 500 kN hydraulic jack. A roller support was placed at one end of the beam for horizontal movement. The loading set-up is shown in Fig. 6(a). As shown in Fig. 6(b), the loading procedures were divided into two stages: Stage1: Fatigue loading—The cyclic loads were sinusoidal, with a constant amplitude, and had a frequency of 4.1 Hz (250 times per minute) which was much different from the natural vibration frequency of beams. Higashiyama et. al.[15] indicated that resonant vibrations would happen when the loading frequency was approached to the natural vibration frequency of beams[15] . Therefore, sympathetic resonant vibrations would not occur in this fatigue experiment. Stage 2: Residual static loading—after the specimens failed in fatigue, they were reloaded with a static loading to obtain their residual static bearing capacity. In the tests, the beams were instrumented to measure the deflections, concrete strains, and strains of the bottom flange of steel beams.

To complement crack investigations, Ovuoba et. al. [16] presented an experimental and numerical study into the behavior of headed shear studs, to address the lack of existing experimental data near the assumed CAFL (constant amplitude fatigue limit), and to better characterize the effects of fatigue uncertainty on predicted response.

The following sections describe the fatigue test setup, including specimen geometry, fabrication, instrumentation, and loading.

**Figure 2** shows the experimental push-out specimen geometry, consisted of a rolled W10×54 wide-flange section having 4 headed shear studs and a 6 in. cast-in-place concrete slab on each flange. The chosen geometry for the specimens (called herein double-sided push-out specimens) was based on guidelines for shear-stud testing prescribed in the Eurocode[8]. Double sided push-out specimens are advantageous over single-sided push-out specimens (having a slab on only one side) as they help reduce loading eccentricities and multi-axial stress states within the stud (combined tension and shear). An applied multi-axial stress state in the stud could provide an overly-conservative estimation of fatigue capacity. In this study, a total of 6 double-sided push-out fatigue tests were performed at four different applied stress levels ranging in value from 30MPa to 60MPa. Due to the significant time associated with high-cycle fatigue testing, only two replicate stress-ranges were considered in the test matrix (replicates at 5.8ksi and 8.7ksi).Concrete slabs of the test specimen were designed to represent typical composite bridge conditions. All concrete sections considered normal weight concrete from a standard highway bridge deck mix design, and each concrete section was casted with the beam in a horizontal position.

**(Figure 1 to be inserted here)**

**(Figure 2 to be inserted here)**

**2.2. Numerical Model:**

A non-linear finite element model was performed using the software package ANSYS. A three-dimensional (3D) finite element model was developed to simulate the geometric and material non-linear behavior of the composite beams. The finite element models for the various components that make up the concrete-steel composite beam are presented next.

**2.2.1. Modeling method and interface:**

The finite element method is an approximate technique, and as such, results computed using the finite element method must be critically evaluated before relied upon in a design application . This process of critical evaluation involves several steps for any structure being analyzed. The number of elements used in a model can greatly affect the accuracy of the solution. In general, as the number of elements, or the fineness of the mesh, is increased, the accuracy of the model increases as well as multiple models are created with an increasingly finer mesh, the results should converge to the correct numerical solution such that a significant increase in the number of elements produces an insignificant change in a particular response quantity. Not all response quantities will converge at the same rate, however,displacements will generally be the most accurate response quantity computed and will converge faster than stresses, with the exception of some elements derived with hybrid stress formulations, in which case the stresses can converge at the same rate or higher than the displacements. Generally, however, convergence of displacements does not guarantee convergence of stresses since stresses are computed as derivatives of the displacement field. Furthermore, it may be necessary to increase the number of elements in areas of the model near either concentrated loads or boundary conditions, where the stress gradient is steeper.

**Concrete modeling**

The F.E.M. of the concrete slab must be suitable for representing cracks, crush failures and shear transfer capability of concrete after cracking occurred. Three dimensional brick element with 8 nodes (SOLID 65) was the selected element to simulate concrete slab. This element is capable of cracking in tension and crushing in compression. The element also consists of eight nodes with three degrees of freedom controlled each node behavior.

**Steel section modeling**

Shell element was used to model the steel beam, which has four-node element having six degrees of freedom for each node. The major usage of this element was for layered applications for modelling composite shells and sandwich structures. The theory of first-order shear-deformation (referring to Mindlin-Reissner shell theory in usual) is controlling the accuracy of modelling composite shells.

**Shear connectors modeling**

Shear connector is the key tool to link steel I- beam to the concrete slab in order to achieve the needed composite action. Most of researchers in the literature review have to use single line elements to model shear connectors. In this study, shear connectors were modeled as a three-dimensional body with multiple elements consisting of stud body, stud root which connect with bonded connection with welding collar modeled with a brittle material property[17].

**Beam-slab interface modeling**

Node-to-node contact element was selected to represent the interface surface connecting the steel top flange and the concrete slab. At each node, this element had three degrees of freedom translation. This contact element can support compression in the normal direction of contact and tangential friction. Preventing penetration ensuring physical separation between the two surfaces was the reason of using this element.

**Reinforcement modeling**

The steel re-bars were modeled in longitudinal and transverse directions using a 3D-link element (LINK8). For each node, cross section area and initial strain, the element was defined by two nodes with three degrees of freedom. It also includes plasticity, stress reinforcement and large deflection capabilities.

**Steel plates at loads points**

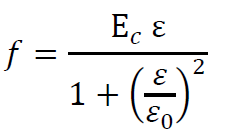
SOLID185 element was used for steel plates at loads points. It is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. The element has plasticity, hyperelasticity, stress stiffening, creep, large deflection, and large strain capabilities. It also has mixed formulation capability for simulating deformations of nearly incompressible elastoplastic materials, and fully incompressible hyperelastic materials. **Figure 3** shows the components of the used finite element model of composite steel –concrete beam

**(Figure 3 to be inserted here)**

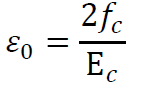
**2.2.2. Material properties:**

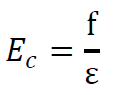
**Material Modeling of the Concrete**

ANSYS requires as input the uniaxial stress-strain relationship for concrete in compression. Numerical expressions (Desayi et. al.[18]) and **Equation 2** and **3** were used along with **Equation 4** (Gere and Timoshenko[19]) to construct the uniaxial compressive stress-strain curve for concrete in this study.



**(2)**

 **(3)**

 **(4)**

The simplified stress-strain curve for each beam model was constructed from six points connected by straight lines; as shown in **Figure 4.** The curve starts at zero stress and zero strain. Point 1 at 0.3fc was calculated from **Equation 2**, while points from 3 to 7 were obtained from **Equation 3**, Points 8 and 9 were the strain at the ultimate strength of the concrete which were calculated using **Equation 4.**

**Material Modeling of the Steel I-Beam**

In contrast to concrete, the mechanical properties of steel are well known. The stress-strain behavior in tension and in compression can be assumed identical. Properties, i.e., elastic modulus and yield stress, for the steel I-beam used in this study followed the design material properties used for the experimental investigation[12]. The bi-linear stress-strain relationships for the web and the flanges represented in **Figure 5** were used in this paper.

**Modeling of Reinforcement**

Since the ordinary steel bars and post-tensioning cable are slender, they can be assumed to transmit axial force only. The stress-strain relationship for ordinary reinforcing steel and post-tensioning tendons were represented as shown in **Figure 5.**

**(Figure 4 to be inserted here)**

**(Figure 5 to be inserted here)**

**Material Modeling of the Shear Connectors**

In practice [20], [8], [21] and [22], shear connectors are never fully rigid, and there is always some slip between the slab and the steel section. The flexibility of the connectors allows more ductility and a variation in the distribution of longitudinal shear between slab and steel section.

The load-carrying mechanism of stud shear connectors is complex and analytical methods for predicting the shear resistance is not applicable. Instead the resistance of the connectors may be determined using empirical formulae or from tests.

The empirical formula according to Eurocode [8] relates to stud and concrete resistance; the design shear resistance is therefore given by the smaller of:

Stud shear resistance:

PRd = 0.8 fu (πd2/4) / γv **(5)** [23]

Where the shear resistance of the shear connector is related to the tensile strength of the steel fu, using a factor of 0.8; concrete resistance:

PRd = 0.29 α d2 √(fckEcm)/γv **(6)** [23]

Where:

d is the diameter of the shank of the stud (not greater than 22mm).

fu is the specified ultimate tensile strength of the stud material (not greater than 500N/mm2).

fck is the characteristic cylinder strength of concrete at the age considered.

Ecm is the mean value of the secant modulus of concrete.

and  is given by:

for 3  h/d  4;

 = 0.2 [(h/d) + 1] **(7)**

Where h is the overall height of the stud,

For h/d > 4; d = 1.

The partial safety factor γv is normally taken as (1.25)

Previous researches[17] has been guided and used as a representation of welding under the stud .This is to ensure a good and realistic representation of the real composite beams. The approximate minimums of the stud welding toe (h), the welding height (v), the welding thickness (t) were respectively around 3.2 mm, 3.8 mm, and 3.2 mm. **Figure 6** show the components of the modeled shear stud.

**(Figure 6 to be inserted here)**

**2.2.3 Analysis assumptions**

The following are the analysis assumptions in this study:

* + The bond between concrete and reinforcement is assumed to be perfect.
  + Poisson’s ratio is assumed to be constant throughout the loading.
  + Time-dependent nonlinearities such as creep, shrinkage, and temperature change are not included in this study.
  + The concrete cylinder compressive strength fc' is assumed to equal (0.8 fcu).

**2. 3. Finite Element Model Verification:**

The finite element analytical models were validated against experimental results obtained from two earlier studies. The results included those of a beam (FSCB-2) tested by Yu-Hang et. al. [12], for which full details were available, As shown in **Figures (7 and 8)** and **Table1**,The model predictions were in good agreement with the data obtained from experimental test in terms of deflection and slippage; but was stiffer by 5and 8% respectively in both linear and nonlinear behavior stages. The reason for this difference could be the assumption in the numerical model of the perfect bond between concrete and the steel re-bars. The residual mechanical static deformation of composite beams was obtained by applying a monotonic static load after reaching the fatigue failure of the beam. Similar to experimental observations, no concrete cracks could be predicted numerically at the start of the fatigue loading, as the composite beam’s neutral axis existed within the depth of the steel section.

**(Figure 7 to be inserted here)**

**(Figure 8 to be inserted here)**

The second test considered was one of the push-out tests performed by Ovuoba et.al. [16] (Slab 2 of Specimen 1). This test was considered in the model validation in order to assess the quality of representation of sliding between the steel section and the concrete slab under fatigue loads. As shown in **Figures** (**9 and 10)** and **Table1**, the finite element model was stiffer by 9% in the linear stage, from the beginning of loading up to 3×106 cycles when the first noticeable slippage occurred at the beginning of the nonlinear stage. Following this point, the numerical predictions of cumulative slippage deviated gradually from the test results, possibly due to the perfect bond between studs and steel section. This difference continued to grow until failure took place at 13×106 cycles.

**(Figure 9 to be inserted here)**

**(Figure 10 to be inserted here)**

**(Table 1 to be inserted here)**

**2.4. Parametric Study:**

There were 13 models in terms of degree of shear connection divided into 13 degrees (from 40% to full shear connection with 5% increment in the degree of interaction). In each type of these models, there were two loading intervals; Interval 1: Fatigue loading and the cyclic loads would be sinusoidal waveform, with a constant amplitude [12].The repeated load Pr (which will be located in rang between (Pmax. and Pmin.)) Will be determined using the beam initial cracking load (Pcr=Pmax. = Pu/3).A residual static loading step will be performed on models which had not suffered a fatigue failure after each 2.5 \*105 repeated cycles. Interval 2: Residual static loading after fatigue failure of the beam, to obtain its residual static capacity, it would be reloaded with a static load. **Figure 11** shows the used cyclic loading pattern and **Figure 12** shows the details of the tested beams.

**(Figure 11 to be inserted here)**

**(Figure 12 to be inserted here)**

The range of fatigue loads was obtained from the shear stress range of 118 MPa for shear connector [16], [24] and according to AASHTO specification. The average fatigue load was obtained from 30% of the static shear strength of the analyzed shear connector [7]. This can be considered a load pattern of high cyclic fatigue with an estimated fatigue life more than one million cycle. We can control the degree of shear interaction by changing the total number of shear connectors over the span length. For the effect of the shear interaction between the top flange of the steel beam and the concrete slab on the overall behavior of the composite beam, thirteen beams with 13 different degrees of shear connection were evaluated under named sections ((HB-N) where, N was the degree of interaction as a percentage from full shear interaction case). The parametric analysis were summarized in details in **Table 2**.

**Figure 13** shows the proposed numerical analysis work in this study.

**(Figure 13 to be inserted here)**

**(Table 2 to be inserted here)**

**2.5. Pitch Variation for Shear Studs:**

Fatigue demands on shear studs are calculated assuming uniform shear flow at the interface between concrete slab and steel beam[8], [25]. It was believed that the horizontal fatigue shear stress amplitude (τ𝑠𝑟) was continuous across the region being studied even though the actual shear transfer occurs at discrete stud locations. A horizontal shear amplitude of fatigue loading was measured using the influence line analysis to determine the total shear demand throughout the length of the beam. In order to improve design-economy and simplify construction, partitioning beam spans and measuring (τ𝑠𝑟) at distinct positions along the beam span are common. This method simplifies beam fabrication and allows for conservancy in the composite load transfer by preventing a continuously varying stud pitch.

τ𝑠𝑟=(𝑉×𝑄)/I **(8)**

In **Equation 8**, V was the vertical shear force range under the applicable load pattern; Q was the first moment of the transformed short-term area of the concrete deck about the neutral axis of the short term composite section; and I was the moment of inertia of the short term composite section [24]. The number of shear studs per length was chosen to be the same in (HB-series) in order to compare the overall shear resistance between tested beams groups (HB and HBM series). This was achieved by grouping studs with higher spacing values.

Currently, the maximum allowable stud pitch ranged from 55 to 60 centimeters in the current AASHTO[20] and Eurocode[26] specification. Some researchers have proposed increasing this limit[27] while researchers have tried to extend the allowed stud spacing there has been limited research to study the impact of this increase on the resulting shear demands. As mentioned earlier, current equations suggest a constant demand of shear flow at the interface between steel and concrete. Furthermore, load was always assumed to be transferred through studs that have distinct locations for flange attachment. Research on the maximum allowable stud spacing and the resulting shear demands at the concrete interface is needed to provide guidance on this issue. The above mentioned issues related to the development of the fatigue capacity equations and maximum stud spacing, combined with the lack of empirical evidence suggesting any fatigue issues within existing composite beam studs. In order to study this point, the test was based on four additional beams. The number of shear studs between the two loading points (zero shear region) was adjusted by half, but the number of studs outside this region remains the same (not increased).These modified beams (HBM-100, HBM-80, HBM-60 and HBM-40) were divided according to the degree of shear connection between the concrete slab and the steel beam. **Figure 14** shows discrete pitch variation according to shear force distribution and **Table 3** shows the detailing of HBM-Models.

**(Figure 14 to be inserted here)**

**(Table 3 to be inserted here)**

**3. Results and Discussion**

**3. 1. Results of Uniform Stud Distribution**

**3.1.1. Failure modes:**

Although a ductile behavior can be characterized in global for all beams, the brittle failure were the main mode of failure in nature, with the shear failure of the connectors. In addition there is no concrete crushing observed, with the exception of the concrete located at the welding foot of the shear connectors.

A similarity in the initial behavior at the beginning of fatigue loading for all beams was observed, followed by a reduction in the rigidity and capacity of the beam by decreasing the degree of shear interaction with the initial separation between the top concrete deck and the steel beam. For the lower degrees of the shear interaction (lower than 80%) the capacity of the beam were affected mainly by the resistance capacity of shear connectors which leads beam to failure before the it could realize the overall composite action and sectional strength, as it can be observed from **Figures (15 and 16)**.So, the recommendation would be always achieving 80% or more than it of the degree of shear connection between the concrete deck and the steel beam in order to obtain the full benefit of the composite action.

It can be seen from **Figure 17**, the effect of uplift forces which developed between the steel top flange and the R.C. slab along loading span which leaded to uplift deformation.it was observed that uplift deformation of the tested beams had the same variation law. As well as the curve derived from theoretical calculations .And the main bearing of studs was tension and stud near the support has the maximum value of uplift value. Comparing beams HB-100, HB-90 and HB-80 with the other beams, the value of uplift of these beams were below the theoretically calculated uplift. This indicated that the value of uplift force is relatively small in composite beam with high levels of shear connection (more than 80%). **Figure 18** shows the deformation occurred in the welding zone at the toe of the shear stud and the beginning of failure at it as aresult of fatigue loading for HB-100 (This mode of failure is consistent with laboratory results for the same beam from experimental tests of Yu-Hang et. al.[12]

**(Figure 15 to be inserted here)**

**(Figure 16 to be inserted here)**

**(Figure 17 to be inserted here)**

**(Figure 18 to be inserted here)**

**3.1.2. Fatigue life:**

From this study it’s indicated that fatigue failure of shear connectors was essentially the only failure mode. Such an irregular cyclic strain and stress shear foot distribution, which resulted in higher strain levels than expected due to the varying distribution of the shear force on the interface between the shear connectors with the number of cycles due to the varying degree of shear connection and the damage region of the shear connectors, as shown in **Figure 16**, which developed in the steel beam top flange. The estimated shear connectors fatigue life herein was the number of cycles corresponding to the crack initiation and failure of the welding region, which was the bottom area around the shear connector foot. Therefore, **Figures (19 (a** and **b))** showed the fatigue life and stress amplitude of shear connectors resulting from the fatigue load in relation with degree of shear connection. The curves indicated that the degree of composite action had important influence on the fatigue life and shear stress amplitude. As the degree of interaction decreased the shear stress increased due to the redistribution and over stresses on the connectors, on other hand a reverse relation between fatigue life and degree of interaction assure that achieving full interaction is the only way to have the ultimate capacity and rigidity of the composite section.

**3.1.3. Deflection changes for beams:**

It’s observed that, because the loading cycles inflated the deflection of composite beams increased additionally. For multi degrees of shear connection, the growth rate was rapid at the start of loading, and slowed when the loading cycles gradually increased. Meanwhile, the changes in beams stiffness and its effect on composite action under the fatigue load were influence factors on the rate of growth of deflection for composite steel concrete beams, and was more obvious when the degree of interaction was low. The deflection of all tested beams was practically linear before longitudinal reinforcement was yielded. When the applied cyclic load was between 20 and 100 × 104 cycles, the load-deflection relation of beams (from HB-100 to HB-90) were almost the same. However, in the yield stage of the tested beam, the rigidity of the (HB-100 was gradually greater than HB-90, which can be attributed to the beam fatigue test. Also it should be noted that after 250×104 repeated cycles, the lower degree of interaction beams (HB-70 and lower) had good ductility. **Figure 20** shows cumulative deflection versus number of loading cycles for all tested beams and **Figure 21** shows the deflection range for all tested Beams.

**(Figure 19 to be inserted here)**

**(Figure 20 to be inserted here)**

**(Figure 21 to be inserted here)**

**3.1.4. Stiffness changes for beams:**

For beam HB-100, there was no obvious change in stiffness before 50×104cycls. After that, a gradually decreasing in stiffness began to be more distinctive. There were a depression of the stiffness by 16% after a crack appeared in concrete slab due to initial slip for shear connectors and expanded and reduced to 76% when the beam failure occurred. For beams (from HB-90 to HB-80), the stiffness remained similar to HB-100 for the majority of lifetime of the fatigue loading. After the shear connectors begin to fail (indicated by observation of initial crack around weld region), the stiffness began to decrease with obviously observation by approximately 22% when an initial concrete cracks started to appear around broken shear connectors, after which the stiffness started to decrease with a rapid rang. For beams (from HB-60 to HB-40), stiffness was almost as close but there was a sudden drop by 26% and the reason this was attributed to the increase on residual deflection at this stage and the beginning of the disappearance of the composite action. Even after that, the broken shear studs and increasing deflection didn’t had an important influence on stiffness because of composite action effect were already existed, in addition the stiffness had decreased by 34% when the beam reached its fatigue life and failed. Normalized stiffness for each beam with the number of cycles were shown in **Figure 22**.

**3.1.5. Strain changes at the mid-span of beams:**

To determine the strain on the steel beam at different points along the height of steel beam section and to observe the occurrence of shear-lag phenomenon (over lapping shear strain and stress between constructional connected surfaces) at the contact region between the steel beams and shear connectors, strain values were detected for the steel beams, as shown in **Figure 23 (a, b and c)**. There were a linear strain distribution along the steel beam section but also it seems to be there is some irregularity of strain continuity on the interface between the surface of the steel compression top flange and shear connectors. According to initial design and the assumption of fully interaction case, the calculated neutral axis was 94 mm from the top of the concrete slab, or 16 mm above the top flange of the steel beam. Prior to the application of fatigue loading, the beam does not exhibit full composite interaction. Instead, there is a strain discontinuity and the bottom of the concrete slab was still in tension. This significance of discontinuity could be titled the slip strain. Slip values like those in **Figure 15** exist when the slip - strain is incorporated along the beam length. Since the slip was observed along the length of the beam, there was no surprise that full composite interaction does not exist and the existence of neutral axis in the steel section as a result.

In **Figure 23-(d)**, the comparison between the actual compost action at the start of fatigue loading and the residual value of the composite action at the end of fatigue loading (at failure) (which calculated by comparing actual stresses on the composite section and the section's static bearing load after end of fatigue loading), shows that the loss value was linear in a clear way from full composite action case to 60% of composite action case. After, that the decline was greater and the curve began to take a more downward trend and the value of the decline was steady. This was due to the move of the neutral axis to the web as well as the small effective section at this interval. In all cases, the European code [6] in its latest recommendations has set the minimum allowable value of the composite action, which is 40%. Therefore, the proposed recommendation of the current results was possibility of return to statically reloading beams that have a compost action more than 60%.

**(Figure 22 to be inserted here)**

**(Figure 23 to be inserted here)**

**Figures 24** and **25** show the strain profiles for various cycle counts. When compared to **Figure 23** it was apparent that less composite interaction exists prior to the application of fatigue loading. Additionally, Compared to the high compost action levels (more than the 80%), the percentage of rapid decrease in the compost action of these beams was greater in the other beams, which the percentage of compost action was less than the permitted level in the applicable codes, if we also take into consideration that the composite neutral axis in these beams was already existed under the concrete slab. In addition to the importance of studying the chronology of the loss of compost action and that the assumed composite section became and acts only as steel section. According to the last mentioned results, shear lag phenomenon was already existed and clearly obtained also it had an influence effect on the welding region between shear connectors body as a target element and top flange of the steel beam. Furthermore it leads the focus of upcoming studies on strengthening this region for a better resistance performance of shear stud. The indication of the assumption that plane sections remain plane[14], [28] was also agreed well with the used steel cross section.

**(Figure 24 to be inserted here)**

**(Figure 25 to be inserted here)**

**3.1.6. Concrete fatigue damage accumulation feature:**

The analyzed thirteen beams under load cycle accumulated fatigue induced crush distributions of the concrete at the side of stud roots are shown in **Figure 26**. The analyzed concrete damaged regions were contoured by variable colors, which were presented as slices at critical sections. The concrete surrounding the outer studs was severely damaged in a group than the concrete surrounding the inner studs. According to the shape of the expected shear force diagram, the force and direction of shear flow was the main determinant of the internal deformation of the concrete slab. However, there is no substantial difference between the tested beams in this regard. Also, the effect of moving from a minimum value to a maximum value for shear forces in the form of deformation, whether from the top to the bottom in the direction at hinged support region or vice versa for the Roller one.

**(Figure 26 to be inserted here)**

**3.1.7. Evaluation of results on codes:**

**Figure 27** shows a comparison between S-N curves for AASHTO and European standard and values resulted from tested finite element models (number of cycles N and shear stress range and Δτ).The equation in AASHTO had the closest guarantee rate for predicting the fatigue life of studs from the comparison results in **Figure 27**, and with 72.6 percent of the predicted results, it was secure. The guarantee rate of the equations for steel-concrete composite beam in Eurocode 4 was similar but with a lower predictive efficiency, which gives preference to the use of AASHTO equations. Although the guaranteed rating for the equations in AASHTO and Eurocode were reasonably high, the discreteness still existed in the results of the finite element compared to the curves of prediction from these codes. This was due in part to the various stress states in the reported literatures induced by the various loading modes and devices. Furthermore, the discreteness of the material fatigue property of studs and concrete also influenced the test results of studs subjected to fatigue shear load. In addition, several other factors influenced the fatigue behavior of the studs, such as the initial imperfection of the material, the directions of crack propagation, which were very difficult to assess.

The shear stress amplitude versus fatigue life relation of the studs was plotted in **Figure 28**. From the results it can be seen that the shear stress amplitude of studs Δτ had an important influence on the fatigue life of specimens. **Figure 28** indicated that the fatigue life of the specimens increased with the increase of degree of shear connection which also entails an increase in the shear stress amplitude of studs. However, the apparent discreteness still persisted as compared to the regression equation shown in **Figure 28** for the relationship between Log Δτ and Log N.

**(Figure 27 to be inserted here)**

**(Figure 28 to be inserted here)**

**3.2. Results of Pitch Variation for Shear Studs:**

In general, when reducing the number of studs in the zero shear region by half, while keeping the total number as it was (and consequently increasing the degree of shear connection in the regions with the highest impact of shear forces), there was an obvious improvement in varying rates in increasing number of cycles and residual deflection but this increase was not apparent in the higher degrees of shear connection (more than 80%) with the noted increasing the range of shear amplitude on the studs, which usually result from increasing the loading life.

On the other hand, the increase in the number of shear studs and the distance distribution resulted in a noticeable improvement for the lowest levels of shear connection (lower than 70%) and it was observed clearly in the number of loading cycles and stiffness and shear capacity of the shear studs. Comparing the uplift values resulted from the loading HBM-series, it was noted that the values are slightly lower than their normal counterparts (HB-series). This is due to the unequal distribution of the shear studs which was based mainly on intensifying the number in the loading regions, allowing greater resistance in the supporting regions and consequently less uplift value. In any case, the **Table 4** and the following curves showing **Figures 29, 30, 31, 32, 33** and **34** summarized the results in this regard and compared them with the same beams with uniform distribution of shear studs to show the differences between them. As for the failure mode in beams of this variable (Pitch Variation for Shear Studs).

**(Table 4 to be inserted here)**

**(Figure 29 to be inserted here)**

**(Figure 30 to be inserted here)**

**(Figure 31 to be inserted here)**

In spite of using the same loading pattern for the previous samples (HB-series), the increase of shear stresses on the shear studs as a result of the redistribution led to the appearance of new cracks quickly (appeared early compared to the number of cycles) at the stud foot area, which is one of the patterns expected during this type of loading and is also compatible with the three types mentioned in Yu-Hang et. al.[12]. See **Figure 35** which clarifies the distribution of the shear stresses on the stud body, as well as the beginning of the appearance of a failure in the interface between stud body and welding area at 2.5×104 cycles.

**(Figure 32 to be inserted here)**

**(Figure 33 to be inserted here)**

**(Figure 34 to be inserted here)**

**(Figure 35 to be inserted here)**

**4. Conclusions**

In this paper, a nonlinear finite element analysis had been conducted for steel-concrete composite beams under cyclic loading. Using existing test results, the finite element model was validated with a good validation agreements. A parametric study was also carried out to investigate the performance of composite beams under multiple levels of shear interaction and with pitch variation for shear studs. The main conclusions were as follows:

A good agreement was obtained between the presented finite element model and the test data, which ensured the reliability and accuracy in terms of expectation and analysis of the cyclic loading behavior of the composite beams under varied degrees of shear connection. According to the available results, which showed that there were consistent compatibility in models (HB100, HB-90 and HB-80) in terms of Stiffness changes with fatigue life, low deflection values, increased fatigue life and a clear delay in reaching the failure time, it is recommended that it must be at least achieve 80% or more of composite action in order to have the optimum performance under fatigue loading. According to observed results , for high levels of shear interaction (80% and more) failure mode of was the fatigue cracking on welding collar followed by the shear connector fracture, and that would be when the upper fatigue limit was equal to 56% of the ultimate load. Also load-deflection curves for all tested beams contained an elastic part, plastic part and descending part. In the elastic part, the load-deflection curve shows an almost linear relationship especially in the first (80 × 104) cycles. In the plastic part, the deflection increased slowly, while the stiffness reduced much more significant rate, and it was more obvious in lower degrees of interaction (less than 60%).It was clearly observed that not all of the maximum slips and shear capacities of studs in the tested beams had reached, especially those between loading points region and it was at all degrees of interaction. Hence, it is significant that using the minimum number of shear studs at this region would be better for this loading type to ensure the connection at this area and the ductile behavior of studs.

The redistribution of shear studs along the length of the beam to suit the requirements of shear force contributes to the improvement of the properties of the landing and tetanus clearly as follows:

The increase in fatigue life was varying from 1% to 2.4% but it was more obvious in the case of 80% degree of shear connection. Also deflection was almost the same for most cases, although the number of cycles increased but the increase was even greater in beams with less than 60% of shear connection due to the rapid loss of compost action other beams. In addition the increase in shear stress amplitude was the largest in the case of 40% shear connection, this was logical, although this increase compared to the cases with uniform studs distribution was greater in the case of 80%shear connection. This highlights the importance of keeping the degree of interaction that is never less than 80%. Also the steady increase in stiffness was the most prominent factor in explaining the importance of re-distribution of the shear studs again for greater resistance to the stresses and reduce the strain and the increase had reached 7%, which is a is a good impression in this case. On other hand the reduction of slippage between concrete slab and steel beam with a range of 3 to 7% was clearly a result of the increase in the number of cycles at the first failure of the shear studs and this gave the opportunity to quantify the resistance more and increase the fatigue life.

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**7. Data Availability Statement**

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

**8. Conflict of interest statement**

On behalf of all authors, the corresponding author states that there is no conflict of interest.

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